THE LOAD - CARRYING CHARACTERISTICS OF A CONCRETE RESURFACING MIXTURE

FEBRUARY 1957
No. 7

by

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Joint Highway Research Project

PURDUE UNIVERSITY
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THE LOAD-CARRYING CHARACTERISTICS OF A
BITUMINOUS CONCRETE RESURFACING MIXTURE

TO: K. B. Woods, Director
Joint Highway Research Project

February 21, 1957

FROM: Harold L. Michael, Assistant Director

File: 2-4-6
C-36-6F

Attached is a report entitled "The Load-Carrying Characteristics of a Bituminous Concrete Resurfacing Mixture" by J. F. McLaughlin, of our staff.

This report presents the results of a laboratory study on certain of the load carrying characteristics of a bituminous concrete overlay.

This study was also utilized by Dr. McLaughlin as his thesis in partial fulfillment of the requirements for the Ph.D from Purdue University.

Respectfully submitted,

Harold L. Michael

Harold L. Michael, Assistant Director
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HLM:hgb

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FINAL REPORT

THE LOAD-CARRYING CHARACTERISTICS OF A BITUMINOUS CONCRETE RESURFACING MIXTURE

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Project C-36-6F
File: 2-4-6

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February 21, 1957

TABLE OF CONTENTS

LIST OF TABLES

Table | Page

LIST OF FIGURES

Figure | Page

ABSTRACT

McLaughlin, John Francis. Ph.D., Purdue University, January 1957. The Load-Carrying Characteristics of a Bituminous-Concrete Resurfacing Mixture. Major Professor: W. H. Goetz

This report presents the results of a laboratory study which was carried out in order to investigate certain of the load-carrying characteristics of a bituminous-concrete overlay.

The study was subdivided into two parts. In the first part all tests were performed on specimens molded in the laboratory; in the second part, pavement cores for laboratory tests were obtained from a section of pavement which had been resurfaced with bituminous concrete in the summer of 1950.

The bituminous-concrete mixture which was used for the laboratory-compacted specimens was first characterized by performing several series of conventional laboratory tests. Included were the Marshall test, ASTM Compression test, triaxial test, and the Hveem Stabilometer and Cohesiometer tests. Further tests were performed on laboratory-compacted specimens in order to estimate the load-carrying capacity of a relatively thin layer of bituminous concrete when loaded over an area that was small with respect to the size of the specimen and, hence, to estimate the "confinement" which may be present in a thin bituminous-concrete overlay. In addition, the relationship between the strength of a bituminous concrete mixture and the variables of rate of deformation and temperature was investigated by means of compression tests on specimens two inches thick and four inches in diameter compacted by two different methods.

A similar investigation of the strength- rate- temperature relationship was made on pavement-core samples of two-, three-, and four-inch thicknesses. Pavement cores were also subjected to two types of repeated load tests (called rapid-cycle and slow-cycle tests) in order to study the effect of number of load repetitions on cumulative permanent deformation of the specimen. In addition to specimen thickness, the variables of temperature and stress intensity were studied in these tests.

It was found that for the materials, mixtures, and methods used in this study:

1. The load-carrying capacity of a bituminous-concrete mixture, compacted into a relatively thin layer and loaded at a constant rate of deformation over a small portion of the total surface with a plate which is relatively large with respect to the layer thickness, appears to be several times greater than the unconfined compressive strength of the mixture for the same temperature and rate of deformation.

2. The "confinement" in a relatively thin pavement layer of bituminous concrete, loaded over an area that is large with respect to layer thickness, appears to be considerably in excess of the unconfined compressive strength of the mixture.

3. The relationship between the strength of bituminous concrete and the variables of temperature and rate of deformation is of the form

$$x_o = A^{Bx_1^{(Cx_2 + D)}}$$

where x_o = maximum compressive strength

x_1 = rate of deformation

x_2 = temperature, and

A, B, C, D = constants

4. With respect to the repeated load tests, it appears as if the relationship between cumulative permanent deformation and number of load repetitions for the rapid - cycle test results, in the region before failure of the specimen is of the form

$$y = kx^{\frac{1}{n}}$$

where y = cumulative permanent deformation

x = number of load repetitions, and

k and n = constants.

In addition, the rapid-cycle repeated load test appeared to be a promising method for the evaluation of the plastic nature and endurance properties of bituminous-aggregate mixtures.

INTRODUCTION

The use of bitumen-aggregate mixtures as overlays on aging portland-cement concrete pavements has, in the past ten years, become an item of major importance in the highway programs of many states. These overlays, intended to extend the useful life of a pavement, must provide all of the qualities for a riding surface that safety, economy and the American public demand. The overlays must cover-up the faults of the old pavements and must be slow to develop faults of their own in order to justify their use.

Bitumen-aggregate overlays may be of several types; in fact almost any one of the whole range of mixtures possible with the variety of bituminous materials available for use with different aggregates and aggregate gradings could be used for resurfacing purposes. However, this study is primarily concerned with bituminous-concrete overlays. More specifically, the study deals with asphaltic concrete made with 60-70 penetration grade asphalt cement and of an aggregate gradation and asphalt content that is commonly used by the State Highway Department of Indiana.

The bituminous-concrete overlay most used in Indiana is composed of two layerst (a) a binder or leveling course which has a maximum aggregate size of 3/4 inch to one inch, 65 percent coarse aggregate (material retained on the No. 6 sieve), 35 percent fine aggregate essentially none of which passes the No. 200 sieve, and usually contains 4.5 to 5.5 percent asphalt by weight of the mixture, and (b) a surface course which has a maximum aggregate size of 1/2 inch, about 50 percent coarse aggregate (material retained on the No. 6

sieve), 50 percent fine aggregate with about three percent of this passing the No. 200 sieve, and usually contains 6.0 to 7.0 percent asphalt by weight of the mixture. The thickness to which each of these courses is laid is variable depending upon the condition of the road to be resurfaced, the expected traffic intensity and perhaps other factors, but a total thickness of 2 1/2 inches being composed of 1 1/2 inches of binder and one inch of surface is not uncommon.

A primary consideration in the use of the mixtures described above is stability, the name given to that property of the material which permits it to carry the loads imposed upon it in service without disrupting to a point where the mixture no longer can fulfill its function. Much effort has been expended, first, to determine realistically the stability requirements of a bituminous-concrete overlay and, second, to develop a method for measuring this stability in the laboratory so that a mixture might be designed on a sound basis. A considerable amount of work has been done at Purdue University dealing with mixture evaluation by the conventional methods such as Marshall tests and unconfined and confined compression tests. It has been found that the type bituminous concrete used by the State Highway Department of Indiana generally meets the stability requirements of these tests. However, the performance of certain of the mixtures in some road locations has been unsatisfactory, thus indicating that the conventional laboratory tests are not adequate to measure stability in the complete sense of the word.

Rutting in Bituminous Concrete Overlays

The evidence of lack of stability in some bituminous-concrete overlays of the type described is the development of ruts in the overlay in the wheel track areas. This rutting, first investigated in the field in late 1953, was found to be present to some extent in every overlay inspected. However, it was present to an extent that could definitely be judged to be objectionable in relatively few areas each of which was subjected to much heavy, relatively slowly moving traffic. The condition was found to be the most severe at signalized intersections where the pavement and the overlay were subjected to stresses from braking traffic and to static loads. An example of this may be seen in Figure 1.

Figure 1 shows a view looking west at the west-bound lanes of U. S. 12 and 20 near its intersection with 7th Avenue just east of Gary, Indiana. The picture was taken in November, 1953, at which time the bituminous-concrete overlay was three years old. The fact that the mixture had been subject to plastic movement seems evident from the shape of the pavement edge. In Figure 2 which was taken of the inside wheel track near the intersection, one can see that the magnitude of the rutting is of the order of 0.1 feet.

Cores taken at this location, both in the wheel tracks and between the wheel tracks, were found to be approximately one inch different in height, the wheel-track core being the shorter. Figure 3 shows a pair of these cores taken at the intersection. As one proceeded back from the intersection, the rutting became less severe and the difference in height between the wheel track and non-wheel

FIG. 1 OVERLAY PERFORMANCE AT INTERSECTION

FIG. 2 RUTTING IN INSIDE WHEEL-TRACK

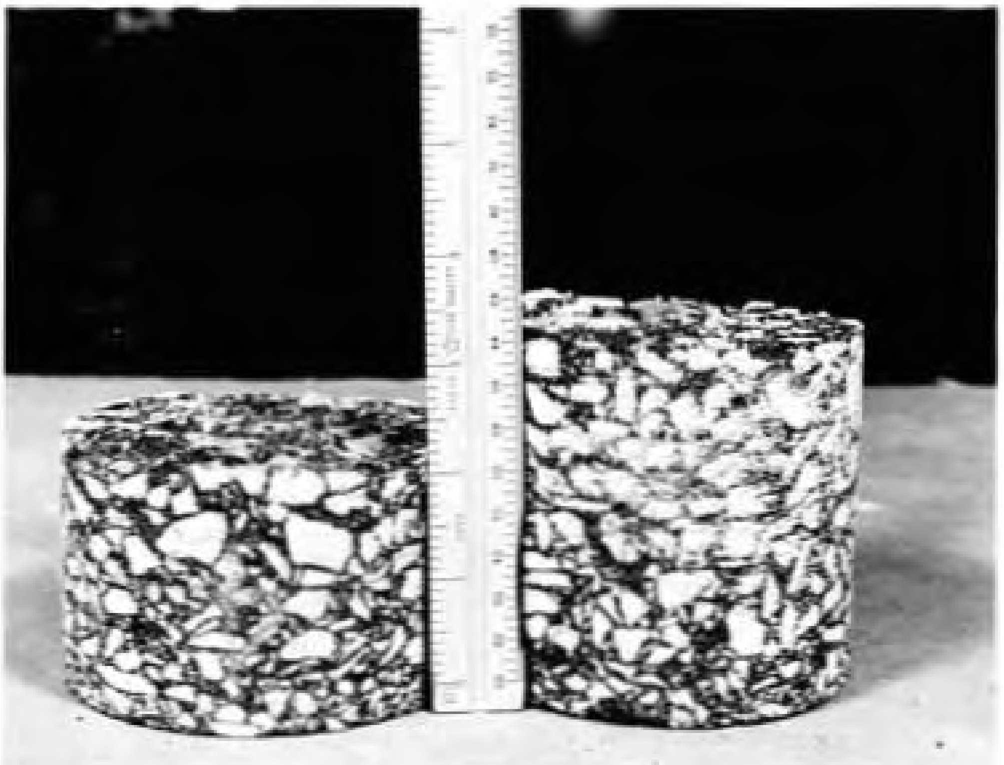

FIGURE 3 HEIGHT DIFFERENCES IN CORES

track cores was less also.

Other locations in the State showed similar results. At one location on U. S. 40 just west of Indianapolis, a trench was cut for the full depth of the resurfacing across two of four lanes of pavement. One very startling result was to find that the overlay had spread out in a transverse direction. That is, while the overlay was placed even with or just slightly beyond the edge of the underlying concrete, it was found at the time of sampling to extend some six inches beyond the edge of the concrete. Also, an examination of the sawed face showed that both the binder and surface layers were distorted in the wheel track areas.

Figure 4 shows a view of one of the two trenches cut in the resurfacing on U. S. 40 west of Indianapolis. The pavement distortion can be seen both from the cut face of the trench and from the water on the pavement which has ponded in the ruts in the wheel track areas. The trench which is shown in Figure 4 was cut approximately 750 feet back from the nearest intersection. Another trench cut at an intersection showed more distortion in the resurfacing than the case illustrated.

In a report to the Advisory Board of the Joint Highway Research Project, Purdue University, dated March 17, 1955, W. H. Goetz and this writer stated the following:

> "The following information concerning AH binder and AH Type B surface has been collected as a result of field and laboratory studies.
>
> a. This type of resurfacing, when constructed with . . . . 7 percent asphalt in the surface and 5.5 percent asphalt in the binder, is plastic enough to displace under repeated,

FIG. 4 TRENCH CUT IN BITUMINOUS OVERLAY

heavy, slowly-moving loads. Depending upon the severity of the exposure, the ruts may become objectionable. The locations in urban areas that have been observed where this condition obtains have developed objectionable ruts.

b. The magnitude of the rutting is such that densification of the mix can account for only a very small portion of the total displacement. Tests have shown that the mixture in areas of severe rutting is quite dense but that this densification is over the entire pavement and not confined to wheel-track areas. Differences in the heights of cores taken from wheel-track and between-wheel-track areas indicated that the total rutting was composed of densification plus lateral movement of the material itself. (An) inability to see the line of demarcation between the binder and surface indicated that both components of the resurfacing were moving.

c. If the Corps of Engineers criteria of minimum Marshall stability of 500 pounds and maximum flow of 20 were used, Marshall tests on the cores indicated adequate stability value and, in general, satisfactory flow values. However, most of the mixtures had a flow of 17 or more and when one considers that these test were made on combined binder and surface, a question of plasticity may be raised.

d. The general conclusion may be stated that the failures under consideration are caused by plastic flow and not by exceeding some maximum shear strength on a critical plane. Trenches cut through the resurfacing on U.S. 40 west of Indianapolis confirmed the fact that both the binder and surface layers are subject to this plastic flow."

The Need for Better Information For Design Purposes

The information presented in the foregoing section would seem to point out an inadequacy in our present design methods for bituminous concrete overlays. Even if the question was answered concerning which method of formation of a laboratory specimen was best, designers are still not sure what laboratory criteria should be used to assure adequate field performance.

In addition, certain anomalies crop up now and then which lead one to wonder if a completely different approach to the problem is not needed. Take for instance the condition shown in Figure 5. Here is pictured a junction between two types of bituminous overlays, the underlying concrete and the imposed traffic being essentially the same for each overlay. The material in the foreground is bituminous concrete binder and surface of the type that has been described; in fact, the photograph shows the eastern terminus of the section where Figures 1-3 were taken. The material in the background is a cold-mixed binder of one-sized aggregate with a rock asphalt surface. It is evident from the photograph that the one-sized aggregate binder is carrying the loads without appreciable deformation while the bituminous concrete is not. However, in the laboratory the bituminous concrete appears to be the stronger, more stable material by all of the conventional test methods. Here, definitely, a fresh approach is needed.

It was these thoughts and the problem outlined that led to the present investigation. Certain fundamental information concerning the behavior of bituminous concrete under load appeared to be needed. Such factors as the influence of temperature and rate of deformation

FIG. 5 COMPARISON OF OVERLAY TYPES

on strength characteristics have certainly been investigated before, but not in a strictly quantitative sense such that the relative effect of each factor at any level could be related to the strength produced. The resistance of bituminous concrete to repeated applications of load also appeared to be a fruitful field for investigation. Other ideas were developed as the study progressed and this thesis reports the results of various lines of endeavor.

The laboratory study herein reported is the second part of a two-part study of the problem. The first study was done by Dr. L. E. Wood (67)[1] who investigated many of the variables mentioned. He used sheet-asphalt mixtures in his study in order to expedite the laboratory testing. It is hoped that the results found by the present investigator will add to and extend the concepts originally proposed by Wood and will also introduce some new concepts to add to the understanding of the behavior of bituminous concrete.

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[1] Numbers in parentheses refer to references listed in the Bibliography.

REVIEW OF LITERATURE

The problem of bituminous mixture design is an old one which has been the subject of a great number of technical papers. Inherent in this problem is the action of a bituminous mixture under load and the attendant effects of temperature, rate of loading, and load repetitions. The design of bituminous mixtures for overlays for portland-cement concrete pavements is not so old. Certainly bituminous mixtures were used over rigid pavements in some instances prior to World War II, but since that time the resurfacing of concrete pavements has become of major importance in the highway program. There are many problems associated with the use of bituminous mixtures for such purposes, some of which have been recognized only recently. The literature on this subject is not so plentiful.

This review is in two parts. In the first part, the literature on the subject of resurfacing is summarized. In the second, the general problem of the measurement of the stability of bituminous mixtures is considered. All available references in the field are not included in this review, but rather it is a restricted coverage which attempts to give a concise version of what has been done on the subject in the past.

Bituminous Resurfacing

Writing in 1949, Gould (18) stated, "Iowa is a fledgling in the field of pavement salvage, having now completed only two seasons of such work." It is believed that this is typical of the depth of experience of many areas of the country in the matter of salvaging concrete pavements by the overlay or resurfacing technique. After describing some techniques and experiences in Iowa, Gould concluded that resurfacing with asphaltic concrete," . . . is justifiable both from the standpoints of good engineering and economics . . . "

Resurfacing is of benefit when pavements have deteriorated to the point where maintenance costs are excessive to keep them in good riding condition. The technique gained popularity rapidly. In 1952, Tittle (55) said, "Resurfacing deteriorating pavements is now recognized and accepted by many states, counties, cities, and the Bureau of Public Roads as an economical, convenient, and highly satisfactory means of salvaging old pavements. It has proven adequate to serve the increased wheel loads now operating on the streets and highways. In addition to providing a smooth, skid-resistant surface, resurfacing imparts considerable strength to the road structure by waterproofing the subgrade, reducing impact stresses, and by the addition of its inherent strength."

Concerning the mixture design to be used for resurfacing, Tittle (55) said, "Laboratory procedure for testing quality of materials has been established and presents no particular problem." It is gratifying to know that some areas of the country seemingly have solved the resurfacing problem, but one questions whether that author would make the same statement today.

Resurfacing has also been employed to strengthen airfield runways and taxiways. As a result of observations of several field installations, Christiansen and Phillippe (9) concluded that for airfield pavements, ". . . the use of a properly designed flexible overlay appears to be the most satisfactory way to increase the bearing capacity of an inadequate rigid pavement."

Experience with bituminous resurfacing in the state of Minnesota was reported by Kipp and Preus (24). It is the practice in that State to use a "single graded" aggregate with 120 to 150 penetration or 150 to 200 penetration asphalt cement for work in rural areas. They reserve the dense aggregate gradations for ". . . extremely heavy traffic in metropolitan areas."

Several papers on various aspects of resurfacing have been published in a recent bulletin of the Highway Research Board (7). This publication deals with such subjects as the conditioning of an existing concrete pavement for bituminous resurfacing, condition surveys of resurfacings, and current practices on controlling reflection cracking.

Since more and more miles of concrete pavement are reaching an age where rejuvenation of this type is needed, it is anticipated that problems associated with bituminous resurfacing will receive more attention in the future.

Stability of Bituminous Mixtures

Many aspects of the stability of bituminous mixtures have been investigated by Vokac (58-63) and by Miller and Hayden in collaboration with Vokac (41). One of Vokac's early investigations was of an impact method for studying the characteristics of paving mixtures (58). The test specimens were two-inch briquettes such as used in the Hubbard-Field test. The samples were repeatedly loaded at different energy levels in a Page Impact Machine, the number of impacts needed to cause failure being measured. Vokac stated that the test would give information regarding, ". . . the 'Impact Resisting Strength' of a mixture, a measure of . . . the 'Degree of Toughness', and also a definite measure of its tendency to deform, i.e., shove in service." He concluded that, "The primary recommendation of the Impact test is the simplicity and directness with which it furnishes information regarding the most important characteristics of an asphalt paving mixture."

An extensive test series was also undertaken by Vokac in an attempt to correlate the results of physical tests made in the laboratory with the observed service behavior of asphaltic mixtures (41, 60, 61). The laboratory stability tests and other laboratory measurements that were considered were Hubbard-Field Stability, Skidmore Shear, compressive strength, density of the mixture, voids in the mixture, elastic limit, modulus of elasticity, and modulus of permanent deformation. Vokac summarized the results by saying, " . . . all of the test methods are useful in differentiating to some extent between good and bad paving mixtures but (it appears) that the compression test requiring no special apparatus is definitely the most reliable for this purpose" (60).

Perhaps the most significant contribution to the understanding of the stability of bituminous mixtures made by Vokac was contained in two successive papers to the American Society for Testing Materials in 1936 and 1937 (59,62). He reported the results of compression tests on asphaltic mixtures, the specimens used being short with respect to their diameter. That is, as expressed by Vokac, the specimens had diameter to height ratios which were generally 1.0 and larger. He investigated such things as the effect of specimen size, density of the mixture, the effect of rate of testing, and the effect of temperature. He proposed equations relating each of these factors to compressive strength but did not propose a general equation relating all of the factors. He concluded that the compression test had considerable possibility for evaluating asphalt paving mixtures. The equations that he derived could be of assistance in evaluating ". . . the constants of a mixture characteristic on the basis of cores cut from a pavement" (59).

One further study, relating to the thermal susceptibility of bituminous mixtures, was made by Vokac (63). Again using the compression test as the method of measurement, he worked with various sources of asphalt and concluded from his results that:

> "1. The susceptibility of a mixture increases with bitumen content at temperatures in the neighborhood of 77 F. and lower. At temperatures approaching 140 F., the order of susceptibility is reversed.
>
> 2. Although, in general, additional filler decreases the susceptibility of a mixture made with a given asphalt, this effect is less evident at low temperatures.
>
> 3. The source and method of processing an asphalt markedly influences the susceptibility of a mixture in which it is used."

This reference (63) also contains a statement which is perhaps obvious but which may be overlooked too frequently:

> "We may logically infer, therefore, that in order to obtain mixtures having similar physical characteristics such as (thermal) susceptibility, strength, voids, etc., using asphalts from different sources, the mixtures must be proportioned individually to suit the unique characteristics of each asphalt."

The results of direct shear tests on sheet asphalt mixtures made with 50-60 penetration asphalt cement were reported by Skidmore (50). These tests were performed at temperatures between - 20 and + 32 ° F. It was concluded that special design considerations should be given to mixtures that will be exposed to low temperatures.

In an approach to analytical testing of bituminous mixtures, Endersby (13) outlined the relative importance and the functions of the components of a bituminous-aggregate mixture. Lewis and Welborn (26) studied the effects of asphalt characteristics on the physical properties of bituminous mixes and found that, in the temperature range - 25 to 140° F, a plot of log Hubbard-Field stability versus temperature was linear for stability values lower than 10,000 pounds. Waller (64) investigated the ASTM compression tests and found what effects compactive effort and rate of deformation had on compressive strength.

Several studies of the strength of bituminous mixtures were made by Mack (27,28, 29). In one study (27) he considered the rheology of bituminous mixtures by determination of: a) elastic flow, b) plastic flow under continuous and successive compressions, c) stresses set up in specimens during flow, and d) rate of dissipation of stresses.

Mack also wrote concerning the bearing strength of road structures (28) in which he reports the results of a type of repeated load test. However, this testing constituted a very small part of the research and no conclusions could be drawn other than the fact that only part of the total deformation of the bituminous mixture was recoverable upon the removal of the load.

In a recent contribution to the literature, Mack (20) investigated both the deformation mechanism and bearing strength of bituminous pavements. Concerning this, Mack stated:

> "The deformation of bituminous pavements consists of an instantaneous and retarded elastic deformation followed by a plastic deformation. The mechanical behavior is primarily determined by the plastic deformation which is accompanied by hardening."

Nijboer(43) used the triaxial test to study the plastic properties of bituminous-aggregate mixtures. He investigated the influence of asphalt cement, coarse aggregate, and fine aggregate and reports that triaxial test results should be useful in design. The same author recently presented a summary of numerous proposed rational approaches to bituminous road design (45). In this study he also reported some data on fatigue of asphaltic mixtures and found that bituminous mixtures showed fatigue properties comparable to other materials such as steel in that repeated loading at higher stress levels cause failure after a smaller number of load repetitions than would be the case at lower stress levels.

A theoretical approach to the design of bituminous mixtures using the results of triaxial stability tests has been presented by McLeod (37-40). He stressed the fundamental inadequacies of the empirical tests commonly used for bituminous mixture design and pointed out the necessity for the development of a rational method of design ". . .

which would evaluate the strengths of bituminous mixtures on a pounds per square inch basis . . ." (38). McLeod also developed equations which indicated that the ". . . amount of lateral support . . . provided by the pavement surrounding the loaded area may be appreciably greater than its unconfined compressive strength" (38). This is a point which has long been debated. McLeod saw no particular difficulty in using his rational design method if the Mohr rupture envelope for the bituminous mixture was a parabola rather than a straight line (39). He showed that ". . . for bituminous mixtures with curved Mohr envelopes, straight line Mohr envelopes drawn through the Mohr diagram, and located by the method of least squares, provide stability values that may be from 10 to 20 percent too high, but may nevertheless be sufficiently accurate for practical pavement design at the present time" (38).

Smith (52) described the theoretical and mathematical considerations underlying the development of the closed-system triaxial stability test. He recommended the ". . . use of the triaxial stability test procedure for design of all mixed-type asphaltic surfaces . . " His paper included ". . . suitable stability criteria for all types of mixes and various traffic conditions. . ."

Shearer (49) proposed a method for measuring the lateral pressures in granular mixtures. He pointed out that "In testing a representative laboratory specimen, it appears selfevident that a realistic lateral restraint must be applied in order that these numerical values may be truly significant." He employed a pressure cell for the measurements, but did not have sufficient data to conclude anything relative to the magnitude of confinement in a thin layer.

Several studies of the stability of bituminous mixtures have been reported by Goetz and Chen (15), by Goetz (16), and by McLaughlin and Goetz (33). In one study the vacuum triaxial technique was applied to the testing of bituminous-aggregate mixtures (15). In another, Goetz compared triaxial and Marshall test results and concluded that for the mixtures and testing methods used in the study, ". . . it appears that the Marshall test will provide the same general qualitative evaluation of an asphalt-aggregate mixture as the triaxial test" (16).

McLaughlin and Goetz compared unconfined and Marshall test results and concluded that the Marshall test is actually a type of confined test (33). In summing up the studies of comparisons of Marshall, triaxial, and unconfined test results these authors stated:

> ". . . (the) correlation work . . done in comparing triaxial, unconfined and Marshall test results shows that the test properties chosen from the Marshall test for design purposes are ones that tend to control the fundamental properties of the mixture . . . (and) the fact that our data show the Marshall test to reflect the same properties as the more fundamental strength tests leads us to have confidence in its use for establishing and controlling the asphalt content of bituminous mixtures within the range of those for which correlating performance data have been obtained."

The problem of molding realistic laboratory specimens has also received attention. Vallerga (56) reported the development of a kneading compactor and offered the general conclusion ". . . that the method of compacting or fabricating laboratory test specimens of bituminous paving mixtures has a profound influence on stability and cohesion (tensile strength) values as measured by the Hveem stabilometer and cohesiometer respectively."

Further studies with the kneading compactor were reported by Endersby and Vallerga (14) and by Hveem and Vallerga (22). It was concluded that if different types of stability tests are to be compared, one should use the same method of compaction for preparing all test specimens. In addition, Endersby and Vallerga (14) favored the kneading compactor as being the one which would provide ". . . better agreement in the results of . . . various test methods."

Philippi (43) also considered the problem of laboratory compaction of bituminous mixtures. He used a gyratory shearing action to compact laboratory specimens at low initial pressures which would permit orientation of the aggregate particles. This was followed by a direct compression of the specimens at a unit pressure of 1590 psi. This method was reported to produce laboratory specimens of bituminous concrete which had ". . . a density approximately equal to that obtained in a pavement produced from the mixture and with approximately the same aggregate degradation."

The literature concerning the mechanical stability of bituminous mixtures was summarized in 1953 by Nepre (42). He included a classification of the different types of mechanical tests in common usage and emphasized ". . . the highly empirical and controversial state of the subject . . . " at that time and urged a more rational approach. An extensive bibliography was also included in his report.

A similar general survey of the fundamentals of stability testing of asphalt mixes was made by Stevens (54). He reviewed the requirements for suitable asphalt mixes and then commented on how these required properties were evaluated by many of the test methods in common use.

Finally, and most recently, Wood (67) reported on the stress-deformation characteristics of asphaltic mixtures under various conditions of loading. Using the unconfined compression test, he developed a relationship between maximum unconfined compressive strength and the variables of temperature and rate of deformation for sand-asphalt mixtures. This relationship was further established for confined compression tests using confining pressures of 15 and 30 psi. Wood also investigated the effects of repeated loads on bituminous mixtures and found that for the mixtures tested there was a stress that could be cycled a number of times without causing failure. This stress was called the "endurance limit."

Wood concluded " . . . a promising means of evaluating the adequacy of a bituminous mixture before its utilization in the field would be to perform the confined, repeated load test on a rational specimen at a temperature of 140° F and a rate of strain of 0.005 in./in./min."

PURPOSE

In broad terms, the purpose of this investigation was to add to the present knowledge of the load-carrying characteristics of bituminous concrete particularly when used as an overlay material over portland cement concrete. Several sub-purposes may be listed which contribute toward the whole:

1. A relationship among the factors of load, rate of deformation, and temperature was proposed by Wood (67) as a result of some laboratory tests on sheet-asphalt mixtures. It was a purpose of this investigation to discover whether this form of relationship exists for bituminous concrete; and further, to find how the thickness-of-lay affects any strength-rate-temperature realtionship that might be developed for bituminous concrete. In addition the variable of method of compaction on any derived relationship was to be investigated.

2. The classical approach of evaluating the strength of a bituminous concrete by means of triaxial tests has been questioned by many investigators for two reasons. First, the triaxial test employs a specimen whose dimensions are much different from the dimensions of the mixture in its service application. Second, the theory of failure usually applied to the results of a series of triaxial tests says that the shear strength of a bituminous mixture is a function of the minor principal stress or the "amount of confinement" that is present. While one can, in the laboratory, develop a curve relating shear strength to total applied pressure or total principal stress to minor principal stress, there is no information derived from experimental results to indicate of what order of magnitude this minor principal stress or

"degree of confinement" might be in an overlay. It was a purpose of this investigation to estimate this confinement by means of laboratory tests.

3. The results of field studies have shown that the difficulty encountered with the use of the particular type of bituminous concrete commonly used in Indiana is not failure of the material in shear but the accumulation of permanent deformation in the areas of the pavement subjected to repeated applications of a transient load. One might question then the use of a strength test as the sole criterion for stability design when it is indicated that failure may occur by repeated applications of loads less than the ultimate strength of the material as measured by some conventional strength test. It was a purpose of this investigation to study the behavior of bituminous concrete under repeated applications of load at various temperatures and to compare these results with those obtained from other laboratory strength tests.

SCOPE OF THE PRESENT INVESTIGATION

In order to accomplish the purposes listed in the proceeding section, the study was first subdivided into two parts. In the first part, all tests were performed on specimens molded in the laboratory and in the second part, pavement cores for laboratory tests were obtained from a section of pavement which had been resurfaced with bituminous concrete in the summer of 1950. The construction of this resurfacing and the variables incorporated in the project are described in a succeeding section under the major heading DESCRIPTION OF TESTS PERFORMED ON PAVEMENT CORES. This section describes briefly each phase of the study and the extent to which the laboratory tests were carried.

Tests on Laboratory-Compacted Specimens

An aggregate source and grading were chosen which met the specifications of the State Highway Department of Indiana (53) for Hot Asphaltic Surface, Type B (Medium - Texture). This aggregate was mixed with 60-70 penetration asphalt cement for all laboratory-compacted specimens.

Initially, the aggregate blend was characterized by performing a series of Marshall, ASTM Direct Compression, and triaxial tests over a range of asphalt contents. Also included were a few Hveem Stabilometer and cohesiometer tests on the blend using a single (6.0 percent) asphalt content[1] compacted at two different densities.

After the aggregate blend had been characterized, a single asphalt content of 6.0 percent and mixture densities of 140 and 146 pounds per cubic foot were selected for further study. The results of the Marshall series, the triaxial test results, density results from numerous pavement cores taken in the past, and current Indiana practice indicated that the indicated asphalt content and density values are realistic ones for the mixture employed. Some test results from pavement samples (36) have indicated that a density of 140 pounds per cubic foot may be comparable to the density achieved during construction and that an increase in this initial density to 146 pounds per cubic foot may be obtained after one or two years of service.

Additional triaxial tests were performed on mixtures containing 6.0 percent asphalt and compacted to densities of 140 and 146 pounds

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[1] As used in this thesis, percent asphalt means percent by weight of the mixture.

per cubic foot. These tests were performed at a temperature of 80° F and a rate of deformation of 0.02 inches per minutes with confining pressures up to 120 pounds per cubic foot.

In order to investigate the relationship among strength, rate of deformation, and temperature on laboratory compacted specimens, specimens two inches thick and four inches in diameter were made at an asphalt content of 6.0 percent and compacted to a density of 146 pounds per cubic foot by two methods of compaction. The first method was simply double-plunger compaction with rodding, but the second was a vibrational compaction method developed in this study. This compaction method is described fully in a later section. For the tests being described, the mixture was vibrated into an eleven-inch square by two-inch thick slab and from this slab, cores four inches in diameter were cut with an abrasive-type core drill.

Both types of specimens, the double-plunger compacted specimens and the vibrationally-compacted cores were tested to failure at two rates of deformation (0.2 and 0.02 inches per minute), and at three different temperatures (40, 90, and 140° F), making a simple 3 by 2 factorial experiment for each compaction type.

Finally, the effect on strength of loaded area to specimen size, the results of which were to be used to estimate the degree of confinement in a thin overlay, was investigated by making two-inch thick specimens by vibrational compaction at 6.0 percent asphalt to two different densities (140 and 146 pounds per cubic foot). The specimens were made in four different sizes: four inches in diameter (cores from slab specimens), eight-inch square, eleven-inch square, and sixteen-inch square slabs. These specimens were loaded to failure at a rate of deformation of 0.02 inches per minute at approximately 80° F. The loaded area, in each case, was 12.57 square inches or a piston four inches in diameter.

Tests on Pavement Cores

The pavement cores tested were all four inches in diameter but of three different heights: two inches, three inches and four inches. A series of tests was performed on these cores to determine the relationship among strength, rate of deformation, and temperature, very much like the tests performed on the laboratory-compacted specimens described in the preceeding section. In this case, however, specimen height was a variable and compaction was not. In addition, a third rate of deformation, 0.002 inches per minute, was included making a 3 by 3 factorial experiment for each of the three specimen thicknesses.

Pavement cores were also subjected to repeated load tests of two types. In the first type, loads representing 75, 50, and 25 percent of the ultimate strength of the core at a temperature of 80° F and a rate of deformation of 0.02 inches per minute were repeatedly imposed upon specimens at this temperature and using this rate of deformation. The amount of cumulative permanent deformation was measured.

The second series of repeated load tests employed a different concept of loading than did the first. For the latter series, the total load was applied instantaneously (or almost so) for a period of 0.3 seconds and then released. The time interval between loadings was four seconds and tests were performed at applied stresses of 100, 150, and 200 pounds per square inch each at a temperature of 80 and 140° F for all three core thicknesses. As in the case of the first type of repeated load test, a record of the cumulative permanent deformation was kept.

DESCRIPTION OF TESTS PERFORMED ON LABORATORY-COMPACTED SPECIMENS

In this section, all details pertinent to tests performed on laboratory-compacted specimens are covered including materials, method of specimen preparation, apparatus, and test procedures. After a description of the materials and the method and apparatus used for vibrational compaction, which are common to most of the tests in this series, each test is individually described.

Materials

The bituminous concrete mixture chosen for this portion of the study was one meeting the specifications of the State Highway Department of Indiana for Hot Asphaltic Concrete Surface - Type B (Medium Texture). This mixture has been widely used in Indiana for the purpose of resurfacing concrete pavements. The composition limits for Type B as listed in the 1952 edition of the Standard Specifications of the State Highway Commission of Indiana are shown in Table 1 together with the composition of the mixture as selected for this investigation. Figure 6 shows a graph of the aggregate grading listed in the right-hand column of Table 1.

Table 1

Specification Limits For Hot Asphaltic Concrete Surface - Type B
and
Mixture Composition Used For Laboratory Specimens

Passing Sieve	Retained On Sieve	Composition Limits For Type B Percent		Composition Selected Percent
		Minimum	Maximum	
1/2 inch	3/8 inch	2	14	12
3/8 inch	No. 4	20	50	36
No. 4	No. 6	0	11	5
No. 6	No. 8	0	11	5
No. 8	No. 16	5	20	11
No. 16	No. 50	10	25	23
No. 50	No. 100	2	17	3
No. 100	No. 200	1	5	2
No. 200	-	3	5	3
Total Ret. on No. 6		45	55	53
Bitumen		6.5	8.5	4-7

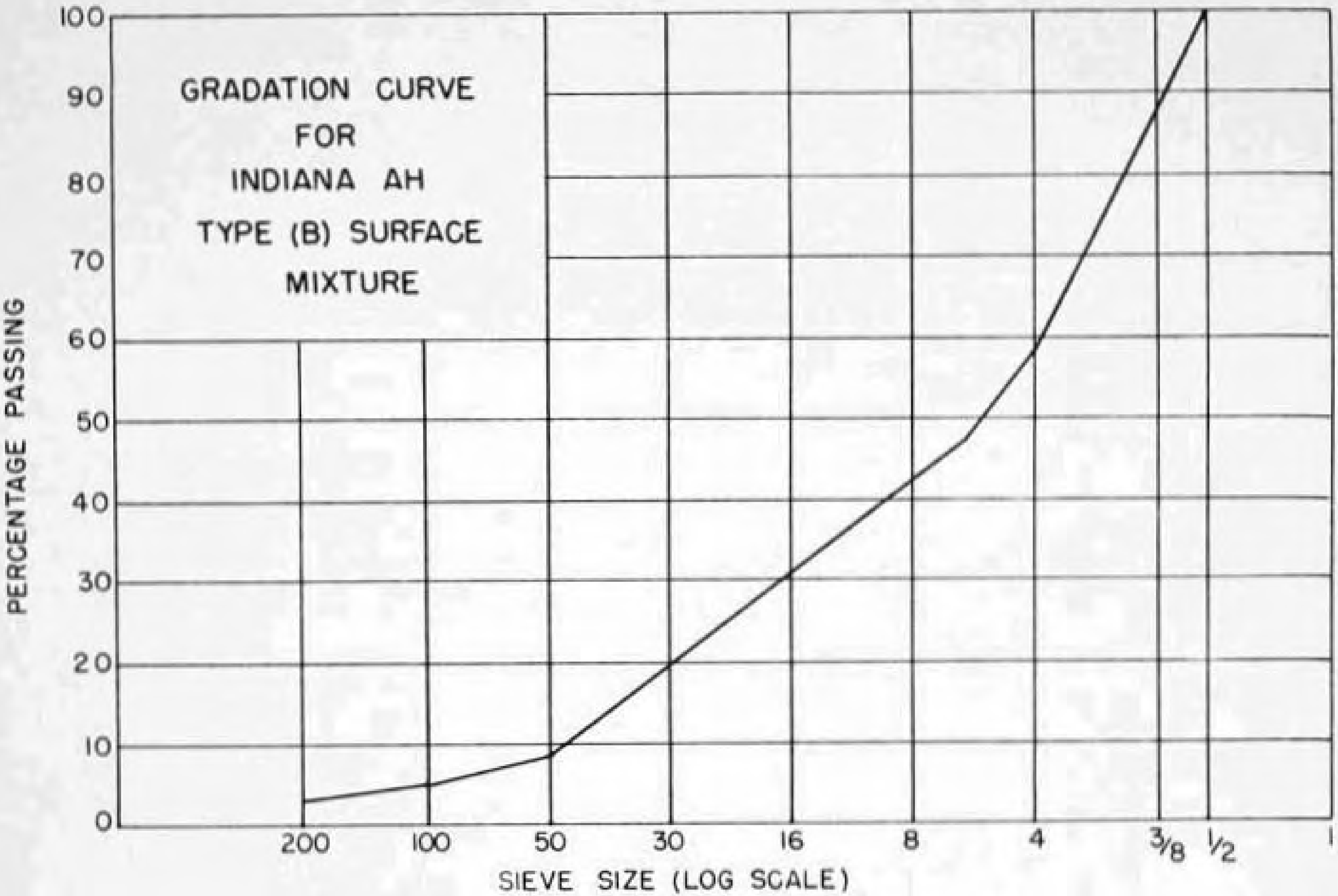

GRADATION CURVE
FOR
INDIANA AH
TYPE (B) SURFACE
MIXTURE
PERCENTAGE PASSING
100
90
80
70
60
50
40
30
20
10
0
200
100
50
30
16
8
4
3/8
1/2
1
SIEVE SIZE (LOG SCALE)

FIG. 6

Aggregates

The coarse aggregate (material retained on the No. 4 sieve) used in this portion of the study was a crushed limestone from central Indiana. The source (laboratory No. 67-25) is located in the St. Genevive formation and is of Mississippian age. The material is fine grained and has a low porosity, its water absorption being of the order of 0.7 percent. When tested for specific gravity by ASTM method C 127, the following results were obtained:

Bulk specific gravity = 2.68

Apparent specific gravity = 2.73

The fine aggregate (material passing the No. 4 sieve), with the exception of the material passing the No. 200 sieve, was a local sand obtained from a river terrace deposit (laboratory No. 79-1). When tested for specific gravity by ASTM method C 128, the following results were obtained:

Bulk specific gravity = 2.61

Apparent specific gravity = 2.71

Portland cement having a specific gravity of 3.15 and a fineness of 1650 square centimeters per gram (Wagner Turbidimeter) was used for that aggregate fraction passing the No. 200 sieve.

The coarse and fine aggregates were air dried and sieved into their respective fractions as listed in Table 1. They were then recombined into batches of the proper size for each specimen to be made. An additional sieve (No. 30) was included for control of the fine aggregate fraction and the 23 percent of the aggregate between the No. 16 and the No. 50 sieve was split 11 percent between the No. 16 and No. 30 and

12 percent between the No. 30 and No. 50 sieves.

Asphalt

A 60-70 penetration grade asphalt cement furnished by the Texas Company from their Port Neches refinery was used for all the laboratory-compacted specimens. Standard test results on this asphalt cement are given in Table 2.

Table 2

Results of Tests on Asphalt Cement

Test	Result
Specific Gravity 77/77° F	1.014
Softening Point, Ring and Ball, °F	125
Ductility, cm	150 +
Penetration, 100 grams, 5 sec., 77° F	67
Penetration, 100 grams, 5 sec., 32° F	19
Flash Point, Cleveland Open Cup, °F	590
Solubility in CCl_4, percent	99.8
Spot Test	Neg.

Vibrational Compaction

For this study, a method of forming slab-type specimens using a pneumatic vibrator was devised. These specimens, which were generally made two inches thick, were made in square shapes either 3 by 3 inches, 11 by 11 inches or 16 by 16 inches. For forming the two smaller size specimens a collapsible steel mold was used. The 16 by 16 inch specimens were compacted in wooden forms. The vibrator that was used was a Cleveland Type FAC, having a 1 1/4 inch piston. This vibrator was operated at a line pressure of 60 pounds per square inch and at this pressure the vibration frequency was 3170 cycles per minute and the air consumption was 3.4 cubic feet per minute. A foot plate or base plate 3 inches by 6 inches by 1/4-inch in thickness was attached to the vibrator to serve as a compacting foot. The vibrator and the steel mold, set-up for an 11-inch square specimen, are shown in Figure 7.

To form a specimen by this method of compaction, the mold was set-up and oiled lightly. The foot-plate of the vibrator was also oiled with a somewhat more liberal application. The aggregate, heated to a temperature of 310 ± 10° F, and asphalt, heated to a temperature of 275 ± 5° F, were brought together in the proper proportions and mixed for three minutes after which an amount of mixture sufficient to form a two-inch thick specimen of the size being made was placed and spread evenly in the steel mold. Care was exercised to spread this material in a uniform layer because it was obvious that a uniformly thick and uniformly dense specimen would result only if proper precautions were exercised at each step in the procedure. This particular step was deemed especially important because there was not much opportunity for

FIG. 7 VIBRATOR AND MOLD FOR SLAB SPECIMENS

redistribution of material in the mold in a sideways direction through the action of the vibrator. The vibrator was then started and the foot plate was moved over the mixture as it compacted due to the vibrating action. When the height of the mixture in the mold was reduced to two-inches, as indicated by marks inscribed on the sides of the mold, the vibrator was removed and a 1/2-inch thick steel plate of the proper size (either 8 by 8-inches square or 11 by 11-inches square) was placed on top of the specimen. The vibrator was then started again and held firmly on top of the steel plate. This action removed any waves or bumps in the surface of the specimen and left a smooth surface for testing. When making the 16-inch square specimens, the 11-inch square steel plate was used for this last operation. After final compaction, the specimen was allowed to cool and then removed from the mold.

This method of compaction proved to be successful and no difficulty was encountered in obtaining the density values that were chosen for this study i.e., 140 and 146 pounds per cubic foot. It is felt that at the air pressure employed, density values much higher than 146 pounds per cubic foot might be somewhat difficult to obtain; however, the vibrator that was used is said to operate at maximum efficiency at 80 psi continuous line pressure and pressures up to 120 psi may be used. Perhaps at these higher air pressures, more dense specimens could be produced if they were wanted.

Characterization Tests

Several series of tests were performed on the bituminous concrete in order to characterize or describe the mixture in terms of conventional laboratory tests. The tests that were performed include the Marshall test, ASTM Direct Compression test, triaxial test, and the Hveem Stabilometer and Cohesiometer tests. The procedures for forming the specimens and for making these tests are described in this section.

Marshall

The detailed procedures for specimen formation and testing by the Marshall method are quite common and appear in many places in the literature (4, 10, 11). The Marshall tests performed in this investigation were of the standard variety so only a brief treatment will be given.

To form the specimens, a batch (2500 grams) of aggregate was heated in a large gas fired oven, described in reference 34, to a temperature of 310 $\pm$ 10° F. A quantity of asphalt sufficient to bring the batch to the desired asphalt content was simultaneously heated to 275 $\pm$ 5° F. The Marshall molds, spoons, spatulas, a brass mixing bowl and the Marshall compacting foot were also heated prior to the mixing. When the components were at the required temperatures, the asphalt was weighed into the aggregate in the mixing bowl and the two were mixed for two minutes in a modified Hobart mixer (see reference 34). The batch was then divided in two, half going in each of two molds. The specimens were then compacted with the Marshall hammer, fifty blows on each face, cooled and removed from the molds. Two Marshall specimens were made at 4, 5, 6, and 7 percent asphalt.

To test the specimens, they were first immersed for a minimum of 25 minutes in a 140° F water bath and then removed and tested to failure in the standard Marshall breaking head at a rate of deformation of two inches per minute. The maximum time that was permitted from removing the specimen from the water bath to the end of the compression test was thirty seconds.

Data collected in this test series were unit weight of specimens, Marshall stability and Marshall flow.

ASTM Direct Compression

The ASTM compression tests were performed according to ASTM Designation D 1074-55 with some changes in the procedure dictated because of available laboratory equipment. These changes, however, are considered to be minor in nature and the test results may be viewed as standard.

The aggregate and asphalt for these specimens were heated to 310 $\pm$ 10° F and 275 $\pm$ 5° F respectively, combined, and mixed for two minutes in the modified Hobart mixer. The specimens were formed in a 4-inch diameter steel mold (described in reference 34) by double-plunger compaction at a contact pressure of 3000 psi. A hydraulic jack fixed in a steel frame supplied the compoctive effort. The specimens were made to a four-inch height, a trial mix being used in each case to determine the quantity of material needed for a specimen of this height. Specimens were made at 4, 5, 6 and 7 percent asphalt.

The specimens were tested "in axial compression without lateral support at a uniform rate of vertical deformation of 0.05 inches per minute per inch of height (0.2 inches per minute for specimens 4 inches in height)" as specified (1). The tests were made in a Riehle testing machine of 50,000 pounds capacity to which had been added a Graham variable speed drive which extended the range of testing speeds possible. The specimens were tested at room temperature which was 77 $\pm$ 5° F during the period in which these tests were made.

Data collected in this test series were unit weight and compressive strength of the specimens.

Triaxial

The specimens for the triaxial tests in the characterization series were made at four asphalt contents, 4, 5, 6 and 7 percent and at two different compactive efforts. The materials were heated and mixed as previously described for Marshall and ASTM compression tests, placed into the 4-inch diameter steel mold in four layers each of which was rodded 25 times with a 7/8-inch diameter steel rod, and compacted by double-plunger action to a contact pressure of either 2000 or 2500 psi. No attempt was made to control density to any given value, the density produced by such compaction being one of the things that it was desired to learn from this test series. However, the amount of material placed in the mold was controlled to produce specimens approximately ten-inches high.

For each asphalt content and compactive effort, four specimens were made. Two of these were tested in triaxial compression at a confining pressure of 15 psi and the other two were tested at 30 psi. A total of 32 triaxial specimens were made and tested for the characterization tests.

The triaxial tests were performed at a rate of deformation of 0.02 inches per minute at room temperature ($80 \pm 5^\circ$ F) in the Riehle testing machine. The triaxial cell used for these tests is shown in Figure 8, the key to which appears on the page following the Figure. The data collected in this test series were unit weight and compressive strength of the mixtures at confining pressures of 15 and 30 psi. From these data, the cohesion and angle of internal friction of the mixture at the various asphalt contents could be derived.

DETAILS OF TRIAXIAL CELL

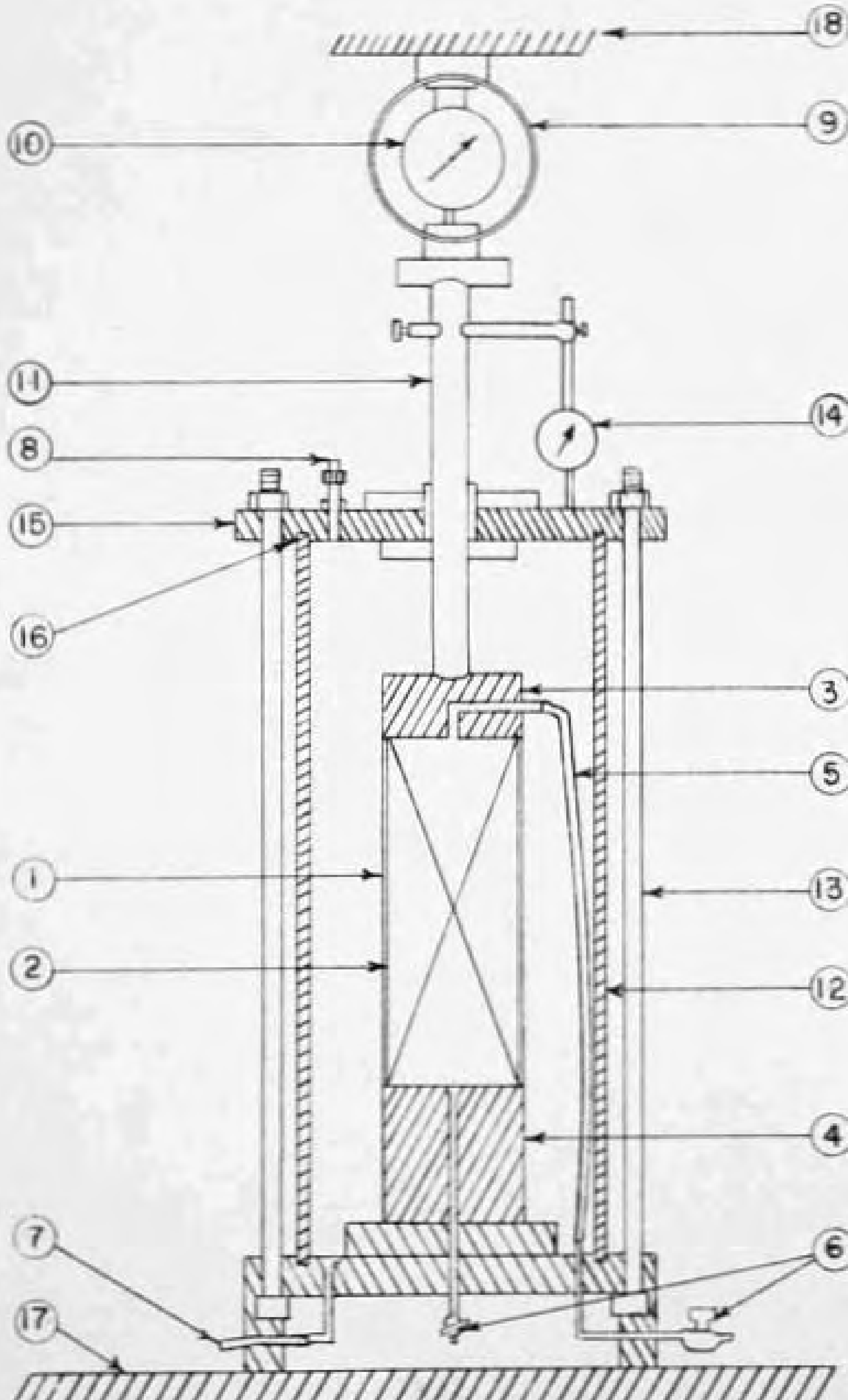


FIG. 8

Key to Figure 8

1. Test specimen
2. Rubber membrane
3. Upper testing-head
4. Lower testing-head
5. Upper drainage line
6. Lower drainage line and drainage control valves
7. Compressed-air inlet line
8. Compressed-air relief valve
9. Proving ring
10. Proving ring deflection dial
11. Loading piston
12. Wall of triaxial cell
13. Connecting bolts
14. Strain dial
15. Cover plate
16. Recess in cover for cylinder
17. Table of testing machine
18. Loading-head of testing machine

Hveem

A Hveem Stabilometer and Cohesiometer were available for a short time during the summer of 1956 and a few specimens were made and tested in this apparatus. The testing procedure is described completely in reference 4. Specimens for these tests were 2 1/2 inches high and four inches in diameter. They were made by double-plunger compaction to controlled density values of 140 and 146 pounds per cubic foot at an asphalt content of 6.0 percent

Information derived from this test series was the Stabilometer value, S, and the Cohesiometer value, C, for the mixture at an asphalt content of 6.0 percent and at 140 and 146 pounds per cubic foot.

Triaxial Tests - Controlled Density Specimens

After characterizing the mixture by the tests described in the previous sections, a decision was made relative to the asphalt content and specimen density to be used for all succeeding laboratory specimens. The results of the Marshall series, the triaxial test results, density results from numerous pavement cores taken in the past, and current Indiana practice led to a decision to make all further specimens at an asphalt content of 6.0 percent and to use two levels of density, 140 and 146 pounds per cubic foot.

Twelve specimens four-inches in diameter and 9 1/2-inches high were made at an asphalt content of 6.0 percent by the general procedures outlined in the section on triaxial tests. In this case, however, the quantity of material going into the mold was controlled so that when the specimen was compacted to a height of 9 1/2-inches, the predetermined density was achieved. By these methods, six specimens were made to a density of 140 pounds per cubic foot and six to a density of 146 pounds per cubic foot.

The specimens were tested in triaxial compression at 15, 30, and 120 psi confining pressure. The highest confining pressure was realized through the use of a tank of compressed nitrogen. The rate of deformation used was 0.02 inches per minute and the specimens were tested at room temperature which was in the range 77 $\pm$ 5 $^{\circ}$ F during the test period.

Data were derived from these tests that were used to compute the cohesion and angle of internal friction of the two mixtures tested.

Variable Specimen Area Test Series

An experiment designed to determine the influence of the total area of the specimen relative to the area being loaded necessitated the fabrication of specimens all having the same thickness (two-inches) but having various areas. For purposes of making comparisons between test results from different-size specimens, all specimens had to be compacted by the same means. It was primarily for this test series that the vibrational method of compaction was developed.

Using the vibrational compaction method and the equipment shown in Figure 7, specimens were made at an asphalt content of 6.0 percent to densities of 140 and 146 pounds per cubic foot in three different sizes: 8 by 8-inches, 11 by 11-inches, and 16 by 16-inches. An additional size of specimen was obtained by taking several of the 11 by 11-inch slabs and, using an abrasive-type core drill, cutting from the slab four, 4-inch diameter by 2-inch thick cores. Several specimens of each size and density were prepared so that at least two replicates of each strength test could be obtained. The procedure for making these specimens is outlined in the section on vibrational compaction. No difficulty was encountered in controlling the specimen density and height. The procedure developed was thought to be highly successful.

These specimens were all tested in direct compression at 0.02-inches per minute and at room temperature which varied from 75 to 80° F during the period during which the tests were made.

A four-inch diameter by one-inch thick steel disc was used to transmit load to all specimens. For the four-inch diameter cores, the loaded area was, of course, equal to the specimen area and the test was simply a direct compression test on a rather thin specimen. When testing the larger specimens, the four-inch diameter disc was placed on the center of the specimen and these tests may be likened to plate-loading tests or CBR tests in that the specimen was loaded over only a portion of its surface area. However, unlike plate-loading or CBR tests, these slabs of compressed bituminous concrete were not contained in a mold of any sort during the application of load and the tests was carried out to a failure point. With the variety of specimen sizes used, one obtained strength results for specimen area to loaded area ratios of 1, 5.1, 9.7, and 20.4.

All but the 16 by 16-inch specimens were tested in the Riehle testing machine. The bay of this machine was not wide enough to accomodate the 16 by 16-inch specimens so these were tested in a Southwark-Tate-Emery hydraulic-type machine.

From the complete series of tests the effect on strength of specimen area to loaded area ratio could be determined.

Strength-Rate-Temperature Series

A series of tests on laboratory-compacted specimens was designed to test the relationship between the strength of a bituminous concrete and the variables of rate of deformation and temperature. A relationship of the form log log strength = C_1 log rate of deformation + C_2 Temperature, where C_1 and C_2 are constants, has been proposed by Wood (67). His work was primarily concerned with sheet asphalt mixtures and, as has been stated, a purpose of the present investigation was to discover whether this form of relationship exists for bituminous concrete.

In this test series bituminous concrete specimens four-inches in diameter and two-inches thick were made at an asphalt content of six percent to a density of 146 pounds per cubic foot by double plunger static compaction and by vibrational compaction. The vibrated specimens were cores cut from 11 by 11-inch slabs made by methods previously described.

Compression tests were performed at two rates of deformation, 0.2 and 0.02 inches per minute, and at three temperatures, 40, 90, and 140° F. The tests were made in the Riehle testing machine; the temperature of the specimens was controlled by immersing them in a constant temperature water bath for 1/2-hour before and during the test period. For the tests made at 40° F, melting ice was used to obtain and control the temperature. For the tests at 90 and 140° F, temperature control was obtained through the use of a 250 or 500 watt immersion heater, the voltage to which was regulated by a powerstat to maintain the desired temperature level in the bath.

These tests produced a strength value for the bituminous concrete made by each method of compaction for every rate-temperature combination. Duplicate determinations were made at all levels.

Marshall Comparison Series

Certain indications that the deformation characteristics of vibrated specimens were different from those of specimens compacted by double-plunger static compaction led to a series of tests in which Marshall-size (4-inch diameter by 2 1/2-inches high) specimens were formed, at an asphalt content of six percent, by standard Marshall compaction, double-plunger static compaction, and by vibrational compaction. Again, for the vibrated specimens an 11 by 11, but in this case 2 1/2-inch thick, slab was formed and from it cores were cut.

Four specimens of each type were made and all were tested at 140° F in the Marshall Apparatus, stability and flow data being observed.

DESCRIPTION OF TESTS PERFORMED ON PAVEMENT CORES

In this section, all details pertinent to tests on pavement cores are described. Since all pavement cores were obtained in 1955, from one particular section of road on which some other experimentation had been done, a section on the background, construction and sampling of this section of resurfacing is included.

Design and History of Experimental Overlay

The portion of Indiana State Road 37 from approximately the north edge of Noblesville to the junction with Indiana State Road 13 south of Elwood was resurfaced with bituminous concrete binder and surface during the summer of 1950. In order to study the effect of thickness of lay on the various aspects of performance of the resurfacing, experimental sections of various thickness were built into the project. Each of these sections was approximately 1000 feet in length and the thickness designs were as follows:

a) one-inch of binder, one-inch of surface

b) one and one-half-inch of binder, one and one-half-inches of surface

c) three-inches of binder, one-inch of surface

Standard thickness design for the project outside of the experimental area was one and one-half inches of binder with one-inch of surface.

Sections of both jointed and non-jointed concrete are included in this part of S. R. 37 and these experimental areas of various overlay thicknesses were constructed in each section in order to study the effect of type of concrete pavement on the performance of the resurfacing. Thus, six experimental sections, each 1000 feet in length, were constructed. Other experimental variables were included in this project, but these have no bearing on the present discussion.

The binder and surface mixtures used in this project were of the type described in the "Introduction." The surface mixture met the specifications listed in Table 1 and the aggregate grading was similar to that used for the laboratory-compacted specimens for this study. However, the surface mixture used on S. R. 37 contained seven percent asphalt compared to six percent for the surface mixture used in this study. The binder mixture used on S. R. 37 contained about 5.5 percent asphalt.

Crushed stone coarse aggregate from a source near Lapel, Indiana, was used throughout the resurfacing. The 60-70 penetration asphalt cement was supplied by the Ohio Oil Company. The fine aggregate was a natural sand with limestone dust being used for that portion of the fine aggregate passing the No. 200 sieve.

Soon after construction and again in 1952, and 1955, samples, to be used to determine density, were taken from each of the six areas being observed. The samples in any one section were taken both in a wheel track and in the area between wheel tracks. Much interesting information concerning the densification of a bituminous concrete overlay was derived from these field test sections. As the results relate to the problem with which this study is concerned, included in Appendix B is an excerpt of a report written by the author and W. H. Goetz to the Advisory Board of the Joint Highway Research Project, Purdue University, dated September 21, 1955 and entitled, "Results of Density

Measurements on Bituminous Concrete Experimental Section - S. R. 37." It should be stated here, however, that in 1955, the time at which cores were taken from this pavement for the laboratory tests herein reported, density test results from all comparable sections of this road were remarkably uniform indicating homogeneity of the mixture in similar sections and all across the pavement. It appeared that, as of the time of sampling for the tests reported, the location of the core within a given section was not a variable that needed consideration.

Strength-Rate-Temperature Series

An investigation, similar to the one made on laboratory-compacted specimens, was made on pavement cores for the purpose of determining the form of the relationship between compressive strength and the variables of rate of deformation and temperature. A separate test series was made for each of the thicknesses of combined binder and surface that was used, i.e.; (a) four-inches, composed of three-inches of binder and one-inch of surface, (b) three-inches, composed of one and one-half-inches of binder and one and one-half-inches of surface, and (c) two-inches, composed of one-inch of binder and one-inch of surface. Three rates of deformation, 0.2, 0.02, and 0.002-inches per minute and three temperatures, 40, 90, and 140° F were included for each thickness.

The cores when brought into the laboratory had rough, uneven bottoms which necessitated their being capped. Plaster of Paris was used for this purpose. The tests were made on the Riehle testing machine modified with the Graham variable-speed transmission. Temperature control was obtained in the same way as was done in the case of the corresponding tests on the laboratory compacted samples; melting ice bath for the tests at 40° F and a water bath and immersion heaters for the tests at 90° and 140° F.

These tests produced a strength value for each size of core and for each rate-temperature combination. Duplicate determinations were made at all levels.

Repeated Load Tests

Pavement cores were subjected to two types of repeated load tests in the laboratory. The cores tested were taken from S. R. 37 by the means already described, brought into the laboratory and capped on the bottom side with plaster of Paris. In the following sections, each type of repeated load test is described and the testing program for each type of test is outlined.

Slow Cycle

The slow-cycle, repeated load tests were patterned after a method devised by Wood (67) in which the specimen at some temperature was loaded at a controlled rate of deformation to a stress that was some percentage, less than 100, of the ultimate strength for the mixture at the given temperature and rate of deformation. As soon as this stress level was reached, the load was removed and the specimen allowed to rebound under no load. After essentially complete rebound, the cycle was repeated. A record of the deformations experienced by the specimen during the various phases of the cycle was made by observing an Ames dial which measured movement of the upper surface of the specimen with respect to a fixed point. By this means information on the total deformation per cycle, the amount of rebound, and the residual or permanent deformation per cycle were obtained. The summation of all preceeding residual deformation values measured the total permanent deformation at any cycle and was the test criterion of interest in this study.

This slow-cycle test was performed in the Riehle testing machine. The specimen to be tested was supported on a hydraulic jack, the piston of which was extended to some degree. Load was applied to the specimen and when the desired stress level was reached, pressure in the oil system of the hydraulic jack was released by opening a valve. This caused almost instantaneous release of the load on the specimen.

Eight pavement cores were tested in this apparatus before it was decided to concentrate efforts on the rapid-cycle repeated load test described in the next section. The slow-cycle tests were all performed at 80° F and, from the results of the tests in the strength-rate-temperature series for cores, the ultimate strengths of the two, three-, and four-inch cores at 80° F and a deformation rate of 0.02-inches per minute were calculated. In the slow-cycle, repeated load series, it was originally intended to test each size of core at 25, 50 and 75 percent of its ultimate strength, but the full test series was not completed. The tests performed in this series were as follows:

Four-inch cores - two at 75 percent and two at 50 percent of ultimate strength.

Three-inch cores - one each at 75, 50 and 25 percent of ultimate strength.

Two-inch cores - one at 25 percent of ultimate strength.

The data on cumulative permanent deformation were plotted versus the number of load repetitions or cycles, a plot on log-log paper appearing to be the most useful.

Rapid Cycle

Apparatus was devised by Havers (19) for a study on soils which would permit the rapid application and release of load. It was thought that this apparatus could be adapted for use in the present study to give a cycle that might be comparable in intensity and time of load application to a service condition on an open highway. Basically, this apparatus consisted of an air motor mounted in a loading frame. Suitable valves and controls were present to admit compressed air to the air motor for a time interval that could be pre-set. The load was transmitted from the air motor to a shaft which transmitted load to a plate which rested on the specimen to be tested. At the end of the pre-set time interval, during which the load was maintained constant, the inlet air line to the motor closed and an exhaust valve opened which released the air and hence quickly removed the load from the specimen. After another time interval, which could also be pre-set, the cycle was repeated.

The above describes the basic operation of the equipment. Havers (19) described the components of the equipment as follows (parenthetcal expressions inserted by this writer):

> "Compressed air at 100 to 110 psi outlet pressure was supplied by a Gardner-Denver model 329 LT air compressor, powered by a 7 1/2 hp. electric motor. From the compressor tank a one-inch diameter galvanized pipe led through an air line filter to remove moisture and any foreign particles from the air, and thence into a Grove

GB 307-06 Powreactor Dome pressure regulator. This regulator, of the diaphragm type was found to regulate the outlet pressure quite accurately while still permitting the sudden and intermittent flow of air required to produce the repetitive type of loading.

"From the pressure regulator the line passed through an air line lubricator and branched to enter two Bellows 4SSV 3/4 Bel-Air electrically-controlled valves, each of which was connected to one end of a Bellows 8002 FR Power Dome air cylinder. This cylinder was mounted vertically to the upper platen of the loading frame and afforded a downward stroke of from zero to two-inches for the loading piston. A loading head attached to the loading piston applied load directly to the samples . . . placed on the lower platen beneath the air cylinder. (For the present study, a ball attachment was screwed on to the bottom of the loading piston and a four-inch diameter, one-inch thick stainless steel disc lay on top of the specimen. This disc had a socket machined in the center of the top surface to receive the ball). The total load which was applied could be controlled by adjusting the Powreactor pressure regulator, while both the duration of load and time interval between successive load applications could be regulated through the Bel-Air valves by means of an Agastat DED-11 double-acting timer . . . Manual valves and switches were so arranged that one Bel-Air valve could be removed from the system if desired, resulting in a single-acting stroke of the loading piston rather than its normal reciprocating action. (The single-acting stroke was the one used in the present study.)

"The Grove regulator could be adjusted to approximately the desired pressure for any test series by observing a dial (pressure) gauge connected directly to the pressure dome of the regulator . . . ."

To measure the deflections that were of interest in the present study, movement of the loading piston was measured through the use of a Schaevitz Model .04-C-.04 Linear Variable Differential Transformer (L.V.D.T.), the voltage output from which was passed through a Brush model BL-320 Universal Analyzer and recorded directly as a deflection-time curve on a Brush model BL-292 oscillograph. The core of the transformer was mounted on the loading piston while the coil was mounted on an arm fixed to the loading frame. As the cumulative permanent deflection in any given test might be expected to exceed 0.04-inches, the range of the L. V. D. T. in one direction, the core of the transformer was actually mounted on the end of a brass rod the other end of which rested on the end of a micrometer screw. This micrometer screw in turn was attached to the loading piston so that actually any vertical movement of the loading piston resulted in the same vertical movement of the core of the transformer. This arrangement enabled one to extend the useful range of the transformer because when the permanent deflections had accumulated to the point where the core was near its extreme limit, the micrometer-screw could be turned so that the core was moved back up into the coil. Readings on the micrometer screw told how much the core had to be reset at any given time and this factor was simply added to all further deflections measured on the oscillograph.

The apparatus used for the rapid-cycle repeated load tests is shown in Figure 9. For clarity, the apparatus is shown without the water bath which was used to control the temperature of the specimen. The numbers on the Figure are explained in the key on the page following the figure.

All three sizes of cores were tested at 80° and 140° F and at contact pressures of 100, 150 and 200 psi in the rapid-cycle repeated load test. For all tests, the load duration and interval between loads was kept constant at 0.3 and 4.0 seconds respectively. The tests were carried out to failure, a criterion for which is explained in the section on RESULTS, or to 2000 cycles. A few of the tests were continued somewhat beyond 2000 cycles.

The following is the stepwise procedure that was used:

1. The bridge circuit of the analyzer was balanced by removing the transformer output leads from the bridge and inserting leads to

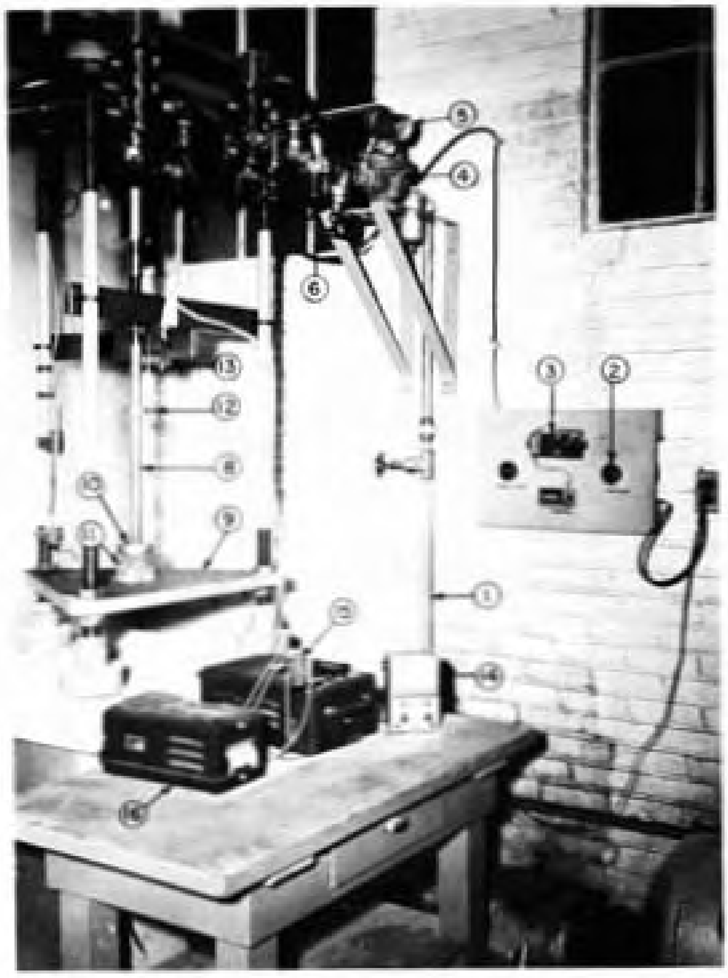


FIG. 9 RAPID-CYCLE REPEATED LOAD TEST APPARATUS

Key to Figure 9

1. Air line from air compressor
2. Main-line switch
3. Agastat
4. Grove GB 307-06 Powreactor Dome pressure regulator
5. Pressure gauge
6. Air line lubricator
7. Bellows 8002 #A Power Dome air cylinder
8. Extension rod for loading head
9. Lower platen of testing machine
10. Testing-head
11. Specimen
12. Assembly for supporting core of L.V.D.T.
13. Assembly for holding coil of L.V.D.T.
14. Voltmeter
15. Brush Model BL-320 Universal Analyzer
16. Brush Model BL-292 Oscillograph

an AC voltmeter in their place. The balance-knobs of the analyzer were adjusted until there was zero voltage on the bridge. This is a rather crucial operation for this type of measurement because if proper balance is not achieved, the oscillograph will not show a linear calibration. It was found that this had to be done only once for all the tests that were performed.

2. The specimen was put in place in the water bath on the lower platen and, before the specimen changed in temperature appreciably from that of the room, one cycle of a 50 psi seating load was applied. The air pressure was then adjusted by manipulation of the regulator to the desired level for the test. The specimen was brought to the test temperature (80 or 140° F) and remained there for a minimum of 30 minutes before the test was begun.

3. Just prior to starting the loading cycles, the transformer core was moved up into the coil, the output from the transformer placed back across the bridge of the analyzer, and the core was moved into its center (null) position by manipulation of the micrometer. This point was reached when the voltmeter, still across the bridge, again showed zero voltage. At this point, the micrometer was read and the voltmeter was disconnected.

4. The knob controlling the AC gain on the analyzer was adjusted so that the pen, when starting from one side of the oscillograph, would travel to the opposite side when the core moved downward a distance of 0.04 inches. A calibration of one division on the oscillograph equal to one thousandth of an inch movement of the core was the most convenient and was sought, not always successfully. As the core was moved

during this calibration, it was returned to null by moving the micrometer to the null reading previously taken.

5. The test was started. This was accomplished by starting the recording oscillograph and closing the main line switch controlling the Agastat. The oscillograph was left running so that a picture of the first forty or fifty cycles was obtained and then it was turned on periodically to monitor the action at intervals during the test. When the permanent deformation of the specimen approached 0.04 inches, the core was brought back toward null by turning the micrometer screw. The new micrometer reading was recorded on the oscillograph paper at the point of change. This process was repeated whenever necessary.

6. After the specimen failed, or 2000 cycles were reached, the test was discontinued. The record on the graph paper from the oscillograph furnished the data for tabulating the cumulative permanent deformation at various numbers of cycles.

RESULTS OF TESTS ON LABORATORY-COMPACTED SPECIMENS

This study was divided into two main sections. In one, laboratory tests of various types were performed on specimens molded in the laboratory. In the second, two types of repeated load tests were performed on pavement cores. In this section, the results of the tests made on laboratory compacted specimens are reported.

The tests on laboratory-compacted specimens were sub-divided into tests used to characterize the mixture used, triaxial tests on specimens made to controlled densities, strength-rate-temperature tests, variable specimen area tests, and tests comparing Marshall test results for specimens compacted by different methods. Both tables and figures are used to present these results in this section.

Characterization Tests

The characterization tests were performed to provide background information concerning the mixture chosen for laboratory testing. The tests were the commonly-employed laboratory strength tests i. e., Marshall, ASTM Direct Compression, triaxial, and Hveem. The results of these tests would be expected to provide one with information on the properties of the mixture in familiar terms.

Marshall

The results of the Marshall tests on the bituminous concrete mixture, with asphalt contents ranging from 4.0 to 7.0 percent, are shown in Table 3. Also included in Table 3 is the percent voids in the mixture and the percent aggregate voids filled with asphalt for the mixture at each asphalt content. These values were calculated using the ASTM apparent specific gravity values for the aggregates as listed in the section on "Materials."

Figure 10 shows the usual curves used in the Corps of Engineers design method using Marshall test results. By this system an asphalt content of 5.6 percent would be selected for use. However, it should be recognized that the aggregate gradation used for this bituminous concrete does not meet the specifications of the Corps of Engineers and strictly speaking, their design method does not apply. The material is included only for background information on the mixture.

Table 3

Marshall Test Results

Specimen Number	Asphalt Content Percent	Unit Wt. Mix. lbs. per. ft.^{3}	Stability Pounds	Flow 0.01 in.	Percent Voids in Mix	Percent Voids Filled
4-1		145.5	1690	6.7		
4-2		145.5	1660	5.7		
Avg.	4.0	145.5	1675	6.2	9.0	50.3
5-1		148.0	1730	7.9		
5-2		148.2	1760	7.6		
Avg.	5.0	148.1	1745	7.8	6.5	63.1
6-1		150.2	1600	11.7		
6-2		150.0	1570	10.8		
Avg.	6.0	150.1	1585	11.2	2.8	83.5
7-1		146.8	1160	13.8		
7-2		147.0	1080	14.3		
Avg.	7.0	146.9	1120	14.0	3.4	83.0

Estimate of Design Asphalt Content by C. of E. Method

Asphalt Content at:

Peak of Stability Curve	= 5.0
Peak of Unit Weight Curve	= 6.0
Four Percent Voids	= 5.7
Eighty Percent Voids Filled	= 5.8
Avg	= 5.6

At 5.6 Percent Asphalt:

Stability	= 1690 lbs.
Flow	= 9.8
Percent Voids	= 4.3
Percent Voids Filled	= 75

MARSHALL TEST RESULTS

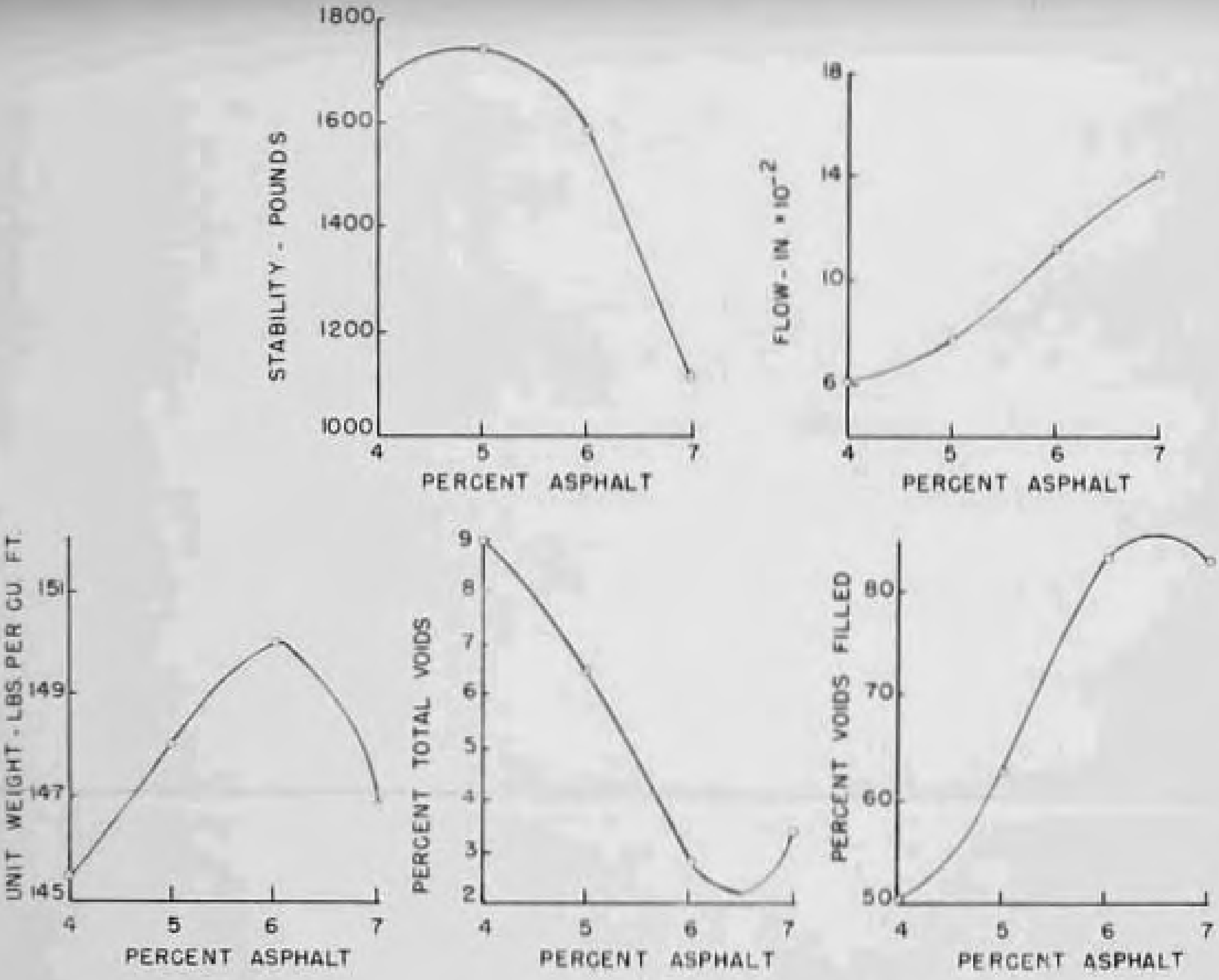


FIG. 10

ASTM Direct Compression

The ASTM direct compression test results are given in tabular form in Table 4. These results are also plotted and shown in Figure 11.

Table 4

Results of ASTM Direct Compression Tests

Spec. Number	Asphalt Content-Percent	Unit Wt. of Mix lbs. per ft.^{3}	Total Load at Failure - lbs.	Compressive Strength psi
1	4.0	141.0	3420	272
2	5.0	143.3	3700	295
3	6.0	144.8	3820	304
4	7.0	145.5	3850	307

ASTM COMPRESSION TEST RESULTS

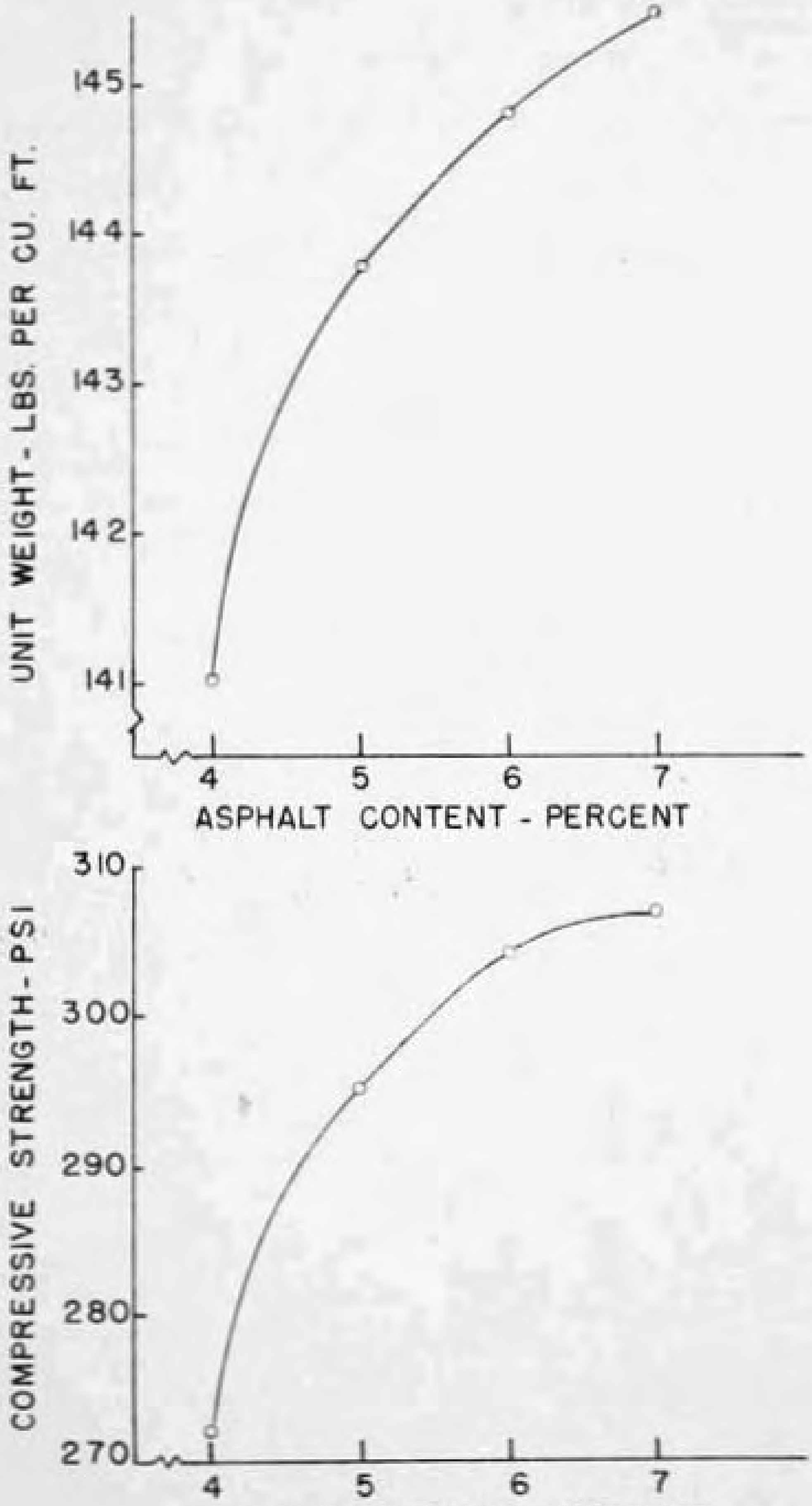


FIG. 11

Triaxial Tests

The results of the triaxial tests are shown in Tables 5 through 7 and in Figure 12. Table 5 contains the density values and total stress at failure for the specimens compacted to a contact pressure of 2000 psi. Table 6 contains the corresponding information for the specimens compacted to a contact pressure of 2500 psi (these will be referred to as "loose" and "dense" specimens).

Mohr rupture envelopes were derived, using the information contained in Tables 5 and 6, for each mix at each asphalt content. The values for cohesion and angle of internal friction so calculated are shown in Table 7. The information found in Table 7 is displayed graphically in Figure 12.

Table 5

Triaxial Test Results
from
Specimens Compacted to a Contact Pressure of [illegible] psi
(Loose)

Spec. Number	Asphalt Content Percent	Unit Wt. of Mix lbs. per $ft.^3$	Confining Pressure psi	Total Stress At Failure psi
1		139.7		117
2		140.1		121
Avg.	4.0	139.9	15	119
3		139.1		193
4		139.9		188
Avg.	4.0	139.5	30	190
5		141.6		126
6		141.7		125
Avg.	5.0	141.6	15	125
7		140.9		199
8		141.1		195
Avg.	5.0	141.0	30	197
9		142.4		129
10		144.1		129
Avg.	6.0	143.2	15	129
11		143.6		204
12		142.9		190
Avg.	6.0	143.2	30	197
13		145.1		115
14		145.6		125
Avg.	7.0	145.3	15	120
15		145.3		180
16		145.1		192
Avg.	7.0	145.2	30	186

Table 6

Triaxial Test Results
From
Specimens Compacted to a Contact Pressure of 2500 psi
(Dense)

Spec. Number	Asphalt Content Percent	Unit Wt. of Mix lbs. per ft.^{3}	Confining Pressure psi	Total Stress At Failure psi
1		142.5		156
2		142.0		156
Avg.	4.0	142.2	15	156
3		141.9		220
4		142.1		228
Avg.	4.0	142.0	30	224
5		144.1		159
6		143.5		165
Avg.	5.0	143.8	15	162
7		144.7		219
8		144.1		223
Avg.	5.0	144.4	30	221
9		146.4		156
10		146.4		149
Avg.	6.0	146.4	15	152
11		145.9		190
12		146.6		206
Avg.	6.0	146.2	30	198
13		146.0		133
14		147.3		124
Avg.	7.0	146.6	15	128
15		148.7		162
16		147.5		176
Avg.	7.0	148.0	30	169

Table 7

Triaxial Test Results-
Cohesion and Angle of Internal Friction

Asphalt Content Percent	"Loose" Specimens Cohesion psi	Angle of Internal Friction Degrees	"Dense" Specimens Cohesion psi	Angle of Internal Friction Degrees
4.0	11.0	41	20.6	40
5.0	12.1	41	26.0	36
6.0	14.3	40	31.3	30
7.0	12.8	39	26.2	28

TRIAXIAL TEST RESULTS - CHARACTERIZATION SERIES
COHESION AND ANGLE OF INTERNAL FRICTION VERSUS
ASPHALT CONTENT

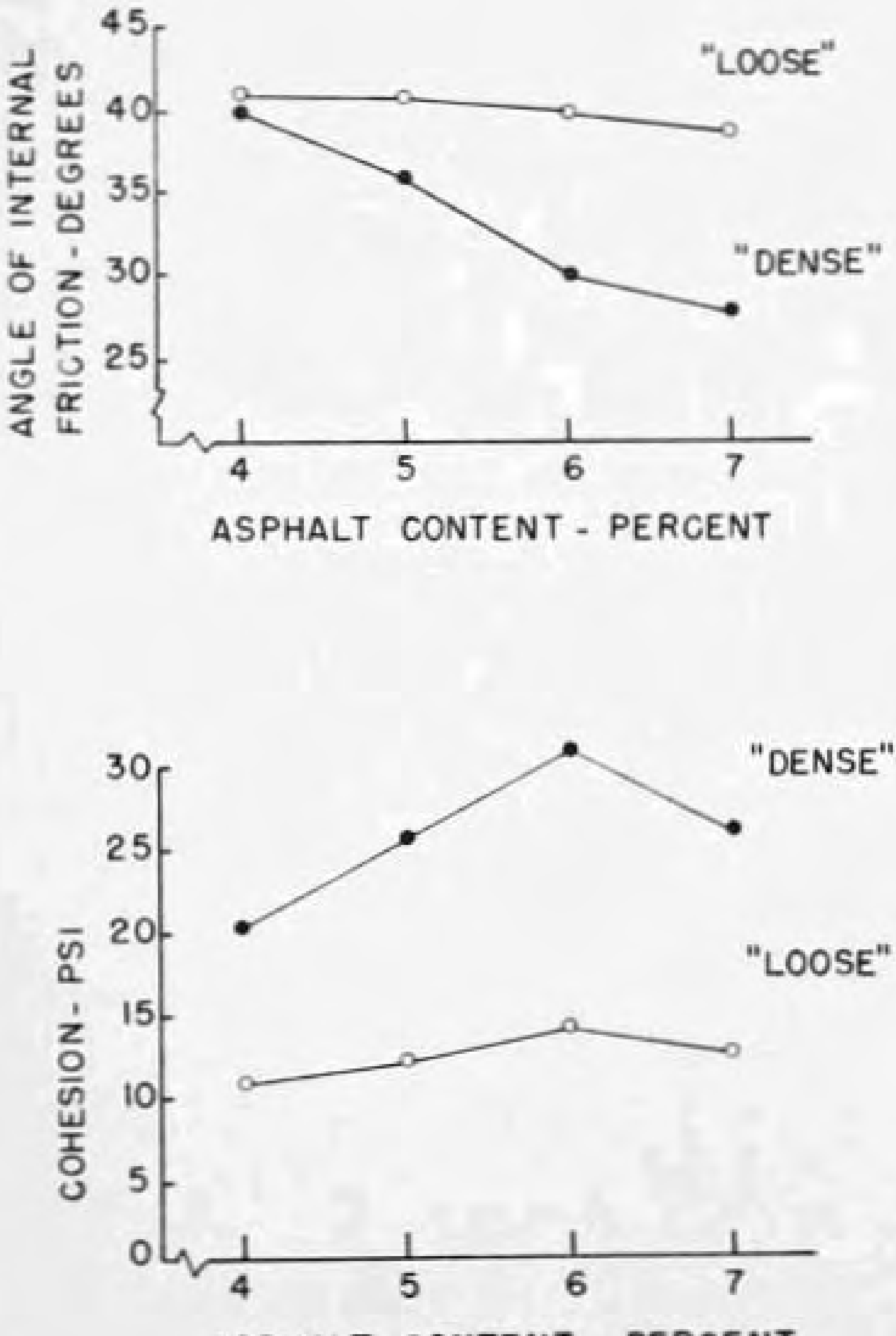


FIG. 12

Hveem

The data obtained from the Hveem Stabilometer and Cohesiometer tests were used to calculate <u>Stability</u> and <u>Cohesiometer Values</u> according to the equations of Hveem (4). It was found that the bituminous concrete containing 6.0 percent asphalt and compacted to a density of 140 pounds per cubic foot had a Stability of 20.8 and a Cohesiometer value of 164. The same mixture compacted to a density of 146 pounds per cubic foot had a Stability of 25.8 and a Cohesiometer value of 267.

Triaxial Tests-Controlled Density Specimens

The results of the triaxial tests on specimens made at an asphalt content of 6.0 percent and to controlled density values of 140 and 146 pounds per cubic foot are given in Table 8. These results were plotted as shown in Figure 13 and the cohesion and angle of internal friction for the mixture at each density was computed. The cohesion and angle of internal friction results are also shown in Figure 13.

Table 8

Results of Triaxial Tests on Controlled - Density Specimens

6.0 Percent Asphalt - 140 and 146 lbs/ft^3

Specimen Number	Unit Weight of Mix lbs per ft^3	Confining Pressure psi	Deviator Load At Failure lbs	Total Stress At Failure psi
1			1450	130
2			1550	139
Avg.	140	15	1500	134
3			2050	193
4			2050	193
Avg.	140	30	2050	193
5			4570	485
6			4700	495
Avg.	140	120	4635	490
7			2100	182
8			2200	190
Avg.	146	15	2150	186
9			2300	213
10			2550	233
Avg.	146	30	2425	223
11			5060	524
12			5200	534
Avg.	146	120	5130	529

TRIAXIAL TEST RESULTS - CONTROLLED DENSITY SERIES

COMPRESSIVE STRENGTH VS. LATERAL PRESSURE

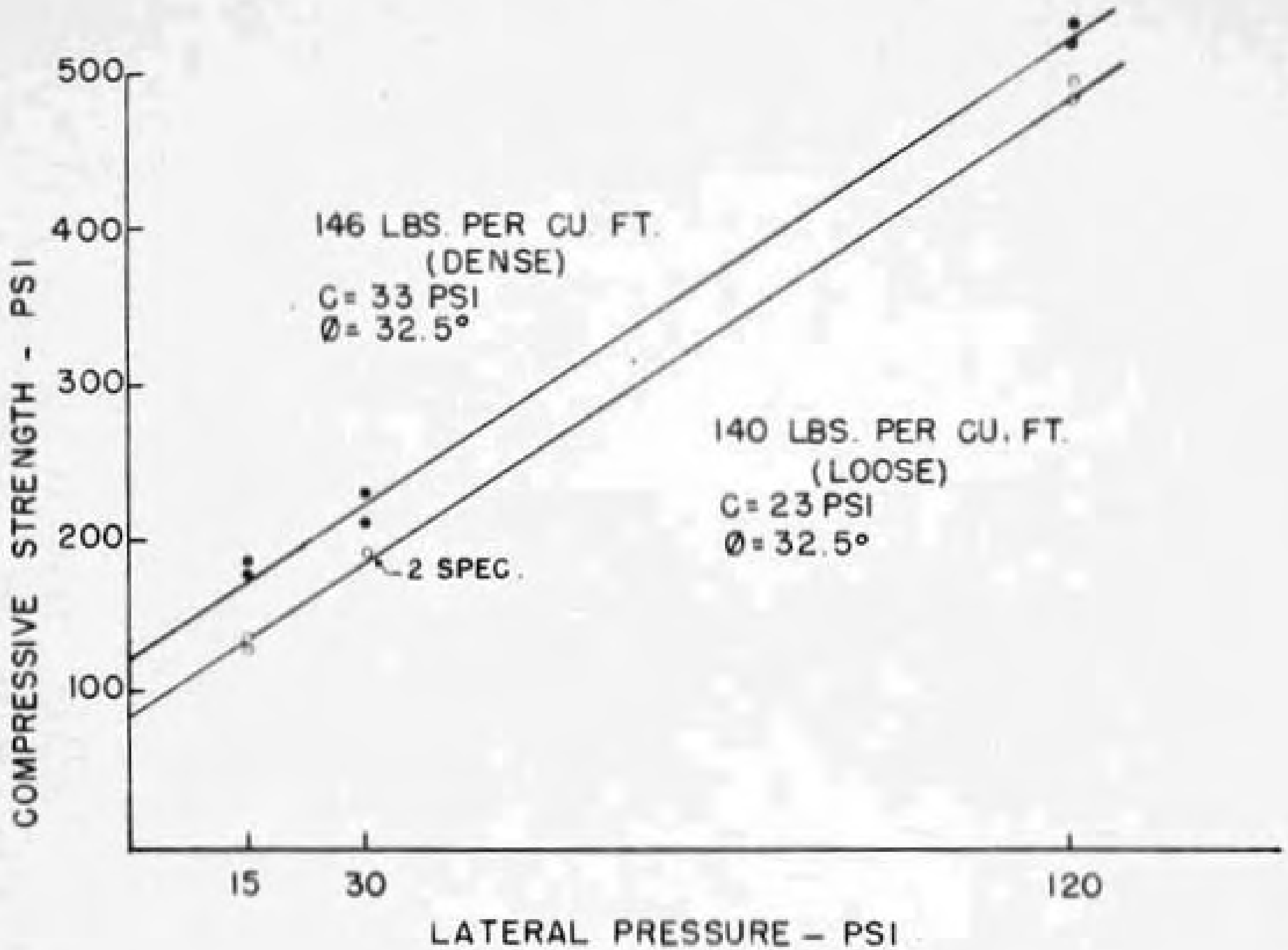


FIG. 13

Variable Specimen Area Test Series

Specimens containing 6.0 percent asphalt were made by vibrational compaction to density values of 140 and 146 pounds per cubic foot. These specimens, all two-inches thick, were made four-inches in diameter, 8 x 8 inches, 11 x 11 inches, and 16 x 16 inches. They were all loaded in compression with a four-inch diameter circular bearing plate and tested to failure.

The results of these tests are shown in Table 9. In Figure 14, the strength values of the various-size specimens are shown plotted against the ratio of the area of the specimen to the loaded area.

Table 9

Compression Test Results-Two-Inch Thick Slab Specimens

Variable Specimen Area Test Series

Specimen Number and Size	Unit Weight of Mix lbs per ft^3	Specimen Area / Loaded Area	Total Load At Failure lbs	Failure Load / Loaded Area psi
1-4 in. dia.			2410	192
2- "			2240	179
Avg.	140	1.0	2325	185
1-8x8 in.			6000	477
2- "			5900	470
Avg.	140	5.1	5950	473
1-11x11 in.			7200	575
2- "			7300	581
Avg.	140	9.7	7250	578
1-16x16 in.			7610	606
2- "			7720	614
Avg.	140	20.4	7660	610
1-4 in. dia.			2880	229
2- "			3050	243
Avg.	146	1.0	2965	236
1-8x8 in.			6380	508
2- "			6800	541
3- "			6800	541
Avg.	146	5.1	6660	530
1-11x11 in.			8600	685
2- "			9700	773
3- "			8400	670
Avg.	146	9.7	8900	709
1-16x16 in.			9580	763
2- "			10,020	797
Avg.	146	20.4	9750	780

COMPRESSION TEST RESULTS - VARIABLE SPECIMEN AREA SERIES

COMPRESSIVE STRENGTH VS. RATIO OF SPECIMEN AREA TO LOADED AREA

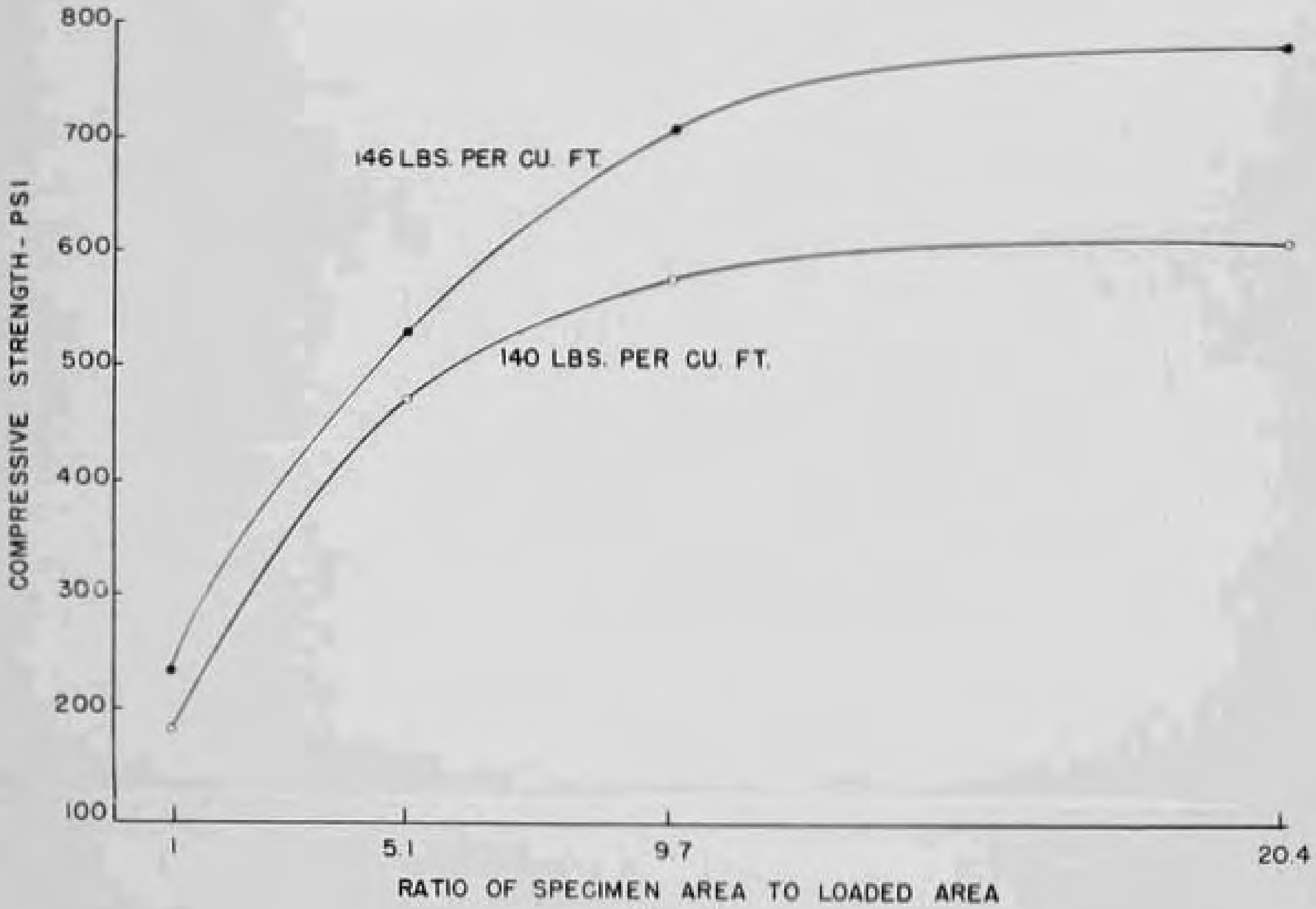


FIG. 14

Strength-Rate-Temperature Series

The results of the series of tests which were performed to determine the relationship between the strength of a thin specimen of bituminous concrete and the factors of rate of deformation and temperature are shown in Table 10.

During this test series it appeared as if the specimens made by vibrational compaction were different from the specimens made by double-plunger static compaction with respect to strain properties. The values for total strain at failure for these specimens are included, in Table 10.

Table 10

Results of Strength-Rate-Temperature Series

Laboratory-Compacted Specimens

Temp. °F	Specimen	Total Load At Failure - Pounds			
		Vibrated		Double-Plunger Compacted	
		Rate of Deformation - in/min.			
		0.2	0.02	0.2	0.02
40	1	12,310	9000	19,580	12,500
	2	14,900	8150	23,000	9,000
	Avg.	13,605	8575	21,290	10,750
90	1	3790	2800	5230	3100
	2	4100	3200	4780	2800
	Avg.	3945	3000	5005	2950
140	1	1240	900	1610	1025
	2	1320	1000	1610	1080
	Avg.	1280	950	1610	1050

Total Deformation at Failure - Thousandths of an Inch

40	140	160	100	70
90	110	125	85	75
140	81	101	68	78

Marshall Comparison Series

Indications that the deformation characteristics of vibrated specimens were different from those of specimens compacted by double-plunger static compaction led to a series of tests in which Marshall-size (4-inch diameter by 2 1/2 inches high) specimens were formed, at an asphalt content of 6.0 percent, by standard Marshall compaction, double-plunger static compaction, and by vibrational compaction. Four specimens of each type were made and tested in the Marshall apparatus. The results of these tests are presented in Table 11.

Table 11

Test Results From
Marshall Comparison Series

(a)
Vibrated Cores Molded to a Density of 146 lbs. per ft^3

Spec. No.	Marshall Stability Pounds	Marshall Flow, 0.01 in.
1	1180	18.9
2	1090	16.5
3	1220	17.5
4	1030	20.0
Avg.	1130	18.2

(b)
Double-Plunger Specimens Compacted to 146 lbs. per ft^3

Spec. No.	Marshall Stability Pounds	Marshall Flow, 0.01 in.
1	1250	10.7
2	1200	11.7
3	1220	9.6
4	1240	11.0
Avg.	1230	10.8

(c)
Specimens Made By Standard Marshall Compaction

Spec. No.	Marshall Stability Pounds	Marshall Flow 0.01 in.	Unit Wt. Mix, lbs. per ft^3
1	1520	12.6	146.8
2	1480	12.6	147.1
3	1260	10.8	145.8
4	1360	10.7	146.1
Avg.	1405	11.7	146.6

RESULTS OF TESTS ON PAVEMENT CORES

Pavement cores were tested to failure in direct compression at various temperatures and rates of deformation in order to establish a relationship among these factors. In addition, two types of repeated load tests were used to study pavement cores.

The results of each of these test series are presented in tabular and graphic form in this section.

Strength-Rate-Temperature Series

Compression tests were made on pavement cores two, three, and four inches thick. These tests were made at three different temperatures (40, 90, and 140° F) and three different rates of deformation (0.2, 0.02, and 0.002 inches per minute). Two cores were tested at each combination of conditions. The results of the tests are shown in Table 12.

Table 12

Results of Strength-Rate-Temperature Series

Pavement Cores 2, 3, and 4 Inches Thick

Thickness of Core Inches	Temperature °F		Total Load at Failure Pounds		
			Rate of Deformation - in. per min.		
			0.2	0.02	0.002
2	40	1	17,500	12,500	8100
		2	19,800	12,100	8270
		Avg.	18,650	12,300	8185
	90	1	5700	4950	3690
		2	6000	5000	3730
		Avg.	5850	4975	3710
	140	1	3540	2460	1750
		2	4050	2330	1860
		Avg.	3795	2390	1805
3	40	1	17,150	11,100	11,200
		2	19,300	12,450	11,580
		Avg.	18,225	11,775	11,390
	90	1	6000	3550	3450
		2	5540	3770	3550
		Avg.	5770	3660	3500
	140	1	2150	1110	940
		2	2270	975	1040
		Avg.	2210	1042	990
4	40	1	14,900	9460	9150
		2	17,000	9300	9050
		Avg.	15,950	9380	9100
	90	1	3850	3250	2360
		2	4380	2180	2440
		Avg.	4115	2715	2400
	140	1	1590	860	490
		2	1240	460	710
		Avg.	1415	660	600

Repeated Load Tests

In this study two types of repeated load tests were made on pavement cores. In the first type, loads representing 75, 50, and 25 percent of the ultimate strength of the core at a temperature of 80° F and a rate of deformation of 0.02 inches per minute were repeatedly imposed upon specimens at this temperature and at this rate of deformation. The cumulative permanent deformation was noted.

The second series of repeated load tests employed a different concept of loading from the first. For this series, the total load was applied almost instantaneously for a period of 0.3 seconds and then released. The time interval between loadings was four seconds. The tests were made at applied stresses of 100, 150, and 200 psi, each at a temperature of 80 and 140° F for all three core thicknesses. As in the case of the first type of repeated load test, a record of the cumulative permanent deformation was kept.

Slow Cycle

A total of eight pavement cores were tested by the slow-cycle method. As this method of test is rather time-consuming, the number of cycles that one may obtain in a day is rather limited. No more than fifty cycles were obtained for any core in this test series and several tests were terminated with a smaller number being obtained.

Wood (67) reports that for his test results a ". . . plot of permanent deformation versus number of load applications starts out as a straight line on a semi-logarithmic plot in all cases. At some stage, dependent upon the applied stress and the number of load applications, the plot deviates sharply from the straight line. The point at which this deviation from linearity occurred was selected as the point of failure . . . "

The approach used by Wood was the one chosen as a first attempt in plotting the repeated load test data collected in the present study. Several of the slow-cycle repeated load tests were plotted this way and these results are shown in Figure 15. The pattern appeared to be similar to that found by Wood. However, when the results of the rapid-cycle repeated load tests were analyzed, it appeared that a better interpretation of the data would be a plot of logarithm cumulative permanent deformation versus logarithm number of load applications. For all the repeated load tests performed for this study, a plot of this sort has an initial linear portion. In addition, it was discovered in the rapid-cycle tests that a criterion for failure could also be derived from this plot.

The results of the slow-cycle repeated load tests are shown in tabular form in Table 21 which appears in Appendix A. Figure 15 shows the results of four of the tests plotted in the form: cumulative permanent deformation versus logarithm number of load applications. In Figures 16 and 17 the results of all of the tests are shown plotted graphically in the form: logarithm cumulative permanent deformation versus logarithm number of load applications.

SLOW-CYCLE REPEATED LOAD TEST RESULTS

80°F

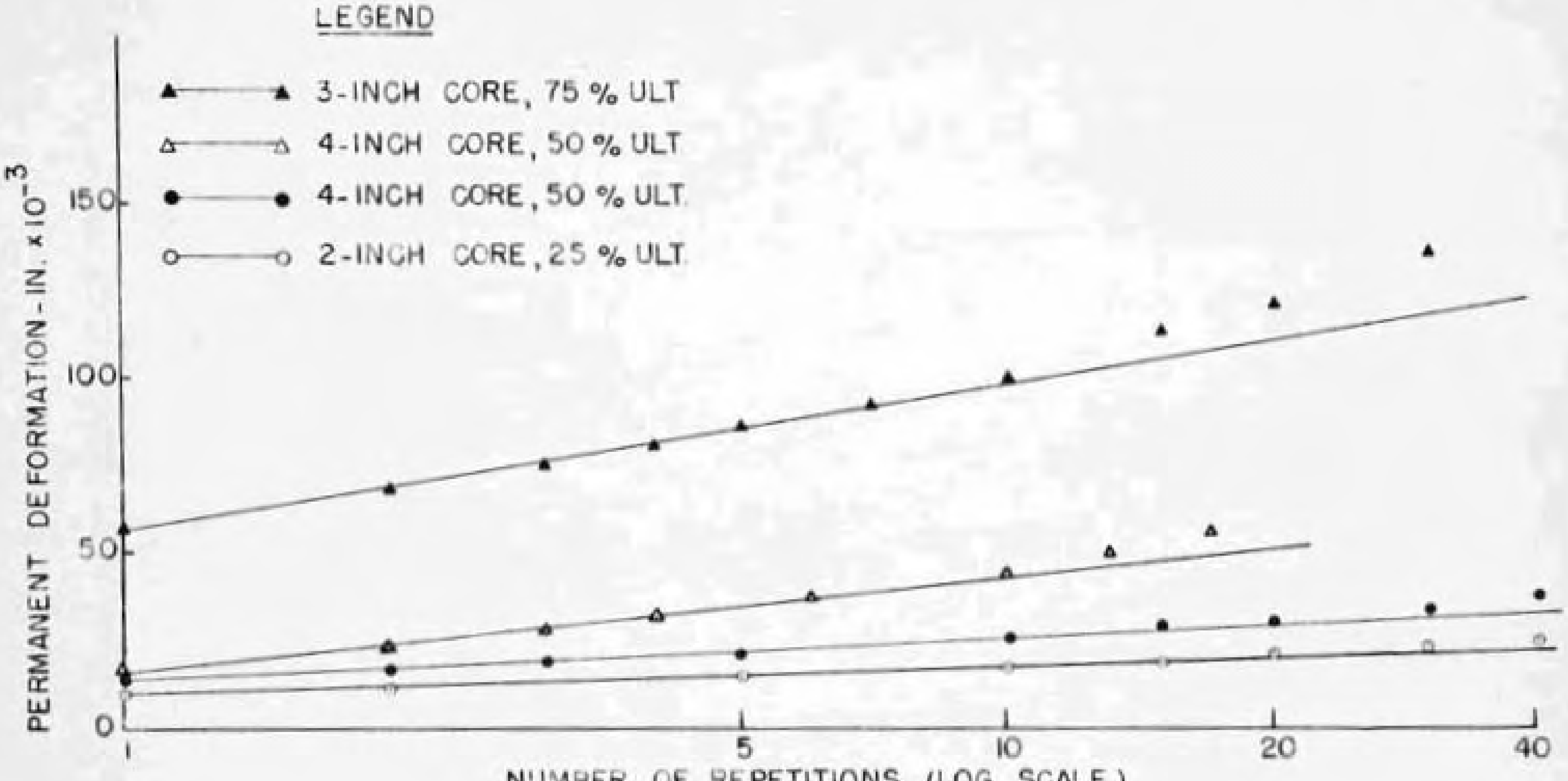


FIG. 15

SLOW-CYCLE REPEATED LOAD TEST RESULTS

80°F

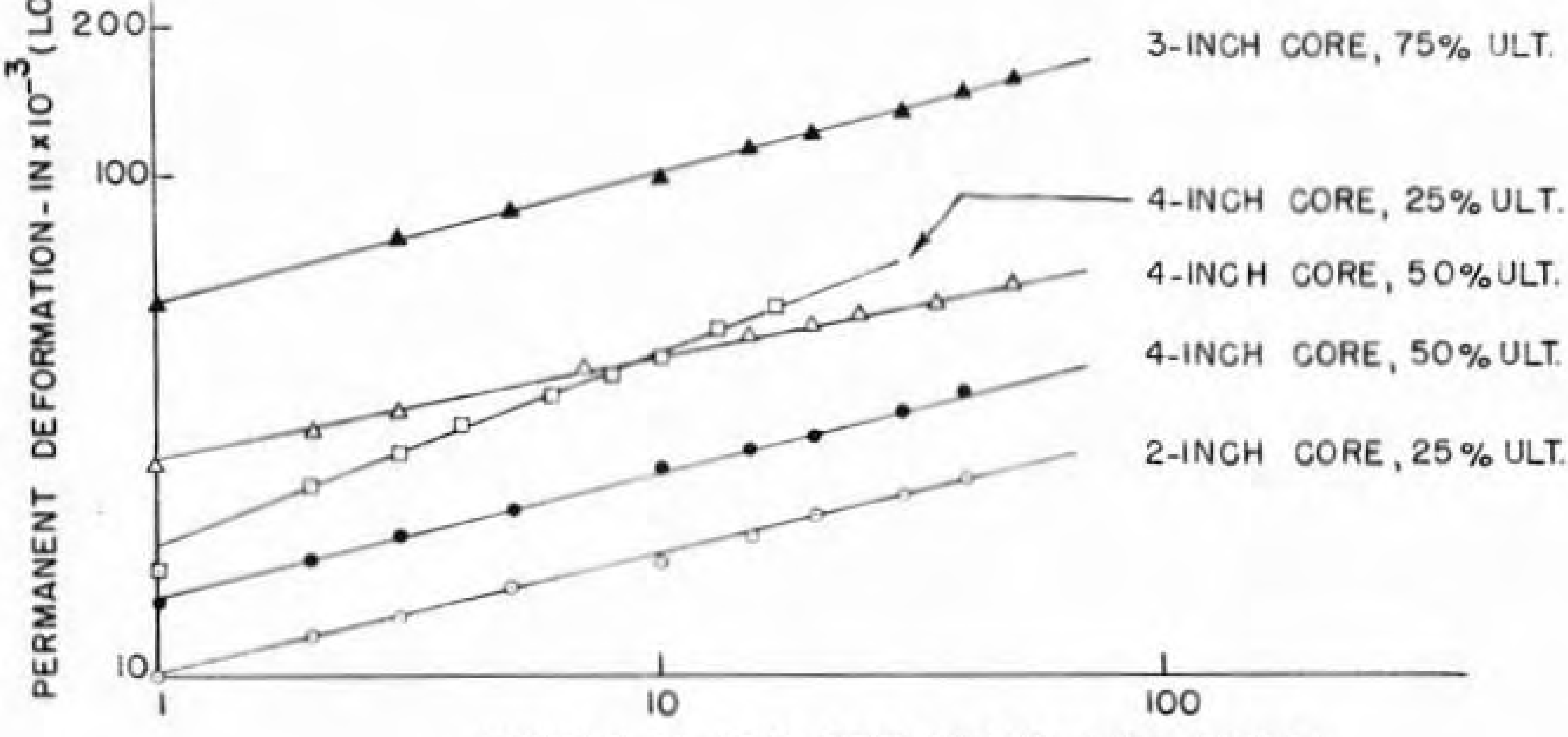
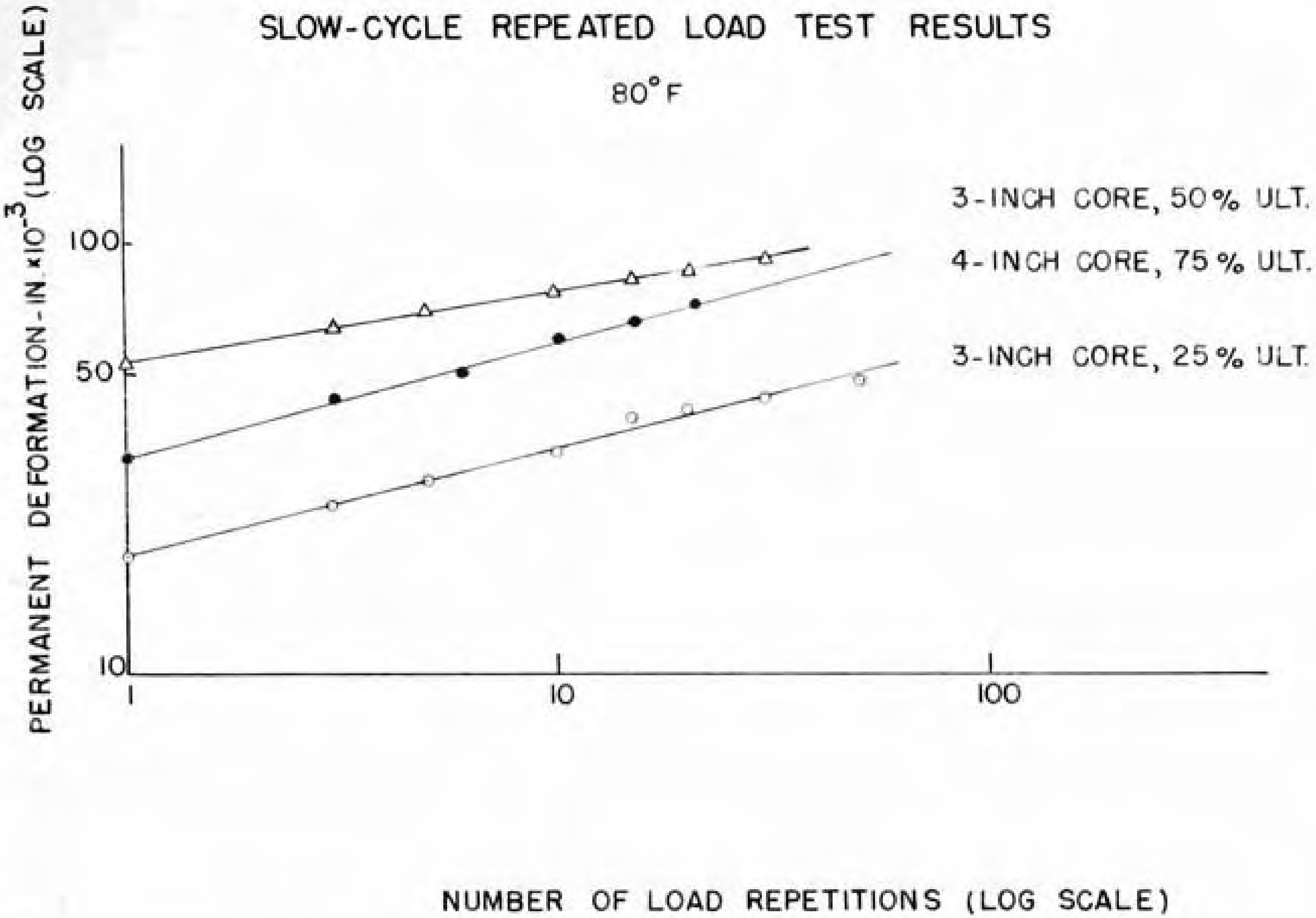


FIG. 16



SLOW-CYCLE REPEATED LOAD TEST RESULTS
80°F
PERMANENT DEFORMATION-IN.×10^-3 (LOG SCALE)
100
50
10
1
10
100
3-INCH CORE, 50% ULT.
4-INCH CORE, 75% ULT.
3-INCH CORE, 25% ULT.
NUMBER OF LOAD REPETITIONS (LOG SCALE)

FIG. 17

Rapid Cycle

The rapid-cycle repeated load tests were carried out to failure or to 2000 load applications. Failure was defined as upward deviation from linearity of a plot of log cumulative permanent deformation versus log number of load applications.

The test results are tabulated in Tables 22 through 27 in Appendix A and are also shown graphically in Figures 18 through 27. Figures 18, 20, 21, 23, 24 and 26 show log-log plots for results of tests on the two- and four-inch cores. Figures 19, 22, and 25 show semi-log plots for some of the same results. Figure 27 is a log-log plot of cumulative permanent deformation versus number of load applications showing all the test results from the tests on three-inch thick cores.

RAPID-CYCLE REPEATED LOAD TEST RESULTS

TWO-AND FOUR-INCH PAVEMENT CORES

80°F - 100 PSI

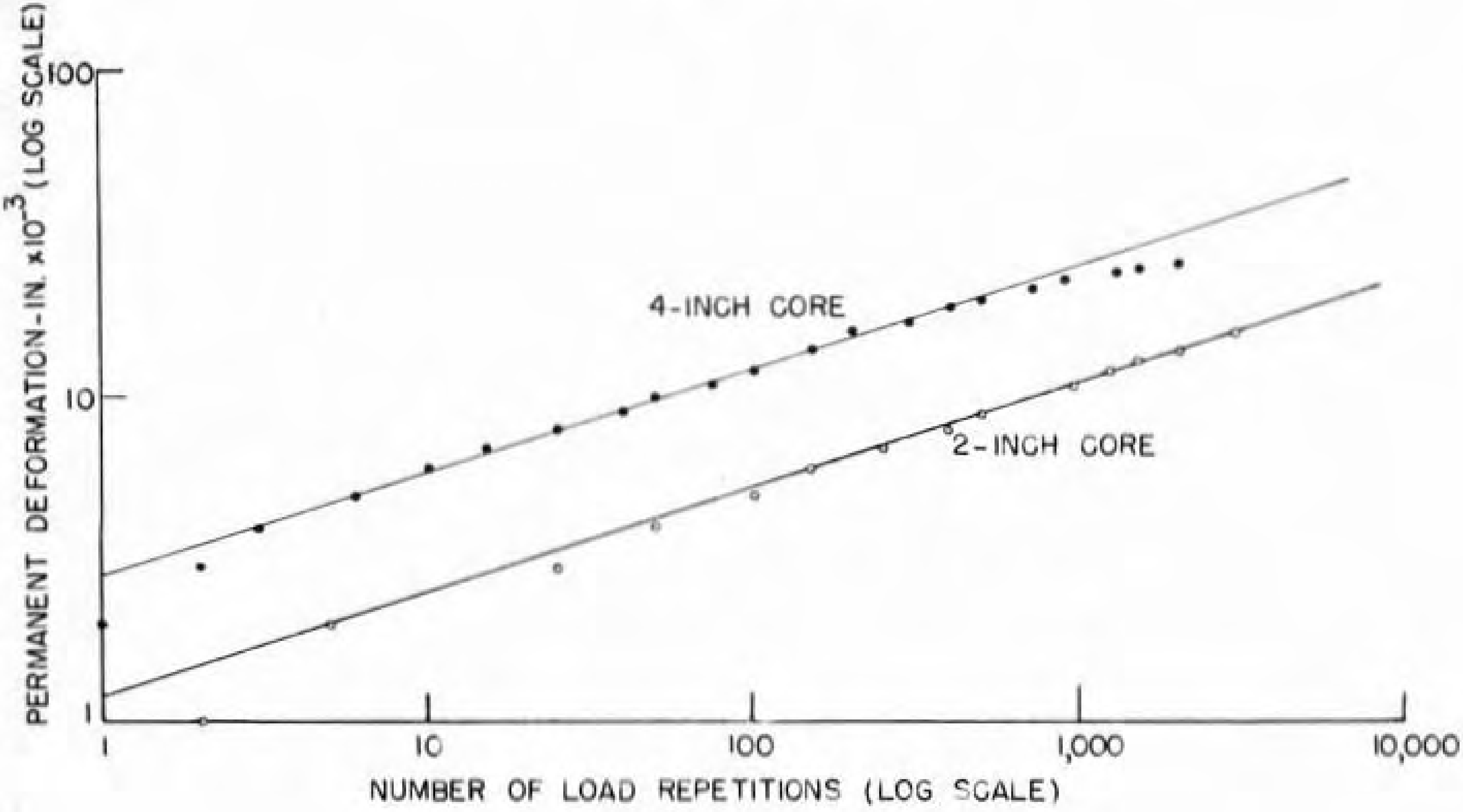


FIG. 18

RAPID-CYCLE REPEATED LOAD TEST RESULTS

TWO-INCH PAVEMENT CORE

80°F - 100 PSI

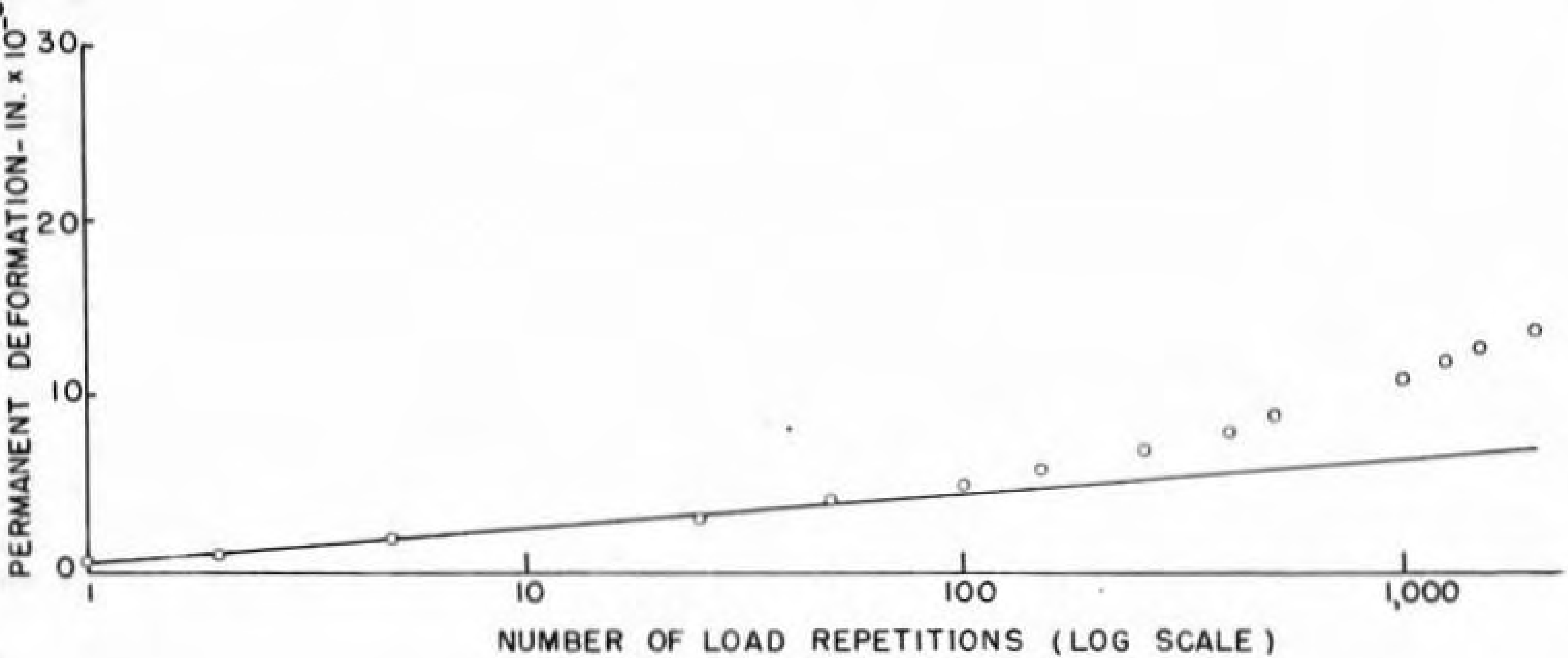


FIG. 19

RAPID-CYCLE REPEATED LOAD TEST RESULTS

TWO-AND FOUR-INCH PAVEMENT CORES

80°F - 150 PSI

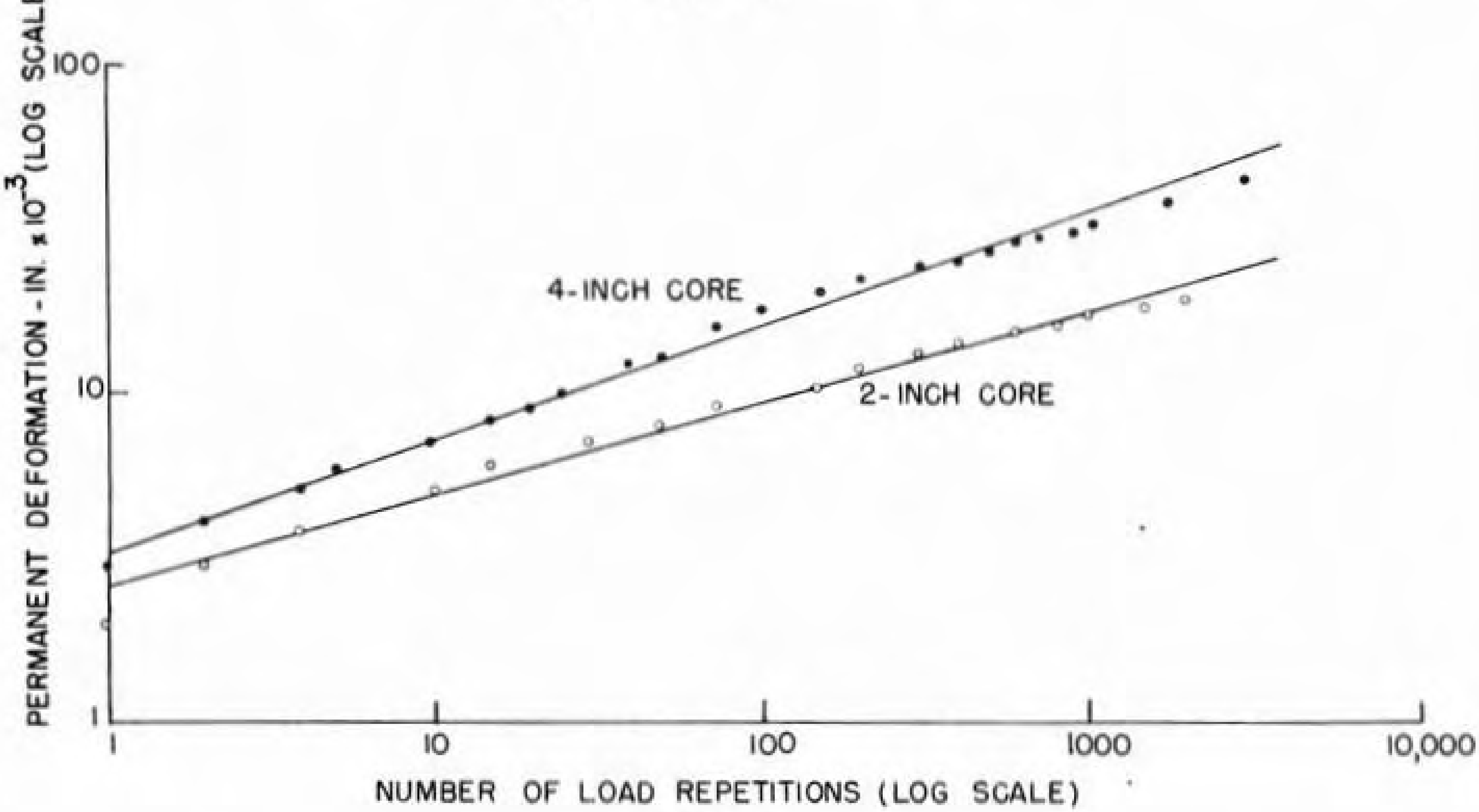


FIG. 20

RAPID-CYCLE REPEATED LOAD TEST RESULTS

TWO-AND FOUR-INCH PAVEMENT CORES

80°F - 200 PSI

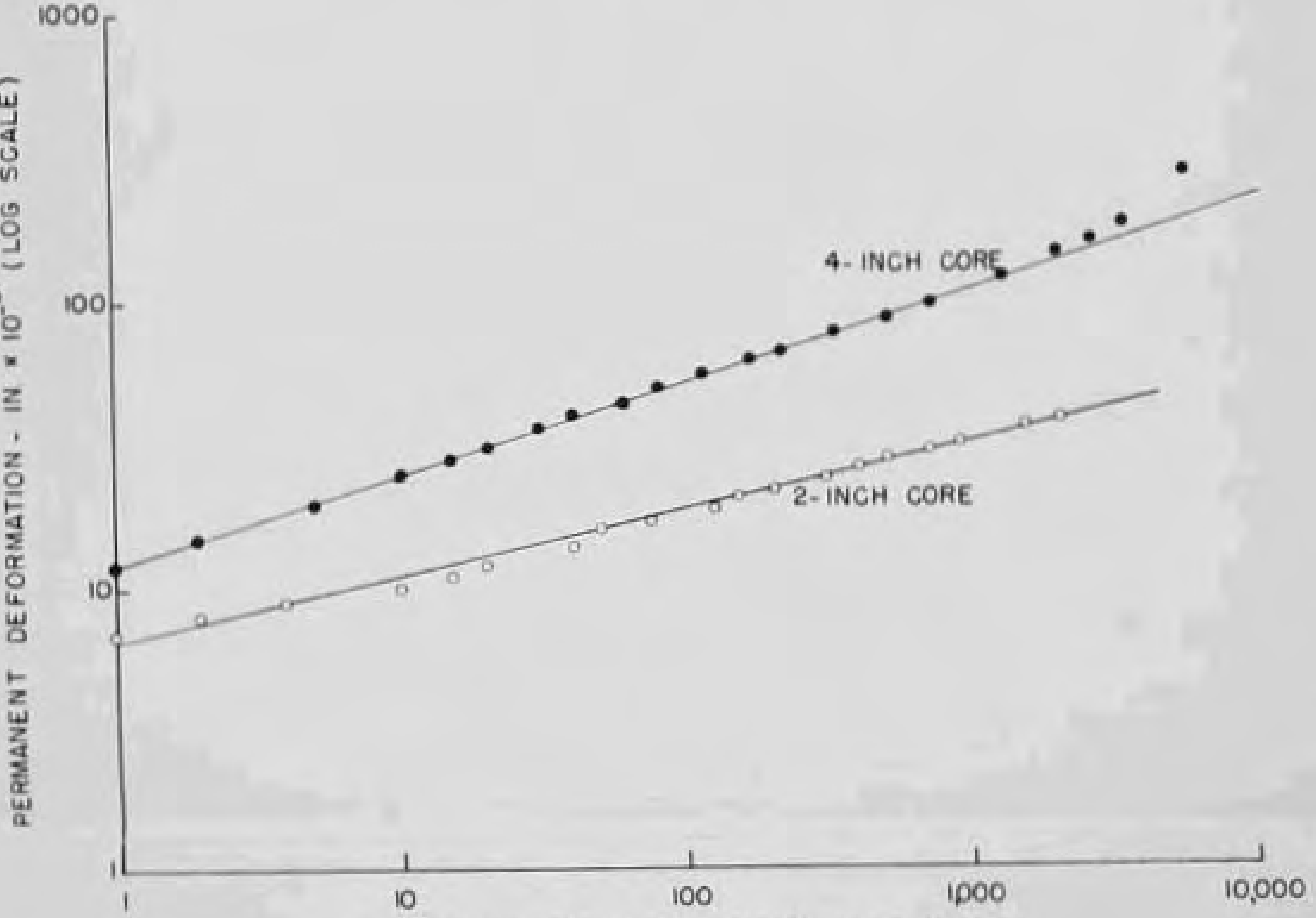


FIG. 21

RAPID-CYCLE REPEATED LOAD TEST RESULTS
FOUR-INCH PAVEMENT CORE
80°F - 200 PSI

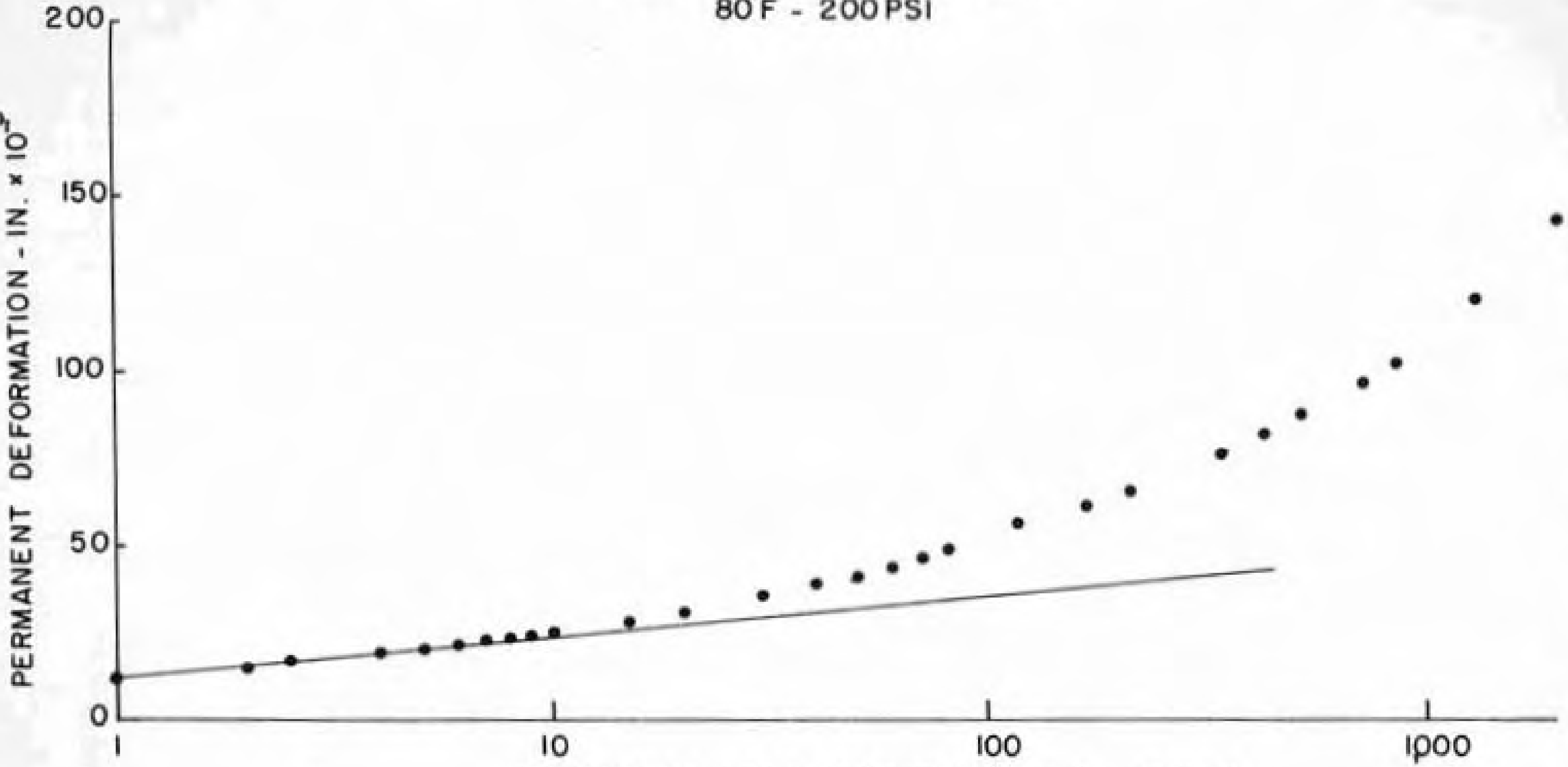


FIG. 22

RAPID-CYCLE REPEATED LOAD TEST RESULTS
TWO-AND FOUR-INCH PAVEMENT CORES
140°F - 100 PSI

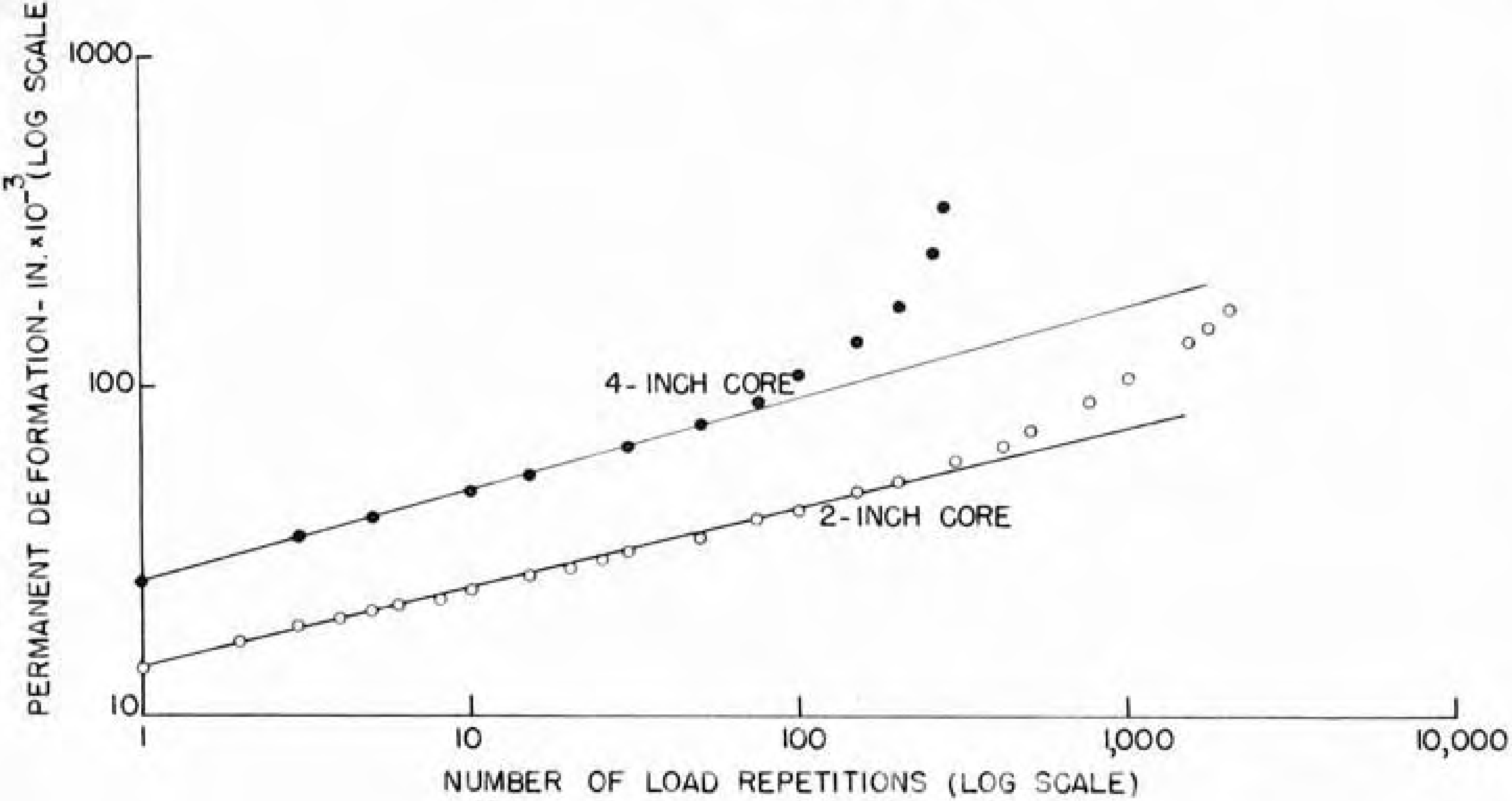


FIG. 23

RAPID-CYCLE REPEATED LOAD TEST RESULTS
TWO-AND FOUR-INCH PAVEMENT CORES
140°F- 150 PSI

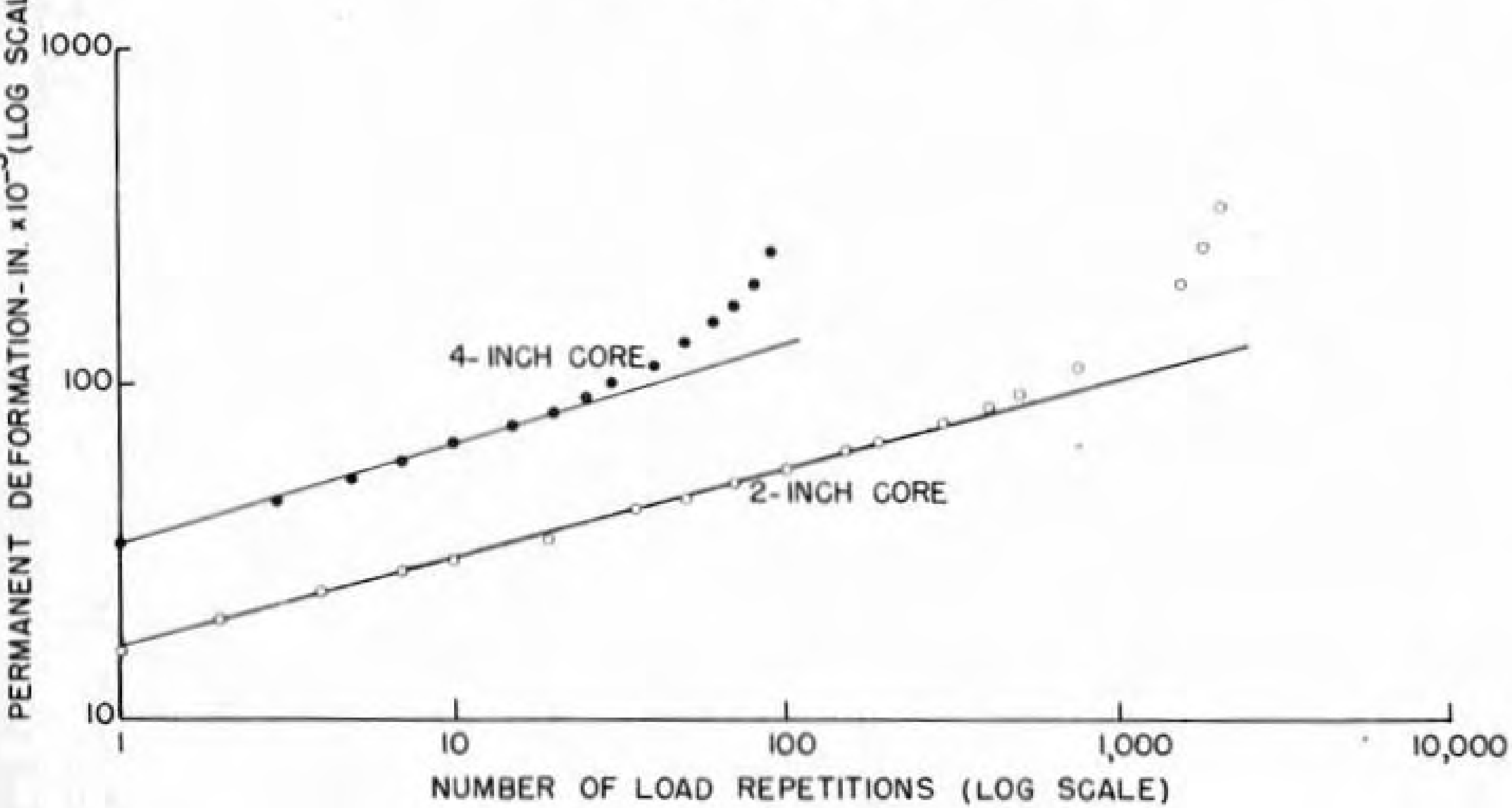


FIG. 24

RAPID-CYCLE REPEATED LOAD TEST RESULTS

TWO-INCH PAVEMENT CORE

140°F - 150 PSI

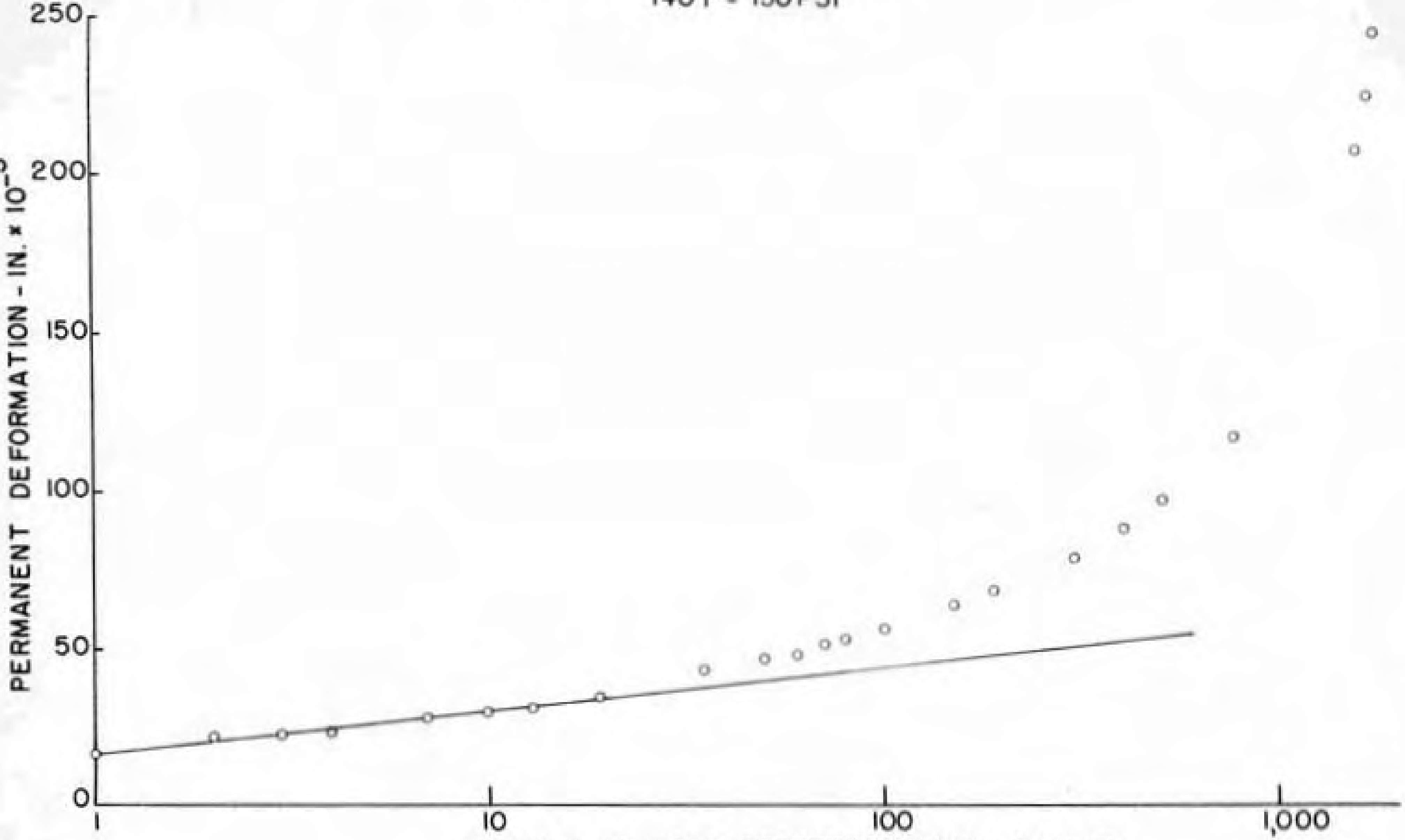


FIG. 25

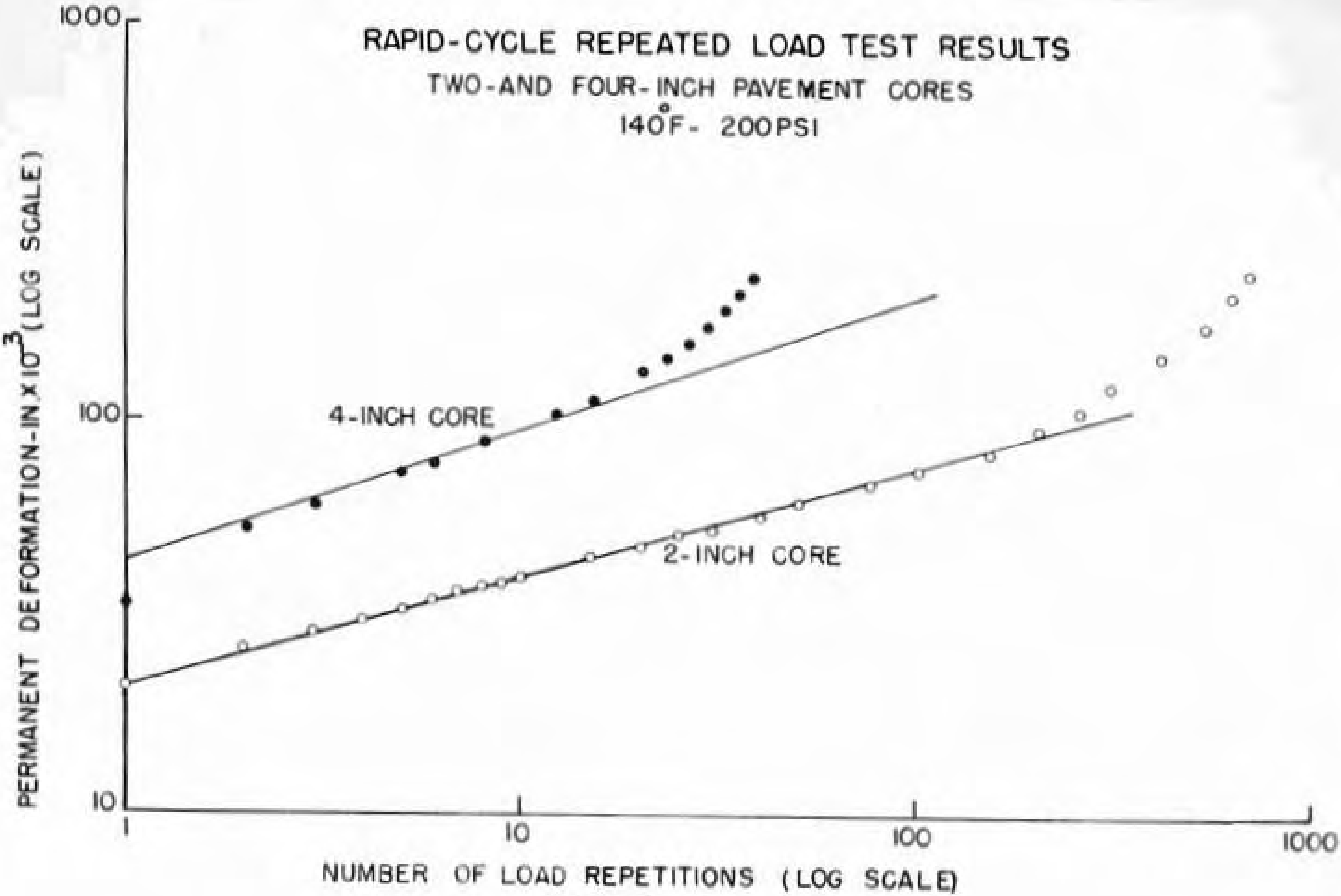

RAPID-CYCLE REPEATED LOAD TEST RESULTS
TWO-AND FOUR-INCH PAVEMENT CORES
140°F - 200 PSI
4-INCH CORE
2-INCH CORE
PERMANENT DEFORMATION-IN.X10^-3 (LOG SCALE)
1000
100
10
1
10
100
1000
NUMBER OF LOAD REPETITIONS (LOG SCALE)

FIG. 26

RAPID-CYCLE REPEATED LOAD TEST RESULTS

THREE-INCH PAVEMENT CORES

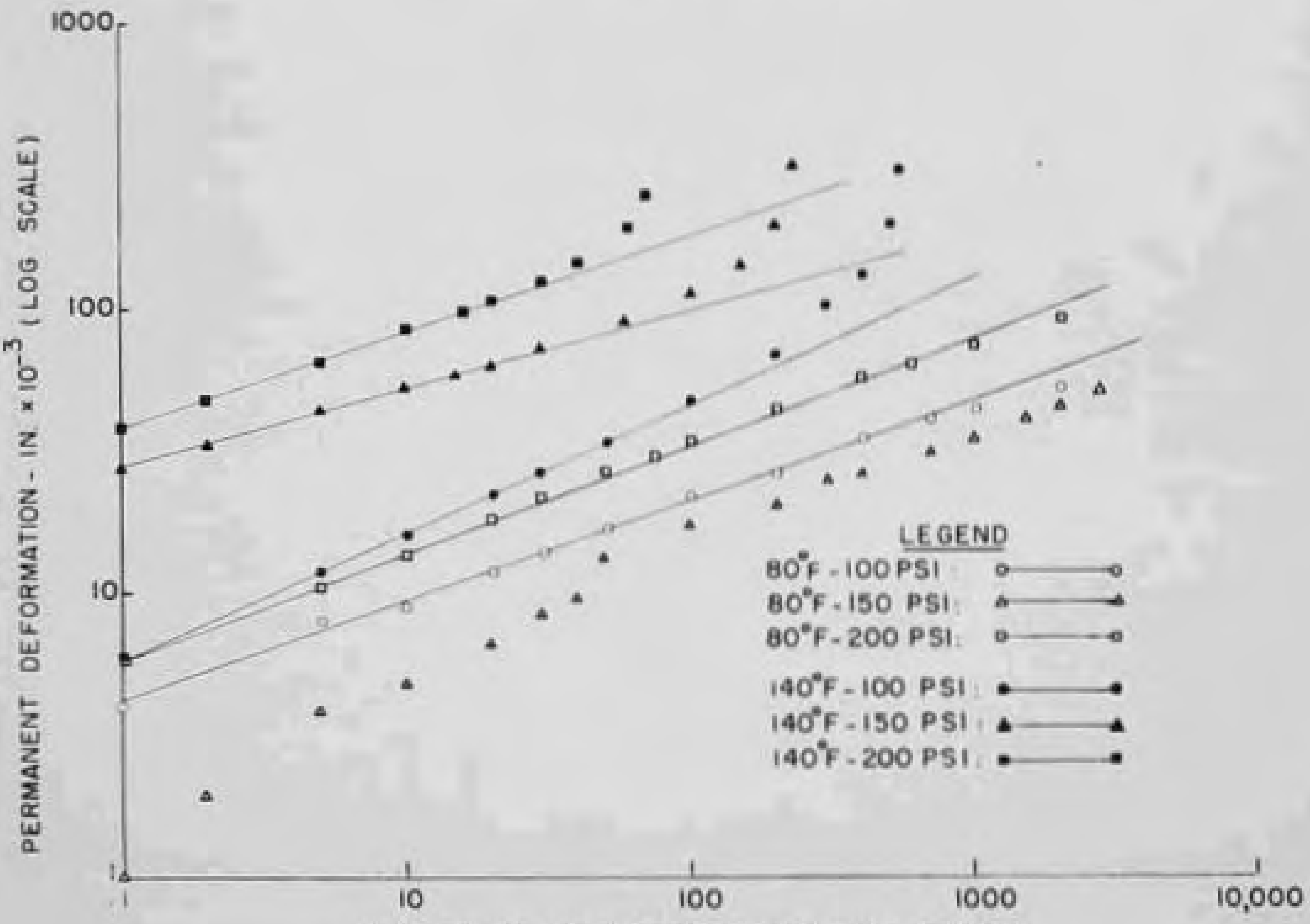


FIG. 27

DISCUSSION OF RESULTS: LABORATORY-COMPACTED SPECIMENS

This section contains a discussion of the results of the tests performed on laboratory-compacted specimens. It is sub-divided into sections dealing with the results of each of the five series of laboratory tests that were performed. These five test series were: a) Characterization Tests, b) Triaxial Tests on Controlled Density Specimens, c) Variable Specimen Area Test Series, d) Strength-Rate-Temperature Series, and d) Marshall Comparison Series.

Characterization Tests

The results of the tests used to characterize the bituminous concrete were within the ranges that one would normally expect with the possible exception of the results from the Hveem Stabilometer and Cohesiometer tests. The Marshall test results interpreted according to the Corps of Engineers design procedure, indicated that a satisfactory mixture might be made from the aggregate blend used, but one cannot make this statement with certainty because the gradation of the blend does not meet the specification of the Corps of Engineers and, strictly speaking, the design criteria cannot be said to apply in the present case.

The triaxial test results show the usual pattern first established by Chen (8). Plotting the cohesion and angle of internal friction values obtained from these tests on the Asphalt Institute's "Triaxial Evaluation Chart" (3), one finds that the mixture, both loose and dense and at every asphalt content tested, is rated as satisfactory. Similar results have been obtained in the past (8, 20, 34).

It might be mentioned that the author does not recommend preparing specimens for the triaxial test by compacting to a fixed contact pressure. Previous experience and experience gained during this study has shown that somewhat erratic compression test results are obtained from specimens prepared in this way. A better method seems to be to compact to a fixed density. In this way, variations in the temperature of the mixture at the time of compaction do not appear to influence the strength results obtained from the specimens.

The Hveem test results pose a problem in interpretation. Most organizations that use the Stabilometer and Cohesiometer specify minimum values of S = 30 or 35 and C = 50 for a satisfactory mixture. Neither mixture tested met the minimum stability requirement of 30. The 140 pounds per cubic foot mixture had an S value of 20.9 and the 146 pounds per cubic foot mixture had an S value of 25.3. However, both mixtures were apparently high in C, having values of 164 and 267 respectively.

The author has mixed feelings about this. He was anything but expert in performing these tests and the possibility exists that he made mistakes in procedure. On the other hand, the results may be entirely valid and if so, application of the Hveem test would appear to warrant further study. In any case, at the present writing Hveem tests on some Indiana binder mixtures are in progress and the results may shed some light on the matter.

Triaxial Tests - Controlled Density Specimens

It appears that up to a confining pressure of 120 psi at least, the Mohr rupture envelope for triaxial tests on the bituminous concrete mixture used in this study is linear (Figure 13). This statement probably should be modified to include only the type of triaxial test performed in this study. That is, constant rate of deformation test at 0.02 inches per minute and at a temperature of about 80° F. However, some authors (39) have speculated on curved envelopes and very little experimental data have been available for confining pressures above 60 psi.

The advantages of high pressure tests are apparent from Figure 13. If one agrees that the envelope of circles is linear to 120 psi and that knowledge of slope and intercept of this envelope is desirable, it is evident that one should obtain test results over as large a range in confining pressures as possible. When values are obtained only at 15 and 30 psi, the results need vary only slightly to obtain rather large errors. In addition, it may be that the experimental error is a greater percentage of mean compressive strength at 15 psi than it is for the mean compressive strength at 120 psi.

Both of the mixtures tested in this series would plot in the satisfactory region of the Asphalt Institute's "Triaxial Evaluation Chart" (3) previously mentioned. The results from the controlled-density tests are comparable to those results obtained from the mixtures containing 6.0 percent asphalt that were tested in the characterization series. Density differences exist between series and the results must be compared with this in mind.

Variable Specimen Area Test Series

Rather large increases in load-carrying capacity of the bituminous concrete were observed for two-inch thick specimens having successively larger areas in the Variable Specimen Area Test Series. In terms of total stress applied to the specimen through the four-inch diameter plate, the mixture compacted to a density of 146 pounds per cubic foot failed at an applied stress of 236 psi when it was in the form of a two-inch thick four-inch diameter core. However, the same mixture compacted to the same density and thickness but in the form of a specimen 16 by 16 inches square failed at an applied stress of 780 psi. Corresponding values for the mixture compacted to a density of 140 pounds per cubic foot are 185 and 610 psi respectively.

Reference to Figure 14 shows that the stress required to cause failure increased by a factor of about 2 or 2 1/2 between specimens having a ratio of specimen area to loaded area of one (4-inch diameter specimens) and those having a ratio of 5.1 (8 by 8-inch slabs). However, further increase in specimen size to 11 by 11 inches produced a somewhat smaller increase in stress for failure. Finally, it appears that the limiting conditions are approached for specimens 16 by 16 inches in size. That is, based on the shape of the curves shown in Figure 14, one would predict that very little difference in the stress required to cause failure would be observed if specimens larger than 16 by 16 inches were tested. The results of this test series indicate that the supporting power of the bituminous concrete for these test conditions may be estimated from the compression tests on the 16- by 16-inch specimens.

A stated purpose of this investigation was to estimate the "degree of confinement" that might exist in a bituminous-concrete overlay. With the data that have been developed from these tests and the triaxial tests, one can make at least an approximation of this confinement.

Stated in another way, the problem appears to be this: if a bituminous concrete produces a certain set of values for cohesion and angle of internal friction from a triaxial test, what would be the maximum stress that could be applied to this material when it was in a thin layer (thickness equals radius of loaded area for these tests) and loaded over an area that was small relative to the area of the mixture? The results of the triaxial tests on controlled-density specimens (Figure 13) are reproduced in Figure 28. The lines relating total stress at failure to confining pressure have been extrapolated to include the maximum stresses found for the 16- by 16-inch specimens. The dashed lines in Figure 28 indicate the values of confining pressure that would have to be used in triaxial tests of the conventional type in order to produce the maximum stresses of 780 and 610 psi that were determined from the tests on the 16- by 16-inch slabs two inches in thickness compacted to 146 and 140 pounds per cubic foot and tested with a four-inch diameter plate. The values of confining pressure so determined are 196 and 148 psi respectively.

These values are, to this author, surprisingly high. They exceed the unconfined compressive strengths of the mixtures, the latter values being estimated from Figure 28 as 125 and 90 psi, respectively. These results, however, appear to reinforce the hypothesis of McLeod (38) who has long maintained that the confinement in a bituminous layer is <u>at</u>

ESTIMATE OF CONFINEMENT IN TWO-INCH THICK BITUMINOUS-CONCRETE SLAB SPECIMEN

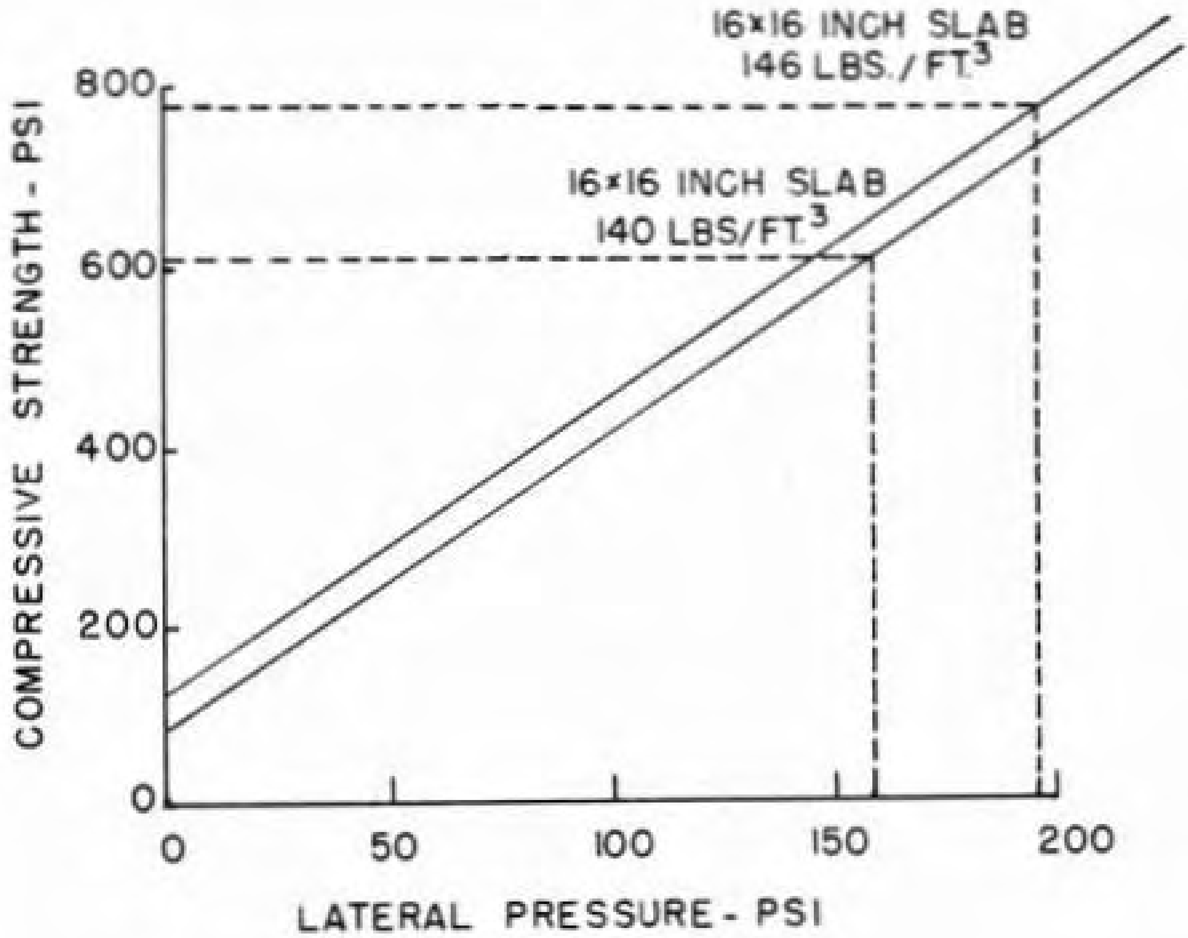


FIG. 28

<u>least</u> equal to its unconfined compressive strength.

There are certainly many limitations to the work reported in this section of the study and one would not generalize from these results to say that the confinement was any given value. Only a single thickness of layer at a single temperature and rate of deformation was tested. In addition, the area over which the load was applied was not varied. Since it is known that these factors affect the strength properties of a bituminous mixture, further investigation would be necessary if precise relationships were to be derived. In spite of these limitations, the results do indicate that, for the test conditions used in this study, the load that can be carried by the relatively thin layer of bituminous concrete is at least several times the unconfined compressive strength of the mixture when tested at the same temperature and rate of deformation.

Strength-Rate-Temperature Series

In analyzing the results of unconfined compression tests on sheet-asphalt mixtures, Wood (67) found that the relationship between the maximum unconfined compressive stress and the variables of temperature and rate of loading appeared to be:

$$x_o = A^{Bx_1^{(Cx_2 + D)}} \quad \ldots\ldots\ldots\ldots 1$$

where x_o = maximum unconfined compressive stress

x_1 = rate of deformation

x_2 = temperature

A, B, C, D = constants

He made use of logarithmic transformations to obtain a linear equation and analyzed his data by multiple linear regression obtaining an estimation equation which related the variables.

Since it was a purpose of this investigation to discover whether this form of relationship exists for bituminous concrete, the results from the Strength-Rate-Temperature Series (Table 10) on laboratory compacted specimens (specimens formed by double-plunger compaction and core specimens from vibrated slabs) were analyzed in a manner similar to that used by Wood. The first step in the analysis was to transform equation 1 into a form that would lend itself to multiple linear regression. By taking the logarithm of both sides of this equation twice one obtains an equation which may be expressed as follows:

$$\log \log x_o = A' + B' \log x_1 + C' x_2 + D' x_2 \log x_1 \quad \ldots 2$$

where A', B', C' and D' are constants

This equation may be used in a multiple linear regression analysis, the factors contributing to the transformed x_0 being the log of the rate of deformation ($\log x_1$), the temperature (x_2), and an interaction or product term having the form temperature times log rate of deformation ($x_2 \log x_1$). Wood used this model and found it to be one which would fit his data very well. This writer on looking over Wood's data, and after preliminary calculations of his own, decided that the purpose at hand would probably be served just as well by omitting the third term in the equation and working with the form:

$$\log \log x_0 = A' + B' \log x_1 + C' x_2 \quad \ldots \ldots \ldots \ldots 3$$

which may be rewritten

$$Y = A' + B' X_1 + C' x_2 \quad \ldots \ldots \ldots \ldots 4$$

in which

$Y = \log \log x_0$, or log log maximum compressive strength, pounds

$X_1 = \log 1000\, x_1$ or log 1000 rate of deformation, the factor of 1000 being inserted to avoid negative logarithms

x_2 = temperature

A', B', C' = constants

The calculations for the multiple linear regression analyses[1] for the mean strengths (lbs.) of core specimens and the double-plunger compacted specimens are shown in Tables 13 and 14. The data on rate of deformation were coded by multiplying each value by 1000 in order to avoid negative logarithms. Since there is zero correlation between temperature and rate of deformation and only two terms involving

---

[1] The theory of multiple regression and the methods of calculation that one may use to obtain an estimation equation are included in many textbooks on statistics (5,46). In addition, Wood (67) gives a brief description of the method he used to obtain his estimation equations.

Table 13

Multiple Linear Regression Analysis and Regression Equation
for
Laboratory-Compacted (Vibrated) Cores

		Maximum Compressive Strength, Pounds	Y log·log Strength	X_1 log 1000×Rate of Deformation	x_2 Temperature °F
	1	13605	0.6163	2.301	40
	2	3945	0.5558	2.301	90
	3	1290	0.4924	2.301	140
	4	8575	0.5947	1.301	40
	5	3000	0.5412	1.301	90
	n = 6	950	0.4739	1.301	140
Sums	ΣV	31355	3.2743	10.806	540
Means	$\bar{V}$	5226	0.5457	1.801	90
	ΣVY		1.802344	5.924364	282.4520
	ΣVx_1			20.96161	972.5400
	ΣVx_2				58,600
	$\frac{\Sigma U \Sigma V}{n}$		1.786840	5.897014	294.6870
				19.46161	972.5400
					48,600
Sums of Squares and Sums of Prod.	$S(VY)$		0.015504	0.027350	-12.235
	$S(Vx_1)$			1.5000	0.000
	$S(Vx_2)$				10,000
r^2's and r's	VY r^2		1.0000	0.0322	0.9655
	VY r		1.0000	0.1795	-0.9825
	Vx_1 r^2			1.0000	0.0000
	Vx_1 r			1.0000	0.0000
	Vx_2 r^2				1.0000
	Vx_2 r				1.0000

Regression Equation:

$$\hat{Y} = 0.5457 + 0.0182\,(X_1 - 1.801) - 0.0012\,(x_2 - 90)$$

for which

$$R^2 = 0.0322 + 0.9655 = 0.9977$$

Table 14

Multiple Linear Regression Analysis and Regression Equation
for
Laboratory-Compacted (Double-Plunger) Specimens

		Maximum Compressive	Y	X_1	x_2
		Strength, Pounds	log log Strength	log 1000xRate of Deformation	Temperature °F
	1	21290	0.6363	2.301	40
	2	5005	0.5631	2.301	90
	3	1610	0.5061	2.301	140
	4	10750	0.6054	1.301	40
	5	2950	0.5403	1.301	90
	n = 6	1050	0.4802	1.301	140
Sums	ΣV	42655	3.3364	10.806	540
Means	$\bar{V}$	7109	0.5561	1.801	90
	ΣVY		1.872778	6.051156	287.5060
	ΣVx_1			20.96161	972.5400
	ΣVx_2				58,600
	$\frac{\Sigma U \Sigma V}{n}$		1.855261	6.008856	300.2760
				19.461606	972.5400
					48,600
Sums of Squares and Sums of Prod.	$S(VY)$		0.017517	0.042300	-12.770
	$S(Vx_1)$			1.5000	0.0000
	$S(Vx_2)$				10,000
r^2's and r's	VY r^2		1.0000	0.0681	0.9309
	VY r		1.0000	0.2610	-0.9648
	Vx_1 r^2			1.0000	0.0000
	Vx_1 r			1.0000	0.0000
	Vx_2 r^2				1.0000
	Vx_2 r				1.0000

Regression Equation:

$$\hat{Y} = 0.5561 + 0.0282\,(X_1 - 1.801) - 0.0013\,(x_2 - 90)$$

for which $$R^2 = 0.061 + 0.9309 = 0.9990$$

variables on the right-hand side of equation 4 the calculation of the estimation equation becomes simple. The regression equations appear in Tables 13 and 14.

A squared multiple correlation coefficient (R^2) was computed for each analysis and is reported in Table 13 for the core specimens cut from slabs formed by vibration and in Table 14 for the double-plunger compacted specimens. Strictly defined R^2 is "the proportion of the sum of squares of the dependent variable which is explained by the multiple regression equation" (46). In perhaps less precise terms, R^2 is that proportion of the variation in the data that can be accounted for by the multiple regression equation. This means that in each of the two analyses presented, over 99 percent of the variation in the results can be accounted for by the equation to which the data have been fitted. This would seem to be an indication that the model chosen by Wood and used herein (with slight modification) is a good one not only for sheet-asphalt mixtures but also for bituminous concrete tested in the manner described for the tests discussed in this section.

The two regression equations themselves are similar. It can be seen that the mean strength for the cores was lower than that for the double-plunger compacted specimens (5226 pounds versus 7109 pounds) and this is no doubt due to the method of compaction. However, when one compares the coefficients of the x_2 (temperature) terms, which are the ones which contribute the most toward the total value of Y, they can be seen to be almost identical (0.0012 vs. 0.0013). This is not the case for the coefficients of the X_1 (rate of loading) terms which are 0.0182 and 0.0282. These results indicate that the differences in strength between specimens made by the two methods of compaction are related to the rate of deformation. This may be a point which is worthy of further investigation.

From these observations one may state that equation 1 is a satisfactory model for relating the strength of a bituminous mixture to the variables of temperature and rate of deformation and that changes in rate of deformation influence the strength obtained from a core (vibrated) specimen to a somewhat lesser extent than a corresponding change for a double-plunger compacted specimen. The strength values for both types of specimens appear to be influenced by temperature to the same extent.

The differences in the amount of total deformation at failure between the core specimens and double-plunger compacted samples (Table 10) is further evidence that the method of compaction is an important factor in laboratory tests on bituminous mixtures. No conclusions were drawn from these results, but it is suggested that further studies might investigate the possibility of developing failure criteria for bituminous concrete based on a limiting strain concept.

Marshall Comparison Series

The Marshall comparison series was conducted to see if some light could be cast on the matter of differences in deformation properties between samples made by three methods of compaction. The results (Table 11) show, as did the results shown in Table 10, that specimens made by vibrational compaction differ with respect to strain properties from specimens made by double-plunger compaction or by Marshall compaction. Since deformation characteristics appear to be an important property of a bituminous concrete, any laboratory test which is to be used for mixture design should reflect strain characteristics similar in magnitude to those which would be obtained for the mixture when placed by rolling.

There is no evidence in the data to show that the vibrational method of compaction is more closely allied to compaction by rolling than double-plunger compaction, but the author believes from observations that this may be the case. In any case, knowing that different methods of compaction may give rise to different strain properties, one might question the practice of taking pavement cores, testing them by the Marshall method, and then using criteria for satisfactory flow values that were developed from laboratory-compacted specimen test results.

DISCUSSION OF RESULTS: PAVEMENT CORES

Two types of laboratory tests were conducted on pavement cores. These were: a) a series of compression tests on 4-inch diameter cores of three different heights at various temperatures and rates of deformation and b) repeated load tests. The tests results have previously been given in tables and charts. In this section, the results of these tests are discussed with emphasis being given to the significance of the findings.

Strength-Rate-Temperature Series

The same type of multiple linear regression analysis that was discussed and described in the section "Strength-Rate-Temperature Series" under the major heading, DISCUSSION OF RESULTS: LABORATORY COMPACTED SPECIMENS was applied to the results of the compression tests of the pavement cores. In the present case three separate analyses were made, one for each of the three core thicknesses. Since the compression tests were performed at three different rates of deformation, the analyses for the pavement cores extend over a wider range of rate of deformation than did the analyses for the laboratory-compacted specimens.

The same general model was used for the regression analyses; that is,

$$Y = A' + B' X_1 + C' x_2 \quad . . . . 4$$

where Y = log log maximum compressive strength, pounds

X_1 = log 100 x_1 rate of deformation, inches per minute

x_2 = temperature, ^{o}F

A', B', C' = constants.

The calculations for and the results of the multiple linear regression analyses for the mean strengths of the three core thicknesses are shown in Tables 15, 16 and 17. The squared multiple correlation coefficients (R^2) were determined to be 0.98, 0.97, and 0.98 for the 2-inch, 3-inch and 4-inch cores, respectively. This again indicates that the general model (equation 1 or equation 4) derived by Wood is one that explains the relationship between the strength of a bituminous mixture and the variables of temperature and rate of deformation. There is now evidence that the relationship holds for sand asphalt tested in rational-sized specimens, for laboratory-compacted 2-inch thick specimens of bituminous concrete made by two methods of compaction, and for pavement cores having different compositions and thicknesses. In addition, all of the results show temperature to be the dominant factor in the strength-rate-temperature relationship.

The regression equations derived and shown in Tables 15, 16, and 17 are similar to each other and to those shown in Tables 13 and 14. No comparisons will be made between the equations derived for the laboratory compacted specimens and those derived for the pavement cores because the compositions of the two types of specimens are quite dissimilar. Comparing the equations shown in Tables 15-17, however, shows that the influence of temperature appears to be larger for the thicker specimens. No relationship of this sort is apparent for the influence of rate of deformation. As the rate of deformation can be said to account for only six percent of the variation in the results in the case of the 3-inch and 4-inch cores and fifteen percent of the variation in the case of the 2-inch cores, it is not too surprising that a trend is not established.

Table 15

Multiple Linear Regression Analysis and Regression Equation
for
Two-Inch Thick Pavement Cores

		Maximum Compressive Strength, Pounds	Y log log Strength	X_1 log 1000xRate of Deformation	x_2 Temperature °F
	1	18650	0.6305	2.301	40
	2	5850	0.5760	2.301	90
	3	3795	0.5538	2.301	140
	4	12300	0.6117	1.301	40
	5	4975	0.5678	1.301	90
	6	2390	0.5287	1.301	140
	7	8185	0.5925	0.301	40
	8	3710	0.5526	0.301	90
	n = 9	1805	0.5128	0.301	140
Sums	ΣV	61660	5.1264	11.709	810
Means	$\bar{V}$	6851	0.5696	1.301	90
	ΣVY		2.931485	6.771846	449.406
	ΣVX_1			21.233409	1053.810
	ΣVx_2				87,900.
	$\frac{\Sigma U \Sigma V}{n}$		2.919997	6.669446	461.376
				15.233409	1053.810
					72,900
Sums of Squares and Sums of Prod.	$S(VY)$		0.011488	0.102400	-11.970
	$S(Vx_1)$			6.0000	0.000
	$S(Vx_2)$				15,000
r^2's and r's	VY r^2		1.0000	0.1521	0.8315
	r		1.0000	0.3900	- 0.9119
	Vx_1 r^2			1.0000	0.0000
	r			1.0000	0.0000
	Vx_2 r^2				1.0000
	r				1.0000

Regression Equation:

$$\hat{Y} = 0.5696 + 0.0171 (X_1 - 1.301) - 0.0008 (x_2 - 90)$$

for which

$$R^2 = 0.1521 + 0.8315 = 0.9836$$

Table 16

Multiple Linear Regression Analysis and Regression Equation
for
Three-Inch Thick Pavement Cores

		Maximum Compressive Strength, Pounds	Y log log Strength	X_1 log 10000xRate of Deformation	x_2 Temperature °F
	1	18225	0.6295	2.301	40
	2	5770	0.5753	2.301	90
	3	2210	0.5243	2.301	140
	4	11775	0.6097	1.301	40
	5	3660	0.5519	1.301	90
	6	1042	0.4797	1.301	140
	7	11390	0.6082	0.301	40
	8	3500	0.5495	0.301	90
	n = 9	990	0.4765	0.301	140
Sums	ΣV	58562	5.0046	11.709	810
Means	$\bar{V}$	6507	0.5561	1.301	90
	ΣVY		2.807480	6.605885	437.069
	$\Sigma V x_1$			21.233409	1053.810
	$\Sigma V x_2$				87,900
	$\frac{\Sigma U \Sigma V}{n}$		2.782891	6.510985	450.414
				15.233409	1053.810
					72,900
Sums of Squares and Sums of Prod.	S(VY)		0.024589	0.094900	-13.345
	$S(Vx_1)$			6.0000	0.000
	$S(Vx_2)$				15,000
r^2's and r's	$_{VY}\ r^2$		1.0000	0.0610	0.9124
	r		1.0000	0.2470	-0.9552
	$_{Vx_1}\ r^2$			1.0000	0.0000
	r			1.0000	0.0000
	$_{Vx_2}\ r^2$				1.0000
	r				1.0000

Regression Equation:

$$\hat{Y} = 0.5561 + 0.0158\ (X_1 - 1.301) - 0.0012\ (x_2 - 90)$$

for which

$$R^2 = 0.0610 + 0.9124 = 0.9734$$

Table 17

Multiple Linear Regression Analysis and Regression Equation
for
Four-Inch Thick Pavement Cores

		Maximum Compressive Strength, Pounds	Y log log Strength	x_1 log 1000xRate of Deformation	x_2 Temperature °F
	1	15950	0.6235	2.301	40
	2	4115	0.5580	2.301	90
	3	1415	0.4984	2.301	140
	4	9390	0.5990	1.301	40
	6	660	0.4502	1.301	140
	7	9100	0.5976	0.301	40
	8	2400	0.5289	0.301	90
	n = 9	600	0.4438	0.301	140
Sums	ΣV	46335	4.8352	11.709	810
Means	$\bar{V}$	5148	0.5372	1.301	90
	ΣVY		2.630901	6.400195	413.7830
	ΣVx_1			21.233409	1053.810
	ΣVx_2				87,900
	$\frac{\Sigma U \Sigma V}{n}$		2.597634	6.290595	435.168
				15.233409	1053.810
					72,900
Sums of Squares and Sums of Prod.	$S(VY)$		0.033217	0.109600	-21.385
	$S(Vx_1)$			6.0000	0.0000
	$S(Vx_2)$				15,000
r^2's and r's	$_{VY}r^2$		1.0000	0.0627	0.9179
	r		1.0000	0.2504	-0.9581
	$_{Vx_1}r^2$			1.0000	0.0000
	r			1.0000	0.0000
	$_{Vx_2}r^2$				1.0000
	r				1.0000

Regression Equation

$$\hat{Y} = 0.5372 + 0.0136\,(x_1 - 1.301) - 0.0014\,(x_2 - 90)$$

for which

$$R^2 = 0.0627 + 0.9179 = 0.9806$$

These results, coupled with the results of the similar tests on laboratory-compacted specimens, and with the results obtained by Wood (67), appear to be quite significant. It is believed that the form of relationship between the strength of a bituminous mixture and the variables of temperature and rate of deformation has been established over a sufficiently wide range of test types to give a large measure of confidence to its use.

Repeated Load Test

If a repeated load test is to be of value for the purpose of design or for the evaluation of the general suitability of a particular resurfacing mixture, it should provide information concerning the plastic nature of the mixture and a measure of the "endurance limit" as Wood called it (67). The first consideration given to the results of both the slow- and rapid-cycle repeated load tests was to determine the nature of the relationship between cumulative permanent deformation and number of load repetitions. The first approximation to this relationship was to test that proposed by Wood (67). He found that for his results, ". . . (a) plot of permanent deformation versus number of load applications starts out as a straight line on a semi-logarithmic plot in all cases. At some stage, dependent upon the applied stress and number of load applications, the plot deviates sharply from the straight line. The point at which this deviation from linearity occurred was selected as the point of failure and was taken as the failure criterion." This method of plotting implies that the underlying relationship between cumulative permanent deformation and number of load applications is of the form:

$$k^y = ax \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \quad 5$$

where y = cumulative permanent deformation

x = number of load applications

k and a = constants

Taking the logarithm of each side of this equation, it may be written in the form:

$$y = A + B \log x \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \quad 6$$

in which A and B are new constants, both of which involve k.

An index to the plasticity of a given mixture would appear to be the slope of the straight line relating cumulative permanent deformation and number of load repetitions or the value of B in equation 6. The value of B is actually $1/\log k$. Thus, it may be said that equation 5 contains the elements needed to provide information concerning the plastic nature of a bituminous mixture. Furthermore, if the equation truly represents the relationship within the entire region before failure, then it is satisfactory for analyzing the results of repeated load tests.

For the type of test, test specimens, and test conditions used by Wood, an analysis of his data by equation 6 appeared to be justified. This was also the first approach used in the present study. However, it soon became evident that the relationship shown in equation 6 did not apply for the test conditions of the present study. The results from several repeated load tests of both the slow- and rapid-cycle types were plotted on semi-log paper; these plots can be seen in Figures 15, 19, 22, and 25. All of these plots show that the data points deviate from the straight line established by the first several cycles. This deviation takes place at a relatively small number of cycles in most cases. In addition, the deviation occurs in some cases at rather low values of cumulative permanent deformation (less than 0.01 inches for the case shown in Figure 19). From observation of the test specimens, a serious question can be raised as to whether this actually constitutes "failure." If it is not failure, then the alternative appears to be that for the types and sizes of specimens tested in this study, a relationship other than that expressed in equation 5 is operating. The whole question hinges on the definition of failure and from this author's viewpoint a specimen that has accumulated deformation of 0.01 inches and shows no cracking or bulging has not "failed."

Observation of the shortcomings of equation 5 to explain the relationship with respect to a failure criterion led to the investigation of an equation of the form:

$$y = kx^{\frac{1}{n}} \quad . \ . \ . \ . \ . \ . \ . \ . \ . \ 7$$

where y = cumulative permanent deformation,

x = number of load applications,

k and n = constants,

for this purpose.

Taking the logarithm of each side of equation 7, it may be rewritten:

$$\log y = k' + \frac{1}{n} \log x \quad . \ . \ . \ . \ . \ . \ . \ . \ . \ 8$$

where k' is a new constant.

This equation is a straight line on a log-log plot and has an intercept k' and a slope $\frac{1}{n}$. This slope again is suggested as an index of plasticity.

The results of all of the repeated load tests performed in this study were plotted to a log-log scale and are shown in Figures 16 and 17 for the slow-cycle tests and in Figures 18, 20, 21, 23, 24, 26, and 27 for the rapid-cycle tests. Deviation from the straight lines is suggested as a criterion for failure based on observations of the specimens.

The slopes of the straight lines derived from the log-log plots of the rapid-cycle test results were computed and these are shown in Table 18. There is a considerable amount of variation in the slopes and one may not say from these data that the slopes produced by tests on 4-inch cores are any different from those produced by tests on 3-inch or 2-inch cores. The plastic nature of the bituminous concrete binder and surface mixture of which these cores were composed may be about the same for the combinations of layer thicknesses tested. In fact this is what these results show, but the number of tests is small and further studies may be needed. In any case, it is suggested that the rapid-cycle test developed in this study has possibilities for providing an index to the plastic nature of bituminous concrete when subjected to repeated applications of load.

Returning to the concept of a failure criterion, one of the shortcomings of the repeated load tests is that all tests were not carried out to failure. However, some observations of a general nature may be made even though a definite relationship between number of cycles to reach failure and the variables of core thickness, temperature, and applied stress cannot be stated.

The results, while limited and at one point somewhat erratic,

Table 18

Slope of Linear Portion of Curve Relating Log Cumulative Permanent Deformation to Log Number of Load Repetitions Rapid-Cycle Repeated Load Tests

Test Temperature °F	Specimen Thickness Inches	Applied Stress psi	Slope
80	2	100	.32
"	4	100	.32
"	2	150	.28
"	4	150	.34
"	2	200	.23
"	4	200	.32
140	2	100	.24
"	4	100	.28
"	2	150	.27
"	4	150	.30
"	2	200	.28
"	4	200	.34
80	3	100	.36
"	"	150	-
"	"	200	.37
140	"	100	.45
"	"	150	.27
"	"	200	.35

indicate that if the concept of an endurance limit for a bituminous-concrete mixture was thought to be an important one, the rapid-cycle test and methods of measurement used in this study could be used to measure this property. It would seem to this author that a test in which the applied stress was of the order of that expected in service would be more realistic than one in which some percentage of the ultimate strength of the mixture, for a given set of test conditions, was employed.

It was estimated from the graph of the load cycle that was produced by the Brush oscillograph that the rate of deformation that was obtained in the rapid-cycle tests was of the order of magnitude of 0.4 inches per minute. One may use the regression equations shown in Tables 15-17 to estimate the ultimate strength of the cores if they were tested at 0.4 inches per minute at temperatures of 80 and 140° F. From these calculated values one can estimate what percentage of the ultimate stress for this type of test was being applied when using contact pressures of 100, 150, and 200 psi in the repeated load tests. This is shown in Tables 19 and 20. It can be seen that some of the values listed in Table 20 exceed 100 percent. Evidently there is a fundamental difference between the ultimate strength of a specimen tested at a constant rate of deformation and the ultimate strength of a specimen subjected to an impact load such as was applied in the rapid-cycle repeated load tests.

In summary, the rapid-cycle repeated load test has possibilities of use in bituminous mixture evaluation. Much more work needs to be done with this test and a possible extention of the work performed in this investigation is outlined in the section on SUGGESTION FOR FURTHER RESEARCH. It is believed that the test and the method of analysis for

Table 19

Estimated Strengths of Two-, Three- and Four-Inch Pavement Cores for a Deformation Rate of 0.4 Inches per Minute and Temperatures of 80 and 140° F

Cores Height Inches	Ultimate Strength-psi	
	80° F	140° F
2	758	290
3	600	154
4	460	100

Table 20

Estimated Percent Ultimate Strength of Pavement Cores Tested at a Rate of Deformation of 0.4 Inches Per Minute

Core Thickness Inches	Applied Stress psi	Percent Ultimate Stength	
		80° F	140° F
2	100	13	35
"	150	20	52
"	200	26	69
3	100	17	65
"	150	25	97
"	200	33	130
4	100	22	100
"	150	33	150
"	200	43	200

the results that has been outlined contain the element necessary to evaluate the plastic and load-carrying characteristics of a bituminous-concrete mixture.

SUMMARY

The following is a brief recapitulation of the major findings of the study:

1. The results of triaxial tests made at approximately 80° F and at a constant rate of deformation of 0.02 inches per minute on specimens of bituminous concrete molded to unit weights of 140 and 146 pounds per cubic foot showed that the Mohr rupture envelope was linear for a range of confining pressure between 15 and 120 psi.

2. Compression tests on specimens of bituminous concrete two inches thick and of various sizes loaded over an area of 12.57 square inches (four-inch diameter plate) showed that large increases in the load-carrying capacity of the bituminous concrete were obtained when the ratio of the specimen area to the loaded area was increased from 1.0 to 5.1. Increase in this ratio from 5.1 to 9.7 brought about an additional increase in the load-carrying capacity of the two-inch thick specimens but less so than that observed between the results from tests in which the ratios were 1.0 and 5.1.

Finally, an increase in the ratio of specimen area to loaded area from 9.7 to 20.4 produced only a slight increase in the load-carrying capacity of the two-inch thick slabs. It was indicated from a plot of stress at failure versus the aforementioned ratio that further increase in the ratio would produce no significant increase in the load-carrying capacity of the slab. That is, that the failure stress found for the 16 by 16 inch specimen (ratio = 20.4) was relatively uninfluenced by the area of the specimen.

3. The average total stress at failure for specimens of bitu-

minous concrete two-inches thick and 16 by 16 inches in area, when loaded over 12.57 square inches in the center of the specimen at a temperature of about 80° F and a rate of deformation of 0.02 inches per minute was 610 psi for specimens having a density of 140 pounds per cubic foot and 780 psi for specimens having a density of 146 pounds per cubic foot.

4. The confining pressures that would be needed in a conventional triaxial test performed at 80° F and 0.02 in. per minute to produce the stress values of 610 and 780 psi determined as the load-carrying capacity of the bituminous concrete mixtures in a two-inch layer were estimated to be 158 psi for the mixture compacted to a density of 140 pounds per cubic foot and 196 psi for the mixture compacted to a density of 146 pounds per cubic foot. These values of confining pressure are considerably in excess of the unconfined compressive strengths of the mixtures indicated by the curves in Figure 23 for the same temperature and rate of deformation, the latter values being 90 and 125 psi respectively.

5. The deformation characterists of laboratory-compacted specimens of bituminous concrete made by vibrational compaction appeared to be different from the deformation characteristics of similar specimens made by double-plunger compaction. The average total deformation for two-inch thick, four-inch diameter specimens compacted by vibration and loaded to failure at 80° F at 0.02 inches per minute was 0.128 inches. The average total deformation at failure for similar specimens compacted by double-plunger compaction and loaded in the same way was 0.074 inches.

In this regard, Marshall test results from vibrated specimens produced an average flow value of 18.2 while results from tests on specimens made by double-plunger compaction and Marshall compaction produced average flow values of 10.8 and 11.7 respectively.

6. A relationship between the strength of a bituminous-concrete mixture and the variables of temperature and rate of deformation was determined in this study both for laboratory-compacted specimens of bituminous concrete two-inches thick and for pavement cores two, -three, and four-inches thick. The relationship proposed by Wood (67) was found also to fit the results of the tests performed in this study.

The following summarizes the equations derived by multiple linear regression:

(a) Laboratory specimens compacted by vibration

$$\hat{Y} = 0.6209 + 0.0182\ X_1 - 0.0012\ x_2$$

(b) Laboratory specimens compacted by double-plunger

$$\hat{Y} = 0.6223 + 0.0282\ X_1 - 0.0013\ x_2$$

(c) Two-inch thick pavement cores

$$\hat{Y} = 0.6194 + 0.0171\ X_1 - 0.0008\ x_2$$

(d) Three-inch thick pavement cores

$$\hat{Y} = 0.6435 + 0.0158\ X_1 - 0.0012\ x_2$$

(e) Four-inch thick pavement cores

$$\hat{Y} = 0.6398 + 0.0173\ X_1 - 0.0014x_2$$

where for all equations

$\hat{Y} =$ log log maximum compressive strength in pounds

$X_1 =$ log 1000 x rate of deformation in inches per minute

$x_2 =$ temperature in degrees F.

The squared multiple correlation coefficients (R^2) determined from the regression analyses were found to be 0.99, 0.99, 0.98, 0.97, and 0.98 for cases (a) through (e) respectively. It is indicated that an equation of the form

$$x_o = A^{Bx_1^{(Cx_2 + D)}}$$

where x_o = maximum compressive strength

x_1 = rate of deformation

x_2 = temperature, and

A, B, C, D = constants

explains the relationship between the strength of a bituminous mixture and the variables of temperature and rate of deformation for a variety of conditions.

7. The results of the repeated-load tests indicated that for the specimens of bituminous concrete tested the relationship between cumulative permanent deformation and number of load repetitions was of the form:

$$y = kx^{\frac{1}{n}}$$

where y = cumulative permanent deformation

x = number of load repetitions, and

k and n = constants

8. A criterion for failure of a specimen of bituminous concrete tested by repeated applications of load was developed for the rapid-cycle repeated load test. For these tests, plots of log cumulative permanent deformation versus log number of load applications was linear for a number of cycles. For many specimens, at some number of cycles .

depending upon the temperature and applied stress, the plot deviated upward from the straight line. This point of deviation is considered to denote the number of cycles required for specimen failure.

9. The slopes of the linear portions of the curves of log cumulative permanent deformation versus log number of load applications were not significantly different for any of the three thicknesses of bituminous concrete cores tested by the rapid-cycle method. The average value for slope was found to be 0.31.

CONCLUSIONS

Based on the results of this study, the following conclusions seem justified. Since this was a laboratory study restricted in scope with respect to materials and methods, the conclusions can logically be applied only to those materials, mixtures and methods actually used.

1. The load-carrying capacity of a bituminous concrete mixture, compacted into a relatively thin layer and loaded at a constant rate of deformation over a small portion of the total surface with a plate which is relatively large with respect to the layer thickness, appears to be several times greater than the unconfined compressive strength of the mixture for the same temperature and rate of deformation.

2. The confining pressure that would be needed in a conventional triaxial test of bituminous concrete to produce a compressive strength equal to that for the mixture when tested in a relatively thin layer and loaded over an area that is large with respect to layer thickness appears to be several times the unconfined compressive strength of the mixture.

3. The relationship between the strength of bituminous concrete and the variables of temperature and rate of deformation is of the form

$$x_0 = A^{Bx_1^{(Cx_2 + D)}}$$

where x_0 = maximum compressive strength

x_1 = rate of deformation

x_2 = temperature and,

A, B, C, D = constants.

4. The relationship between cumulative permanent deformation and number of load repetitions for the rapid-cycle repeated load test results, in the region before failure of the specimen, appears to be of the form

$$y = k\, x^{\frac{1}{n}}$$

where y = cumulative permanent deformation

x = number of load repetitions, and

k and n = constants.

In addition, the rapid-cycle repeated load test appears to be a promising method for the evaluation of the plastic nature and endurance properties of bituminous-aggregate mixtures.

SUGGESTIONS FOR FURTHER RESEARCH

It is suggested that the rapid-cycle repeated load test be studied further to see if it could be developed into a test method usable for mixture design. Several variables could be studied using specimens made in the laboratory, but first a series of tests should be performed to discover whether compaction by vibration or double-plunger compaction would best reproduce the structure obtained by rolling. This may be done by going to a resurfacing job in the field and doing the following:

1. Cut cores from the compacted binder.

2. Obtain mix samples of the binder from the plant.

3. Make samples in the laboratory to the same density as the pavement cores by the two methods of compaction.

4. Obtain and compare rapid-cycle repeated load test results for all three sets of samples.

When the better method of compaction was selected one could investigate whether the value $\frac{1}{n}$ was an index of plasticity by testing specimens of various mix compositions. At the same time, one could investigate the criterion for failure developed in this study and perhaps use the concepts of plasticity and endurance limit as aids in mixture design.

Finally, since this study brings out the fact that the size of the specimen compared to the loaded area has a marked effect on load-carrying capacity, this variable should also be investigated.

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APPENDIX A

Table 21

Results of Repeated Load Tests on Pavement Cores

Slow Cycle

Rate of Deformation: 0.02 in. per min., Temperature: Approximately 80° F

Number of Cycles	Cumulative Permanent Deformation Thousandths of an Inch			
	4-in. thick Loaded to 75% Ult.or 2710 lbs (216 psi)	4-in. thick Loaded to 75% Ult.or 2710 lbs (216 psi)	4-in. thick Loaded to 50% Ult.or 1805 lbs (144 psi)	4-in. thick Loaded to 50% Ult.or 1805 lbs (144 psi)
1	16	32	14	27
2	24	40	17	31
3	28	44	19	34
4	32			
5		51	21	
6	37			
7				41
8	40			
10	44	60	26	44
13	50			
15	56	67	28	48
17				
20		72	30	51
25				54
30			34	
35				56
40			37	
50				61

Number of Cycles	3-in. thick Loaded to 75% Ult.or 3750 lbs (300 psi)	3-in. thick Loaded to 50% Ult.or 2500 lbs (200 psi)	3-in. thick Loaded to 25% Ult.or 1250 lbs (100 psi)	2-in. thick Loaded to 25% Ult.or 1510 lbs (120 psi)
1	57	54	19	10
2				12
3	75	65		13
5	86	71	28	15
10	101	78	33	17
15	113	83	39	19
20	120	87	41	21
30	135	91	44	23
40	148			25
50	160		49	

Table 22

Results of Repeated Load Tests on Pavement Cores

Rapid Cycle

Two-Inch Thick Cores - 80° F

Number of Cycles	Cumulative Permanent Deformation Thousandths of an Inch — Applied Pressure - psi 100	150	200
1	1/2	2	7
2	1	3	8
4		4	9
5	2		
10		5	10
15		6	11
20			12
25	3		
30		7	
40			14
50	4	8	16
75		9	17
100	5		
125			19
150	6	10	21
200		12	22
250	7		
300		13	24
400	8	14	26
500	9		28
600		15	
700			30
800		16	
900			32
1000	11	17	
1250	12		
1500	13	18	37
2000	14	19	39
3000	16		

Table 23

Results of Repeated Load Tests on Pavement Cores

Rapid Cycle

Two-Inch Thick Cores - 140° F

Number of Cycles	Cumulative Permanent Deformation Thousandths of an Inch		
	Applied Pressure - psi		
	100	150	200
1	14	16	21
2	17	20	26
3	19		29
4	20	24	31
5	21		33
6	22		35
7		28	37
8	25		38
9			39
10	24	30	40
15	27		45
19		35	
20	28		48
25	30		51
30	32		53
35		43	
40			58
50	35	46	61
70		51	
75	39		70
100	42	56	77
150	48	64	84
200	52		96
250			108
300	60	79	123
400		88	148
418	67		
500		97	
510			177
600			211
650			233
750	92	117	
1000	108		
1500	139	208	
1750	156	265	
2000	175	347	

Table 24

Results of Repeated Load Tests on Pavement Cores

Rapid Cycle

Three-Inch Thick Cores - 80° F

Number of Cycles	Cumulative Permanent Deformation Thousandths of an Inch — Applied Pressure - psi 100	150	200
1	4	1	6
2		2	
5	8	4	11
10	9	5	14
20	12	7	19
30	14	9	22
40		10	
50	17	14	27
75			32
100	22	18	35
200	27	21	46
300		26	
400	36	27	58
600			65
700	42	33	
1000	46	36	
1500		42	
2000	53	48	95
2300		53	

Table 25

Results of Repeated Load Tests on Pavement Cores

Rapid Cycle

Three-Inch Thick Cores - 140° F

Number of Cycles	Cumulative Permanent Deformation Thousandths of an Inch		
	Applied Pressure - psi		
	100	150	200
1	6	29	38
2		34	48
5	12	45	66
10	16	54	86
15		60	
16			100
20	22	64	108
30	27	74	127
40			146
50	34		
60		91	198
70			258
100	48	115	
150		148	
200	72	199	
227		312	
300	101		
400	132		
500	200		
535	305		

Table 26

Results of Repeated Load Tests on Pavement Cores

Rapid Cycle

Four-Inch Thick Cores - 80° F

Number of Cycles	Cumulative Permanent Deformation Thousandths of an Inch		
	Applied Pressure - psi		
	100	150	200
1	2	3	12
2	3	4	15
3	4		
4		5	
5	5	6	20
10	6	7	25
15	7	8	28
20		9	31
25	8	10	
30			36
40	9	12	40
50	10	13	
60			44
75	11	16	
80			49
100	12	18	
115			56
150	14	20	
165			62
200	16	22	66
300	17	24	
400	19	25	
500	20	27	87
600		28	
700		29	
715	21		97
900	23	31	
1000		32	
1300	24		120
1500	25		
1750		38	
2000	26		143
2515			160
3000		45	180
5360			276

Table 27

Results of Repeated Load Tests on Pavement Cores

Rapid Cycle

Four-Inch Thick Cores - 140° F

Number of Cycles	Cumulative Permanent Deformation Thousandths of an Inch — Applied Pressure - psi		
	100	150	200
1	26	34	34
2			53
3	35	46	
5	40	53	73
6			78
7		60	
8			89
10	49	68	
12			105
15	54	77	114
20		86	134
23			147
26		94	159
29			174
30	66	102	
32			191
35			211
38			236
40		117	
50	78	136	
60		154	
70		176	
75	92		
80		205	
90		259	
100	110		
125			
150	138		
200	182		
250	260		
270	361		