LOAD DEFORMATION CHARACTERISTICS OF BITUMINOUS MIXTURES UNDER VARIOUS CONDITIONS OF LOADING

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by
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BITUMINOUS MIXTURES UNDER VARIOUS CONDITIONS
OF LOADING

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

July 24, 1957

FROM: H. L. Michael, Assistant Director

File: 2-4
Project No. C-36-6

Attached is a paper entitled, "Load Deformation Characteristics of Bituminous Mixtures Under Various Conditions of Loading," by Professors W. H. Goetz, J. F. McLaughlin and L. E. Wood, research engineers on our staff. This paper was presented to the Association of Asphalt Paving Technologists in Atlanta in Georgia in February, 1957.

This paper summarizes much of the information that has been obtained on the study of bituminous concrete in the Joint Highway Research Project Laboratories in recent years. The load characteristics of bituminous mixtures are discussed and laboratory applications to the problem are presented. Relationships have been discovered that should provide a basis for a better understanding of the properties of bituminous concrete.

Respectfully submitted,

Harold L Michael

Harold L. Michael, Assistant Director
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Technical Paper

LOAD DEFORMATION CHARACTERISTICS OF BITUMINOUS MIXTURES UNDER VARIOUS CONDITIONS OF LOADING

by

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Joint Highway Research Project
Project C-36-6
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July 24, 1957

LOAD-DEFORMATION CHARACTERISTICS OF BITUMINOUS MIXTURES UNDER VARIOUS CONDITIONS OF LOADING

INTRODUCTION

The bituminous concrete used by the State of Indiana has been included in their specifications without essential change as to type, at least since 1934. Until about 1948, this material was used primarily as the surfacing layers of high-type flexible pavements. In this application, the performance of the material was considered entirely satisfactory and instances of stability failure were rare.

However, since 1948 this bituminous mixture type has been used more and more for the resurfacing of deteriorated portland-cement concrete pavements. Also, in the period following the war, and particularly in recent years, the number and weight of heavy vehicles using Indiana highways has increased markedly. The result has been that the bituminous mixture when used as an overlay for portland-cement concrete has shown lack of stability in some instances, principally as rutting in the wheel-track areas. This deficiency, although existent, was considered to be of rather minor extent previous to about 1951, but since that time it has become a more serious problem. This rutting, first investigated in a systematic way in the field in 1953, was found to be present to some extent in many of the overlays inspected. However, it was present to an extent that definitely could be judged objectionable in relatively few areas, each of which was subjected to much heavy, relatively slow-moving traffic. The condition was found to be most severe at

FIGURE 1   OVERLAY PERFORMANCE AT INTERSECTION

FIGURE 2 RUTTING IN INSIDE WHEEL-TRACK

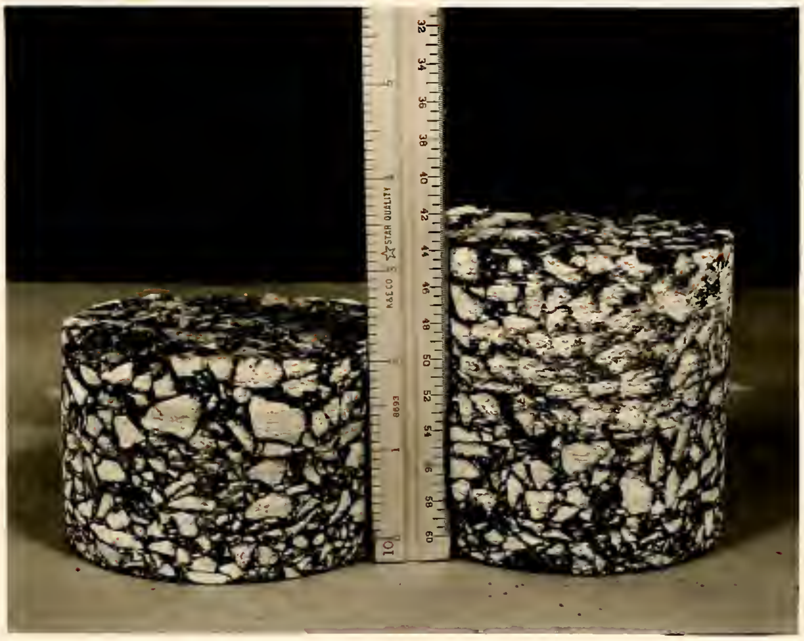

FIGURE 3 HEIGHT DIFFERENCES IN CORES

FIGURE 4 TRENCH CUT IN BITUMINOUS OVERLAY

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. This trench was cut approximately 750 feet back from the nearest intersection. Another trench cut at the intersection showed more distortion than the case illustrated.

From this trenching operation, several pertinent facts were ascertained in addition to the magnitude of rutting. One of these was to find that the overlay had spread out in a transverse direction so that it extended some six inches beyond the edge of the concrete to which it was placed in construction. Secondly, it was found from an examination of the sawed face that both the binder and surface layers were distorted in the wheel-track areas. Thirdly, it was found from density measurements made on samples taken at closely spaced intervals across the traffic lane that the density of the mixture was essentially the same regardless of sample position. All of these facts led to the conclusion that the failures under consideration were caused by plastic flow.

It is pertinent to note also that the density measurements made on samples taken from the trench indicated that the mixture voids had been reduced to values in the neighborhood of two percent. From this it is deduced that the service stability of the mixture could be increased by decreasing asphalt content. From the practical point of view, this is true and it has been done. However, the fact remains that stability tests and design methods now in common use indicate that this mixture <u>should</u> be stable and the adequacy of our present design methods, at least for the overlay condition, is open to question.

The bituminous-concrete overlay that has been used in Indiana is composed of two layers: (1) a binder or leveling course which has a maximum aggregate size of 3/4 to one inch, 65 percent coarse aggregate (material retained on the No. 6 sieve), 35 percent fine aggregate containing little or no material passing the No. 200 sieve, and which usually contains 4.5 to 5.5 percent asphalt by weight of the mixture, and (2) a surface course which has a maximum aggregate size of 1/2 inch and contains about 50 percent coarse aggregate, 50 percent fine aggregate and with about three percent of the total aggregate passing the No. 200 sieve. It 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 composed of 1-1/2 inches of binder and one inch of surface is not uncommon. The asphalt cement used is a 60-70 penetration grade.

This bituminous concrete is based upon a design which avoids the strictly dense-graded mixture concept in favor of one which provides appreciable aggregate voids to accommodate increased percentages of asphalt. It is recognized that there are those who would suggest that the problem outlined could be solved readily by employing a more densely-graded mixture at reduced asphalt content. This is probably true, and it may well be that Indiana will be forced to abandon its present concept of design in order to deal with the present severe service condition, and the even more severe conditions which we can expect in the future. It is suggested, however, that this possibility does not alleviate the basic problem that requires asphalt paving technologists to design mixtures for conditions for which our present methods of design have been shown to be inadequate.

In approaching this problem in the laboratory, two studies were undertaken. The first of these was of a fundamental character using sheet-asphalt and bituminous-concrete specimens of rational size and tested by rational procedures (32).[1] The second phase, conducted to relate these data to field conditions, was performed on bituminous-concrete specimens of irrational size molded in the laboratory, cored from laboratory-molded slab specimens, and cored from pavements (11).

The grading curves for the sheet-asphalt and bituminous concrete mixtures used in a laboratory study of this problem are shown in Figure 5. The sheet asphalt utilized a blend of local natural sand and a limestone dust. The bituminous concrete was composed of crushed limestone coarse aggregate, natural sand fine aggregate and a small amount of limestone filler. A 60-70 penetration grade asphalt cement secured from the Texas Company at Port Natchez was used throughout the study.

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[1] Numbers in parenthesis refer to the Bibliography.

GRADATION CURVES FOR

MIXTURES USED IN THE STUDY

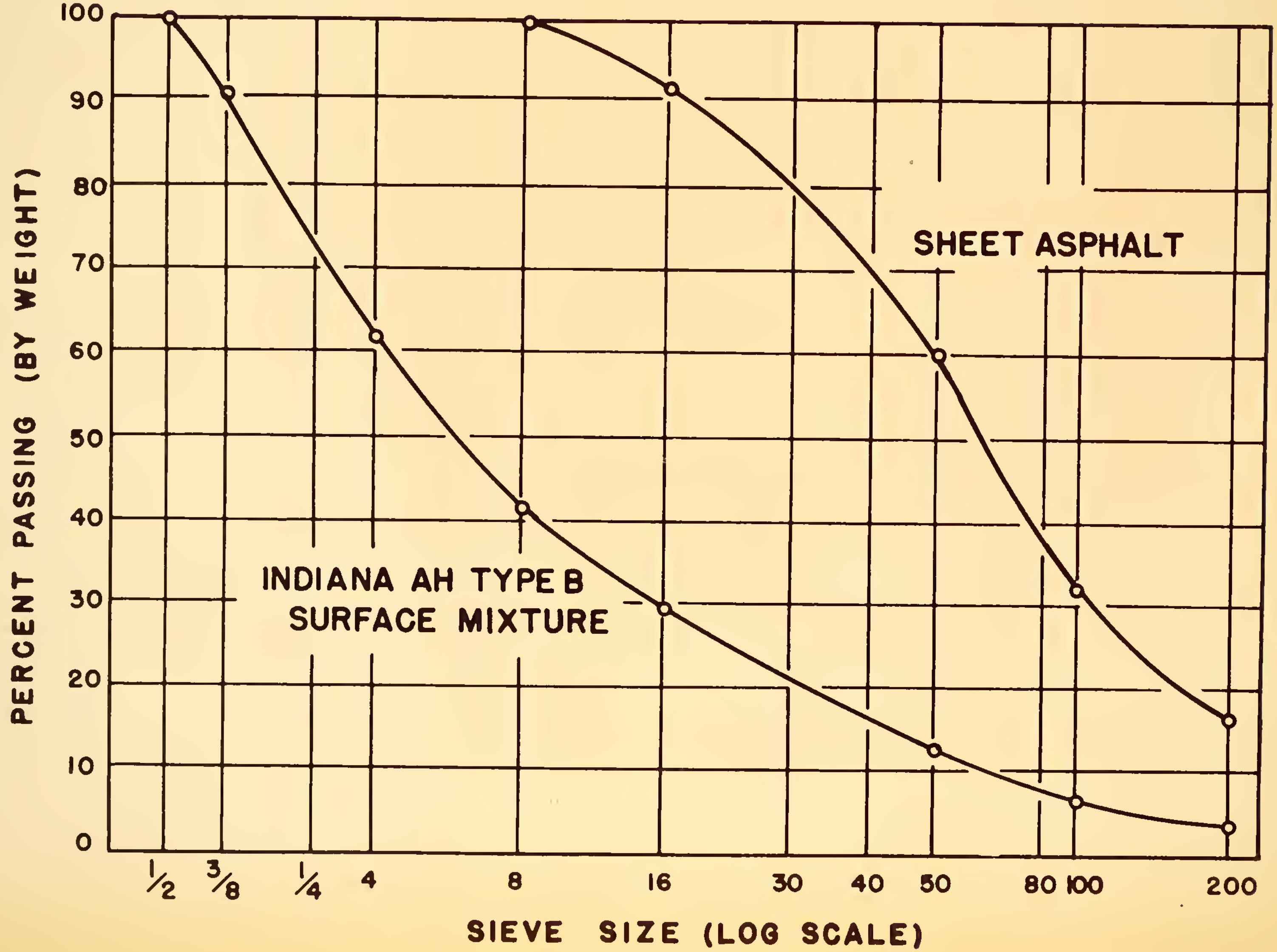


FIGURE 5

ESTABLISHMENT OF A RELATIONSHIP AMONG TEMPERATURE, RATE OF DEFORMATION, AND COMPRESSIVE STRESS

In the strength evaluation of bituminous-aggregate mixtures by present-day design methods, there appears to be a general lack of knowledge regarding the fundamental relationship involved among the variables of temperature, rate of deformation and strength. This lack is apparent when one considers the wide variations in rate of deformation and temperature used in the various test procedures.

A relationship among temperature, rate of deformation and compressive stress may be determined by testing a series of specimens to failure at varying rates of deformation and at various temperatures. A mathematical model can be determined that best expresses the desired relationship by using regression analysis.

Unconfined Compression Tests

The first phase of this study was to determine the effect of temperature and rate of deformation upon the maximum unconfined compressive stress for two widely different mixture types. It was decided that these two types could be a sheet-asphalt mixture and a bituminous-concrete mixture. The basic study could be performed using the sheet-asphalt mixture.

Sheet Asphalt Mixture

Mixture I was a sheet-asphalt mixture containing nine percent asphalt. The gradation curve is shown in Figure 5. This mixture was compacted into specimens two inches in diameter and four inches in height by a double-plunger static compaction method which included rodding the material into the mold. To obtain the required density controls the specimens were molded to a predetermined density by compacting a weighed quantity of mixture to a fixed specimen height.

The specimens were tested to failure in the unconfined state at five rates of deformation: 0.002, 0.02, 0.2, 2.0, and 8.65 in/min. At each of these rates, three temperatures were used: 40, 100 and 140° F. These temperatures were maintained during the test by means of a water bath.

The results of these tests are shown in Table 4 in Appendix B and depicted graphically in Figure 6 where the maximum unconfined compressive stress is plotted versus the logarithm of the rate of deformation for the various temperatures.

It was desired to express the family of curves in Figure 6 in a mathematical equation. The general form appeared to be:

$$x_0 = A^{Bx_1^{(Cx_2 + D)}} \qquad \text{.................(1)}$$

By taking the log of the equation twice and grouping some constants, one obtains:

$$\log \log x_0 = A' + B' \log x_1 + C'x_2 + D'x_2 \log x_1 \text{...(2)}$$

where x_0 = maximum unconfined compressive stress, psi

x_1 = rate of deformation, in./min.

x_2 = temperature, °F

A, B, C, D, A', B', C', D' = constants.

RELATIONSHIP BETWEEN RATE OF LOADING AND MAXIMUM COMPRESSIVE STRESS AT VARIOUS TEMPERATURES

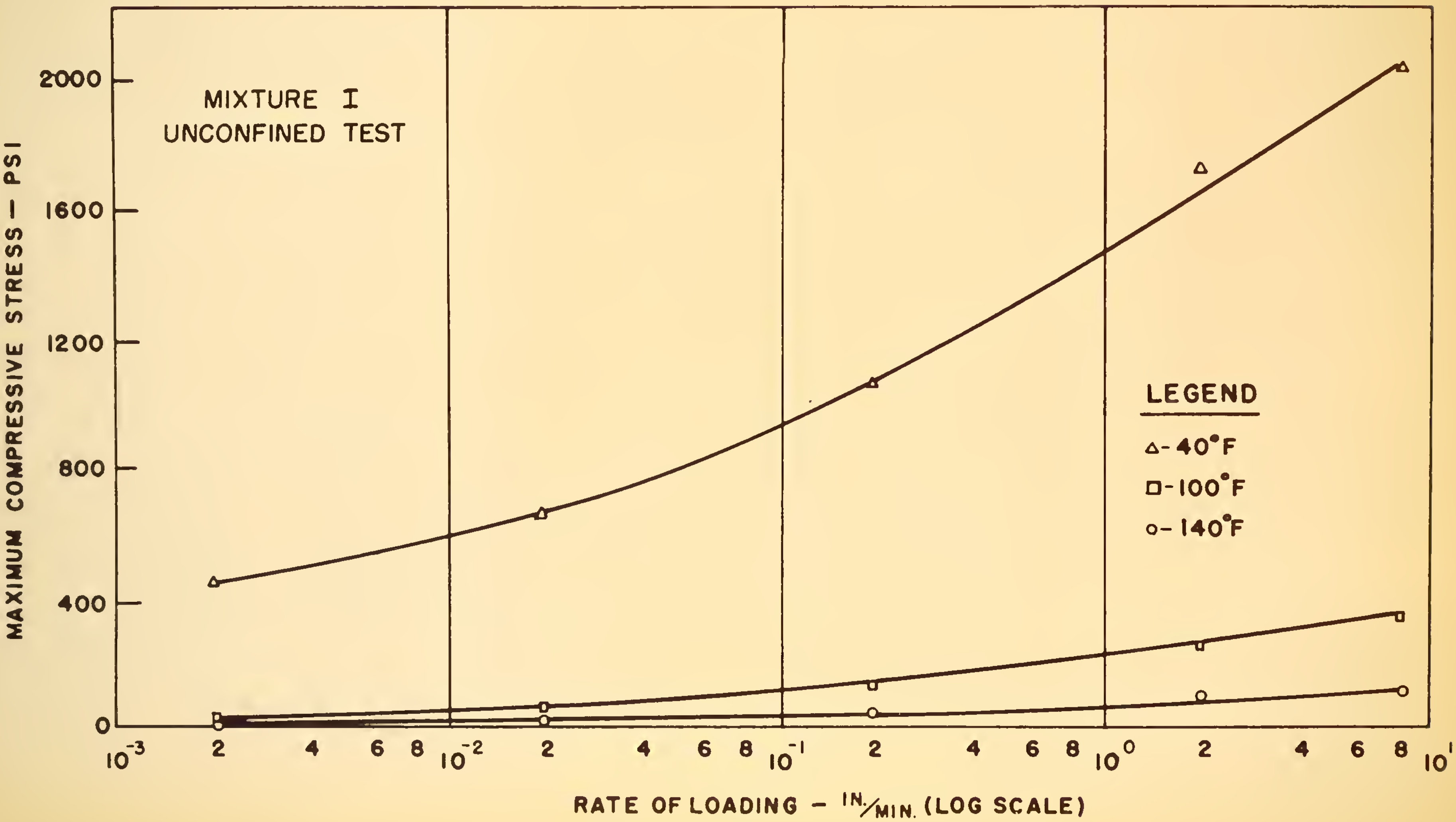


FIGURE 6

Now letting $X_0 = \log \log x_0$

$X_1 = B' \log x_1$

$X_2 = C' x_2$

$X_3 = D' x_2 \log x_1$

and substituting these values in equation 2, one gets:

$$X_0 = C_0 + C_1 X_1 + C_2 X_2 + C_3 X_3 \quad \ldots\ldots\ldots\ldots\ldots (3)$$

where C_0, C_1, C_2, and C_3 are constants.

The problem then becomes to evaluate C_0, C_1, C_2, and C_3 such that one gets the minimum sum of squares of the deviations about the mathematical model. For a good discussion on least squares analysis see reference (18) of the Bibliography.

To simplify calculations, the raw data were coded and transformed so that:

$X_0 = 10{,}000 \log \log x_0$

$X_1 = 1{,}000 \log 1{,}000\, x_1$

$X_2 = x_2/20$

$X_3 = 50\, x_2 \log 1{,}000\, x_1$.

Upon completion of the analysis, the raw data equation (coded and transformed) was:

$$X_0 = 5876.33 - 842.69 X_2 + 0.0224 X_1 + 0.1195 X_3, \ldots\ldots (4)$$

which is the multiple regression equation. For the complete analysis of the data, the reader is referred to Appendix A.

(R^2) is defined as the proportion of the sum of squares of the dependent variable which is explained by the multiple regression equation. (R^2) computed for the laboratory data used in the above model was 0.997. This means that the above regression equation is a very good representation of the underlying relationship between the variables studied. Using this equation, an excellent estimation of the unconfined, maximum compressive stress for many rates of deformation and temperatures could be determined for Mixture I.

It must be remembered that the regression equation estimates the average value of x_0, the maximum unconfined compressive stress. For example, one would wish to know what average unconfined compressive strength a series of bituminous mixture specimens would have when tested at a fixed set of conditions. By use of statistical procedures one may determine an interval within which he is quite certain the mean value will fall.

The multiple correlation coefficient (R) measures the degree of linear association among all of the variables (X_1, X_2, X_3) with X_0. (R) for the regression equation given was 0.998.

One should be aware of the danger involved in extrapolating beyond the range of observations. Unless there is reasonable certainty that the determined function does exist over a wider range of values for the variables than in the present study, the regression equation should be used only within the limits of its determination. While excellent correlation was indicated over the range of temperature and rate of deformation used in this series, a wider range of values for those variables used in this series might indicate that another type of function would be necessary.

With the excellent correlation established using the given general mathematical model $x_0 = A^{Bx_1^{(Cx_2 + D)}}$, it should be possible with a limited number of tests to establish the relationship among maximum compressive stress, temperature, and rate of deformation for any chosen mix.

Bituminous Concrete Mixture

The original model (equation 1) was derived for results obtained for a sheet-asphalt mixture tested in unconfined compression. This mixture was quite plastic in character. Since it was felt that the original model should be applicable to a wide range of mixtures, specimens were formed from a crushed-stone aggregate (1/2 inch maximum size) following the Indiana AH type B surface course gradation (Figure 5) to test this hypothesis.

This bituminous-concrete mixture was less plastic in character, had a lower asphalt content, and had aggregate particles that were much more angular in shape than the sheet-asphalt mixture. It contained seven percent asphalt and was compacted into specimens three-inches in diameter and six inches in height by double-plunger static compaction which included rodding the material into mold by a procedure similar to that used in forming the sheet-asphalt specimens.

The specimens formed from the Indiana AH type B surface course mixture were tested to failure in the unconfined state at the extreme conditions of temperature (40 and 140° F) and rate of deformation (0.002 and 0.2 in./min.). The results from tests at these conditions were used to establish the parameters of the regression equation. This analysis is shown in Appendix A. The established regression equation was used to predict the maximum unconfined compressive stresses at the other test conditions (0.002, 0.02, and 0.2 in./ min. at 100°F and 0.02 in./min. at 40 and 140°F). Specimens were then tested at the above test conditions. The results of this study are presented in Table 5 in Appendix B. The predicted values and the observed values are both shown in Figure 7. Very good correlation was observed at all levels between the observed values and the predicted values. This would indicate that the general expression is valid for the unconfined condition for both sheet-asphalt and bituminous-concrete mixes.

Confined Compression Test

The original model (equation 1) was derived for results obtained from the unconfined compression test. It was hoped that the original model would have wider applicability than for just the unconfined test condition. Since mixtures are loaded in the field in such a manner that some degree of lateral support is provided, it was decided to check the validity of equation 1 by performing compression tests at various confining pressures. The specimens were molded from the sheet-asphalt mixture in the manner previously described. However, the character of the mineral filler was altered slightly and so this mixture is referred to as Mixture II.

Compression tests were made at two confining pressures: 15 and 30 psi. At each of the confining pressures and at the extreme values of temperature (40 and 140°F) and rate of deformation (0.002 and 0.2 in./min.) specimens were tested to failure. The results from tests at these test conditions again were used to evaluate the parameters of the regression equation. This analysis is shown in Appendix A. The regression equations, once established, were used to predict maximum compressive stresses at the other test conditions (0.002, 0.02, and 0.2 in./ min. at 100° and 0.02 in./min. at 40 and 140°F). Specimens were then tested at the above test conditions at both 15 and 30 psi. The results of this study are shown in Table 6

RELATIONSHIP BETWEEN RATE OF DEFORMATION AND MAXIMUM COMPRESSIVE STRESS AT VARIOUS TEMPERATURES

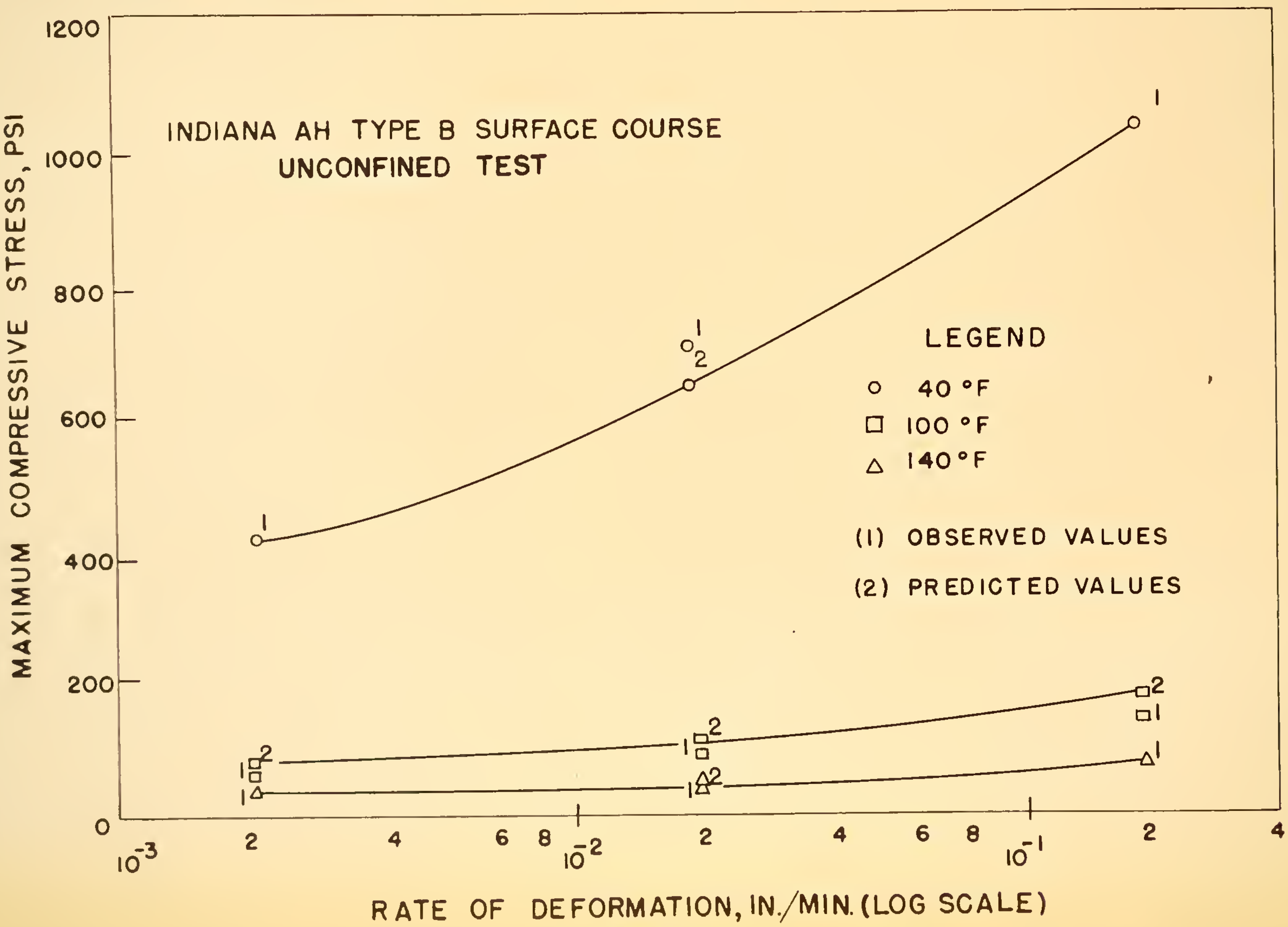


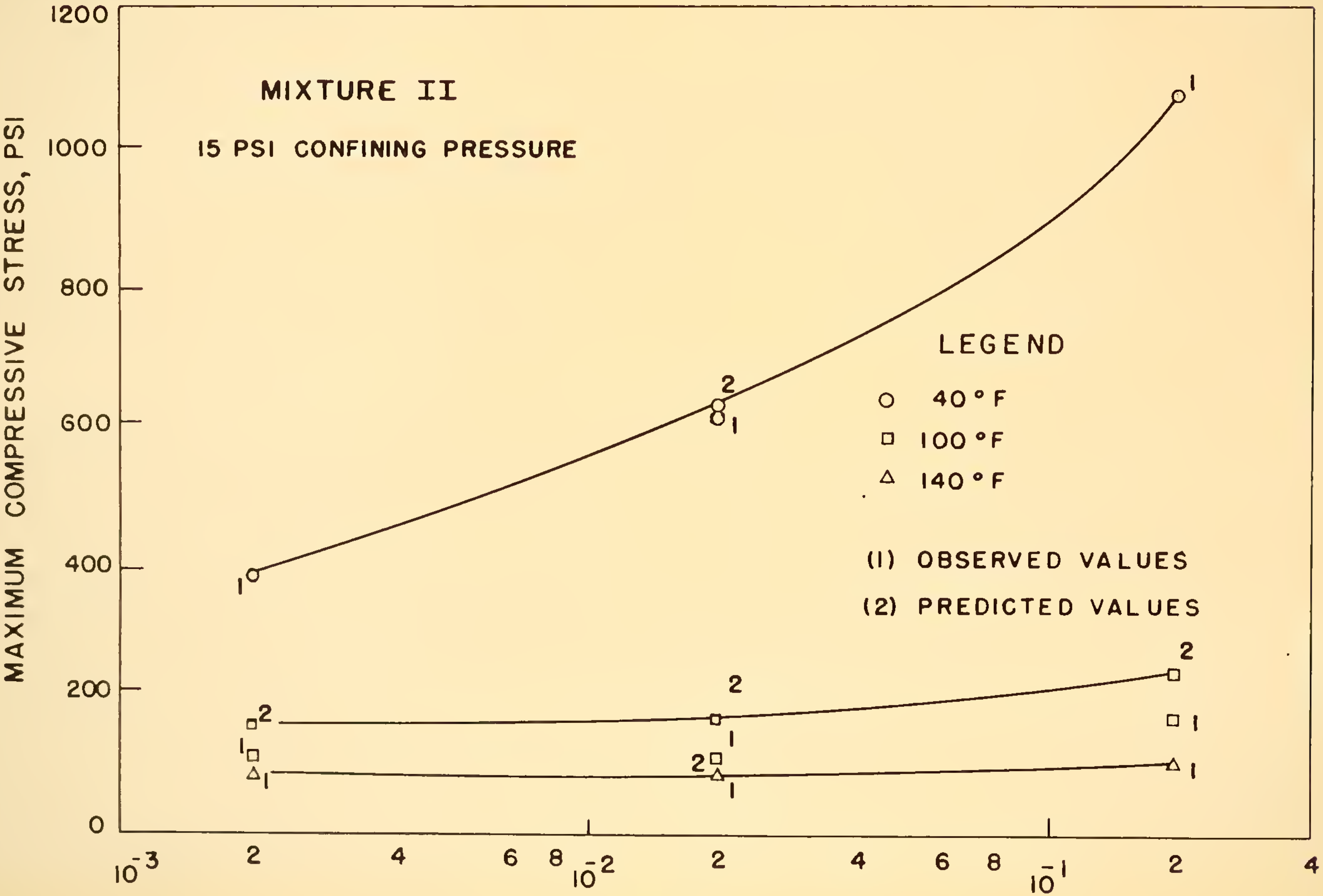
FIGURE 7

in Appendix B. Figure 8 shows both the predicted values and the observed values for a confining pressure of 15 psi. Good correlation between observed and predicted values was noted except in the case of tests made at 100°F temperature. For all three rates of deformation at 100°F the predicted values were higher than the observed values. With the inclusion of the results at 100°F into the regression analysis, a much better fitting regression equation could be found than was obtained using limited data from the extreme conditions of temperature and rate of deformation.

Figure 9 shows the predicted values and the observed values for a confining pressure of 30 psi. Again good correlation between the predicted and observed values was noted except in the case of the 100°F temperature. For all three rates of deformation at 100°F, the predicted values were higher than the observed values. Again, a better fitting equation could be derived if the 100°F data were included in the regression analysis.

From observation of the results of this series of tests, it can be seen that the introduction of lateral support increases the maximum confined compressive strength of the mixture more at the higher temperatures than at lower temperatures. It is at these higher temperatures that the mixture is quite plastic in character. At 40°F the mixture loses most of its plastic nature and it is relatively "stiff." It is obvious that enough change in mixture strength resulted from the introduction of confining pressures that one would no longer be secure in evolving a prediction equation for wide ranges based upon limited results from four test levels. The addition of an interaction term involving the confining pressure and

RELATIONSHIP BETWEEN RATE OF DEFORMATION AND MAXIMUM COMPRESSIVE STRESS AT VARIOUS TEMPERATURES



RATE OF DEFORMATION, IN./MIN. (LOG SCALE)

FIGURE 8

RELATIONSHIP BETWEEN RATE OF DEFORMATION AND MAXIMUM COMPRESSIVE STRESS AT VARIOUS TEMPERATURES

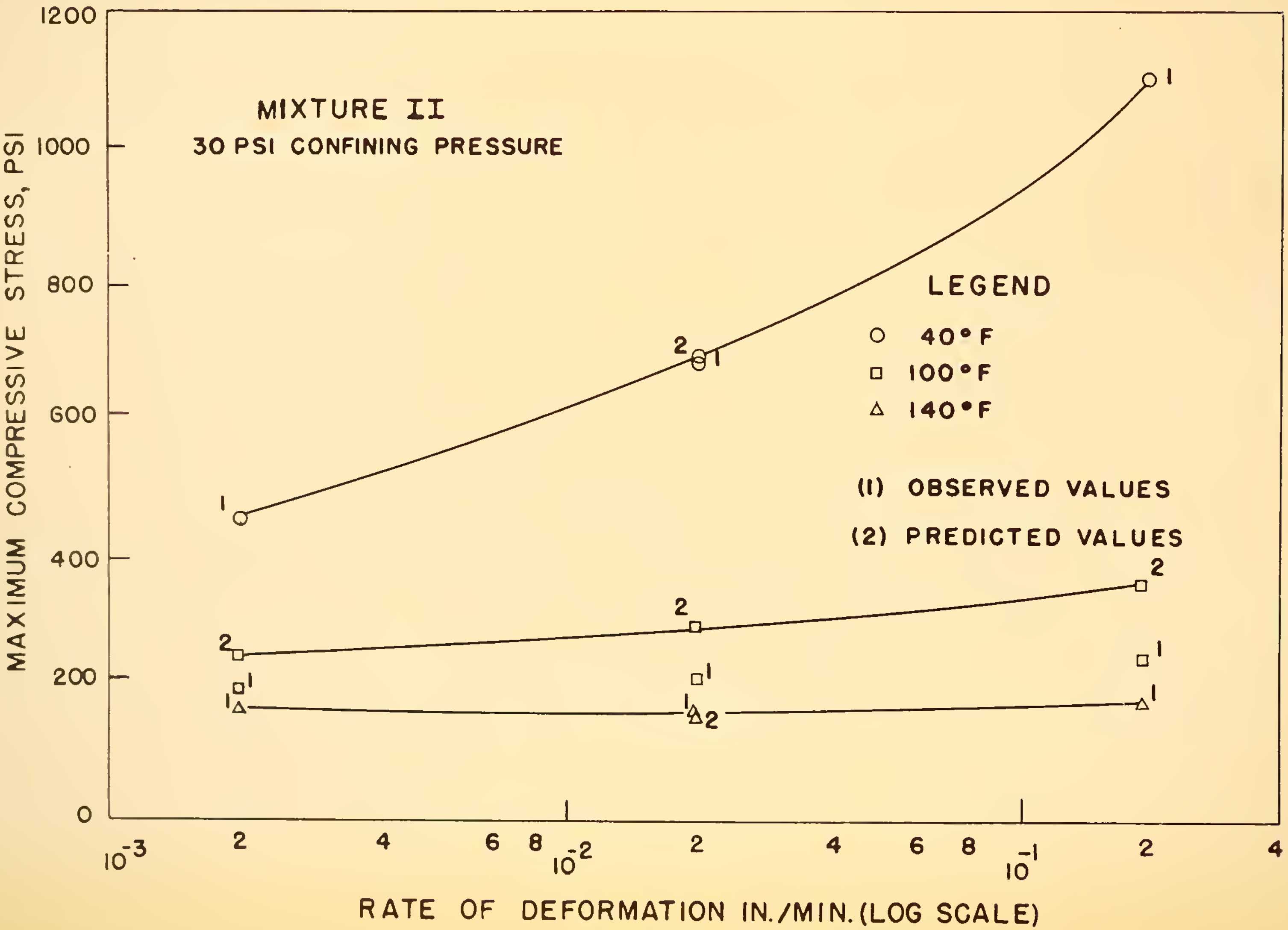


FIGURE 9

temperature to the original model might increase its effectiveness when it comes to the case of tests made under the confined condition.

REPEATED LOAD TESTS

In addition to the apparent lack of understanding of the fundamental relationship existing among the variables of temperature, rate of deformation and strength as evidenced by the variations employed in current test procedures, these procedures also lack recognition of plastic movements that result from repeated loads. Since mixtures in the field are subjected to numerous loadings, it was decided to utilize a test procedure in the laboratory that would simulate this action of repeated loads. In this part of the study, an evaluation of the effect of repeated loads upon specimens of sheet-asphalt Mixture II tested in unconfined and confined compression was undertaken.

The repeated-load sequence was performed by utilizing a combination mechanical and hydraulic system. The test set-up, without the water bath used for temperature control is shown in Figure 10. The rate of deformation was controlled by the mechanical testing machine. A hydraulic jack was used in the system to obtain the immediate release of load when the desired load on the specimen was reached.

Figure 11 gives a general representation of three cycles of load repetition. The dotted portions of the curve indicate that the period of time between loading was an undetermined variable. A sufficient period of time was allowed between load applications in order to permit most of the retarded rebound to take place. Thus, a slow-cycle test resulted.

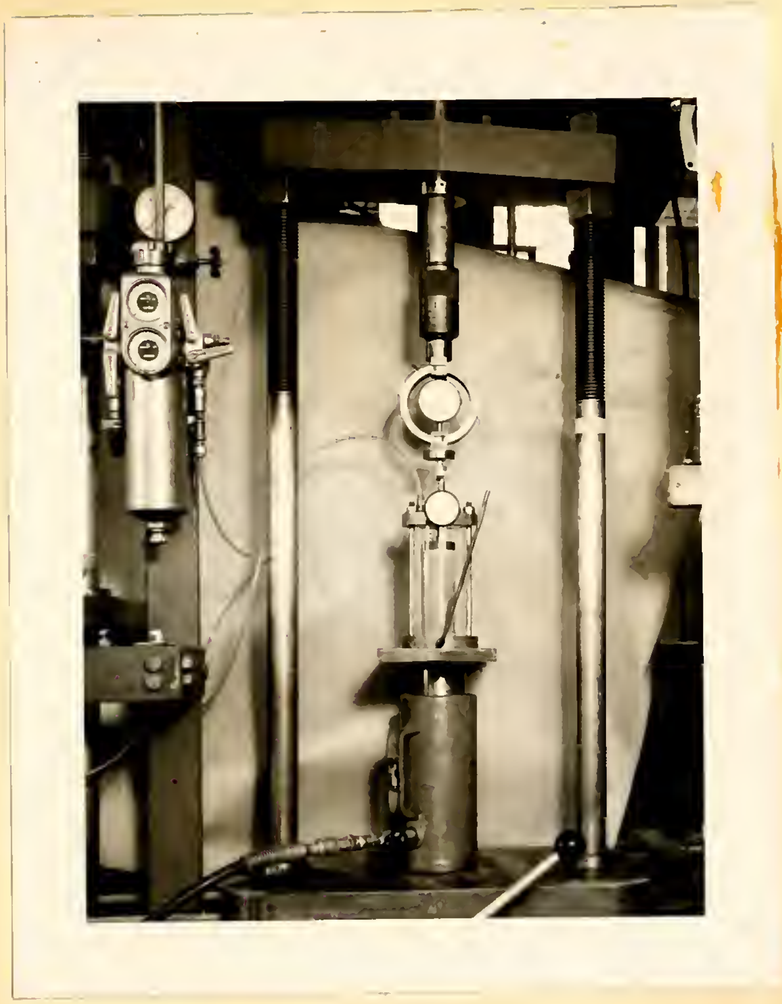


Figure 10 General View of Confined Repeated Load Apparatus Ready for Test

GENERAL RELATIONSHIP BETWEEN DEFORMATION AND TIME FOR REPEATED LOAD CYCLES

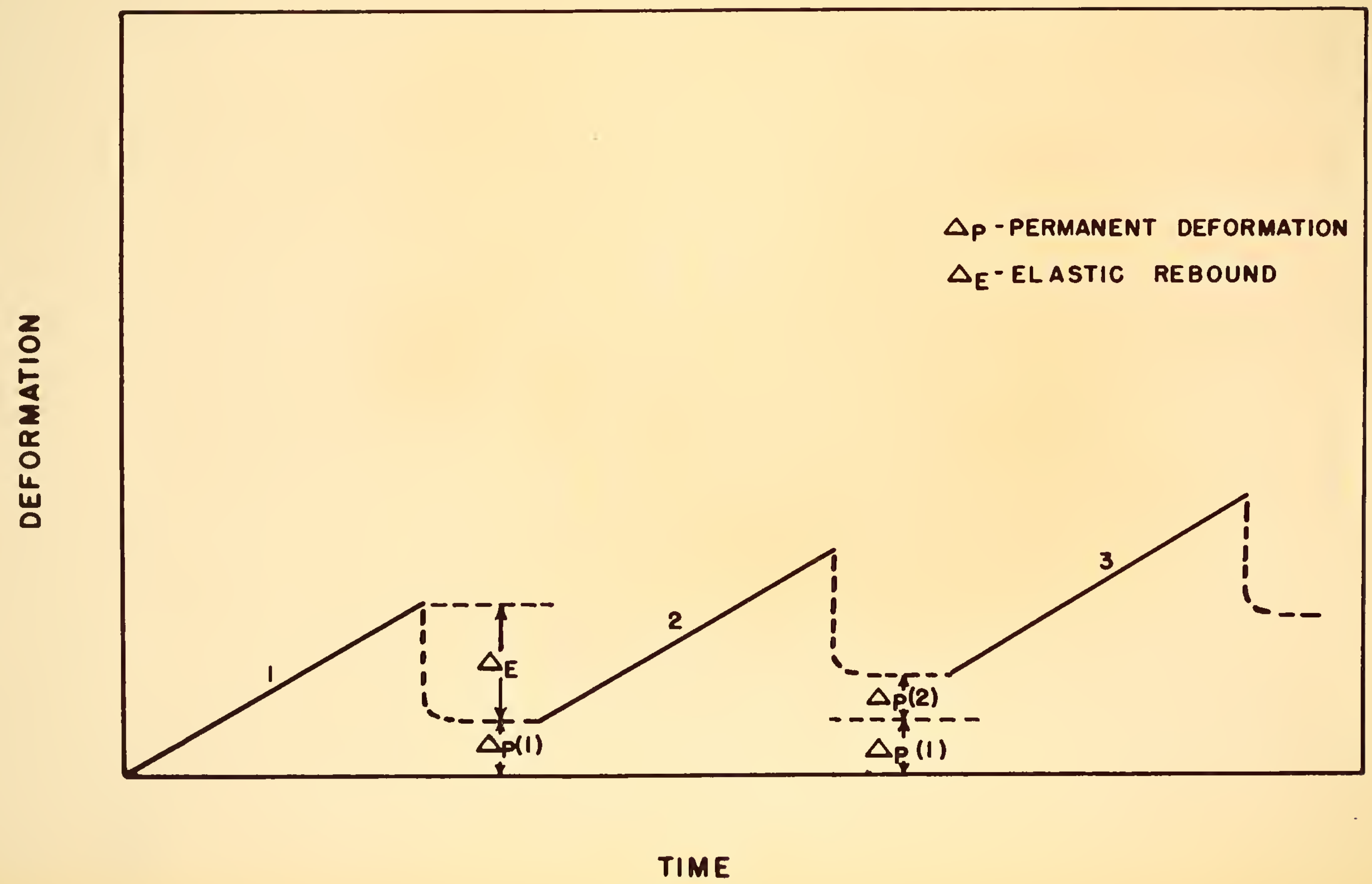


FIGURE 11

Unconfined Repeated Load Tests

The specimens tested in the unconfined, repeated-load test sequence were formed from the sheet-asphalt Mixture II as previously described. The repeated loads were applied at varying rates of deformation and at various temperatures. Their magnitude was selected as fixed percentages of the maximum compressive stress of the mixture as determined from the unconfined compression test. The total deformation and amount of permanent deformation for each cycle was measured.

The effect of repeated loads at varying percentages of the maximum unconfined compressive stress as determined for 0.2 in./min. and 40°F is shown in Figure 12 where permanent deformation is plotted against the logarithm of the number of load repetitions. The top line represents an applied stress of 777 psi., which is 75 percent of the maximum unconfined compressive stress. The middle line represents an applied stress of 518 psi, which is 50 percent of the maximum unconfined compressive stress. The bottom line represents an applied stress of 259 psi, which is 25 percent of the maximum unconfined conpressive stress.

It can be noted that the relationship starts out as a straight line in all cases when the permanent deformation is plotted against the logarithm of the number of load repetitions. At some stage, dependent upon the applied stress, the plot deviates sharply from the straight line, as is shown in the upper two curves of Figure 12. What occurs at this point is a matter of conjecture. It is hypothesized that the asphalt film between aggregate particles is being reduced in dimension until some critical thickness is reached. At this

RELATIONSHIP BETWEEN PERMANENT DEFORMATION AND NUMBER OF LOAD REPETITIONS FOR VARIOUS APPLIED STRESSES

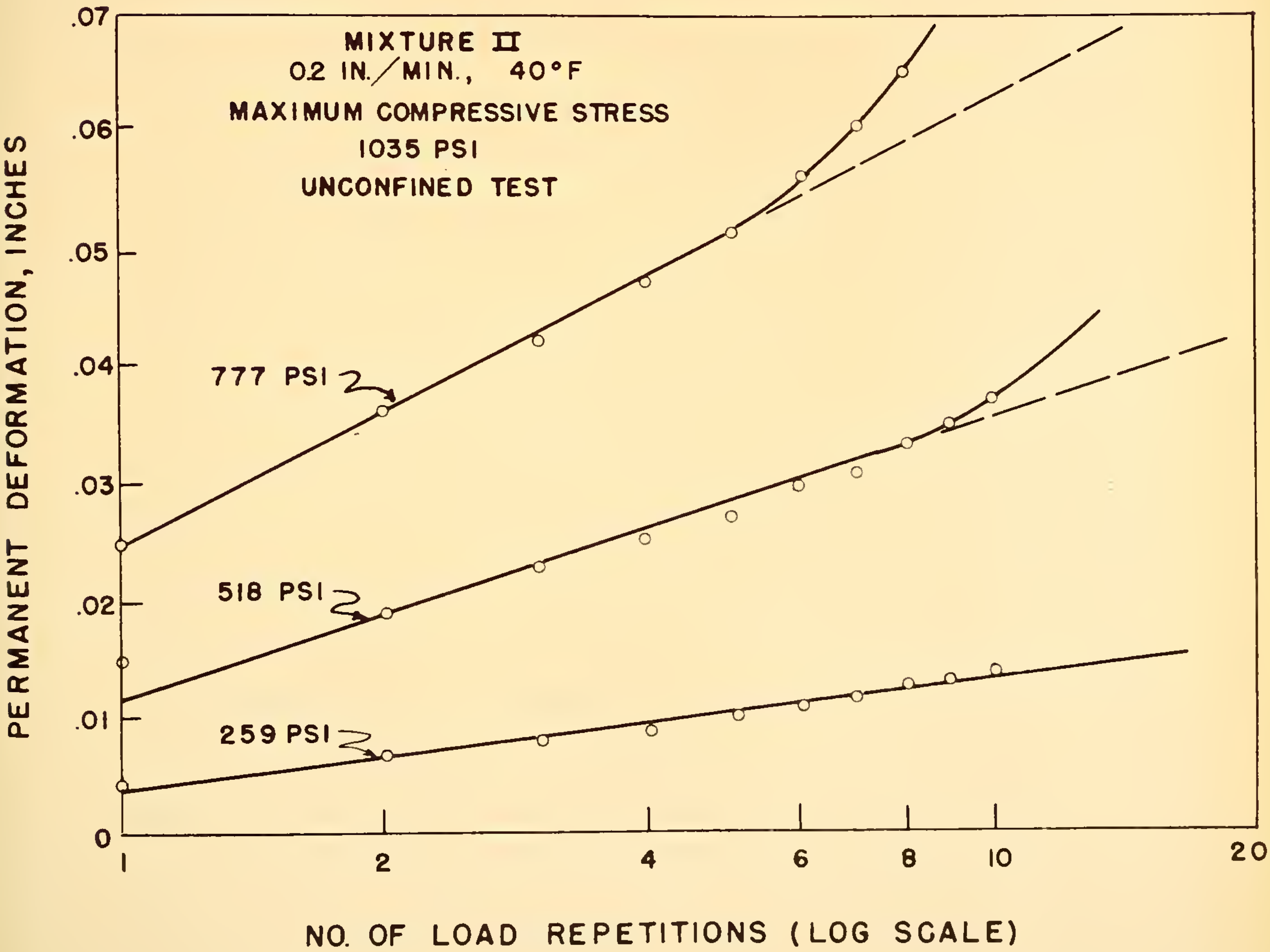


FIGURE 12

point, in order to sustain the load, adjustment in the specimen takes place by reorientation of the aggregate particles themselves. This gives rise to excessive shear deformations which are measured as permanent deformations. The point where excessive shear deformation occurs was taken as the failure criterion.

Under this hypothesis, the elastic part of the deformation would take place principally in the asphalt film which is bound firmly to the aggregate in a polymolecular layer. As the applied stress decreased, the number of loading cycles necessary to cause deviation from a straight line plot of deformation versus log of load repetitions increased. When the applied stress was 25 percent of the maximum compressive stress, no deviation was observed for the number of load repetitions used in this sequence of observations.

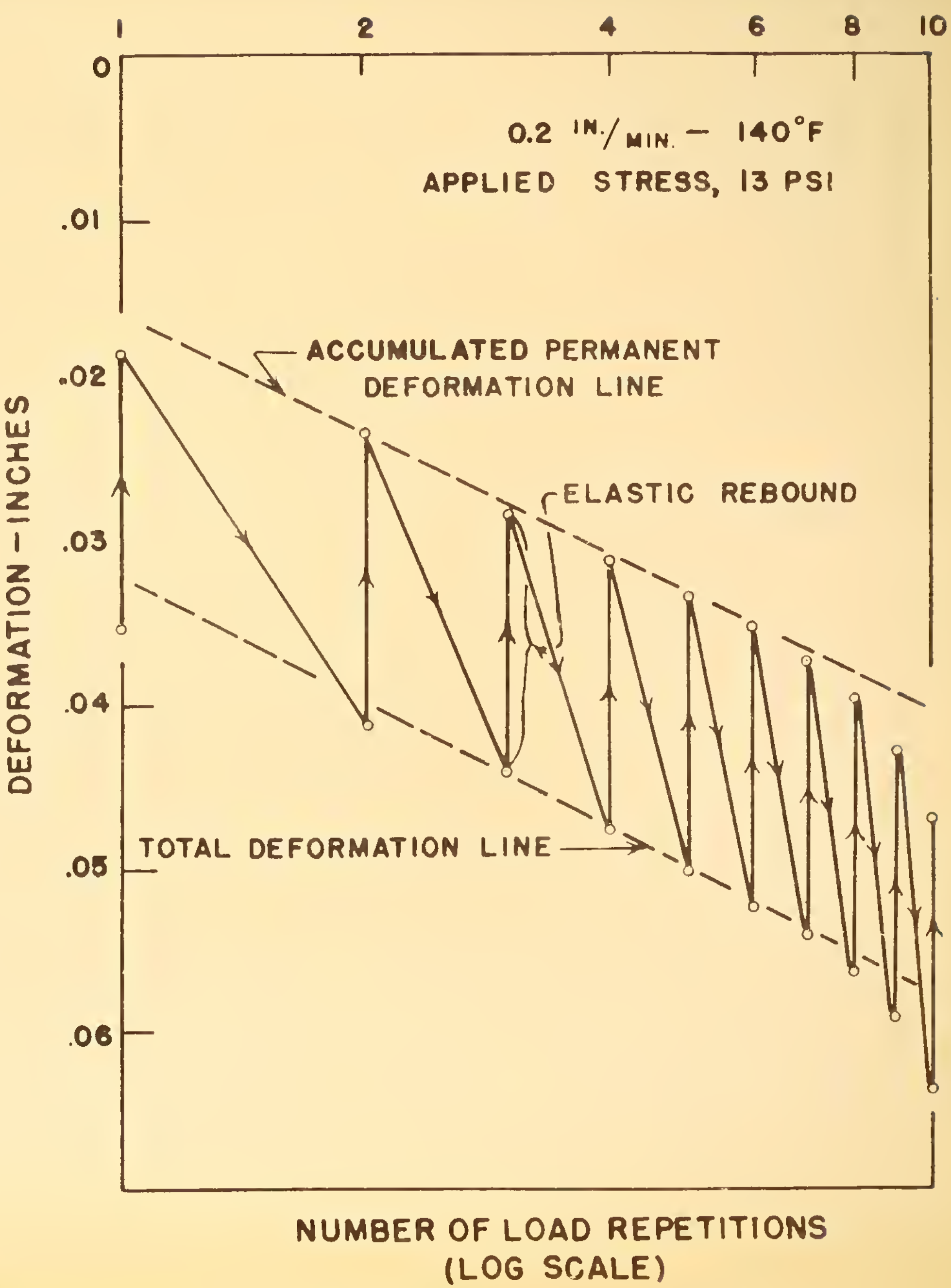
A graphical representation of the deformations experienced by a specimen subjected to unconfined repeated loads at a temperature of 140° F and at a rate of deformation of 0.2 in./min. is shown in Figure 13. The applied stress in this test was 13 psi. The arrows indicate the deformation record of the specimen while being loaded and unloaded. An important point brought out by this plot is that the elastic rebound appears to be independent of the past deformations the specimen has undergone. The dotted parallel lines show this. These two lines are the accumulated permanent deformation line and the accumulated total deformation line. Even after excessive shear deformations have taken place, the elastic rebound appears to remain constant. This tends to support the concept that the elastic part of the deformation takes place principally in the asphalt films surrounding the aggregate particles.

DEFORMATION RECORD

OF A SPECIMEN SUBJECTED

TO REPEATED LOAD

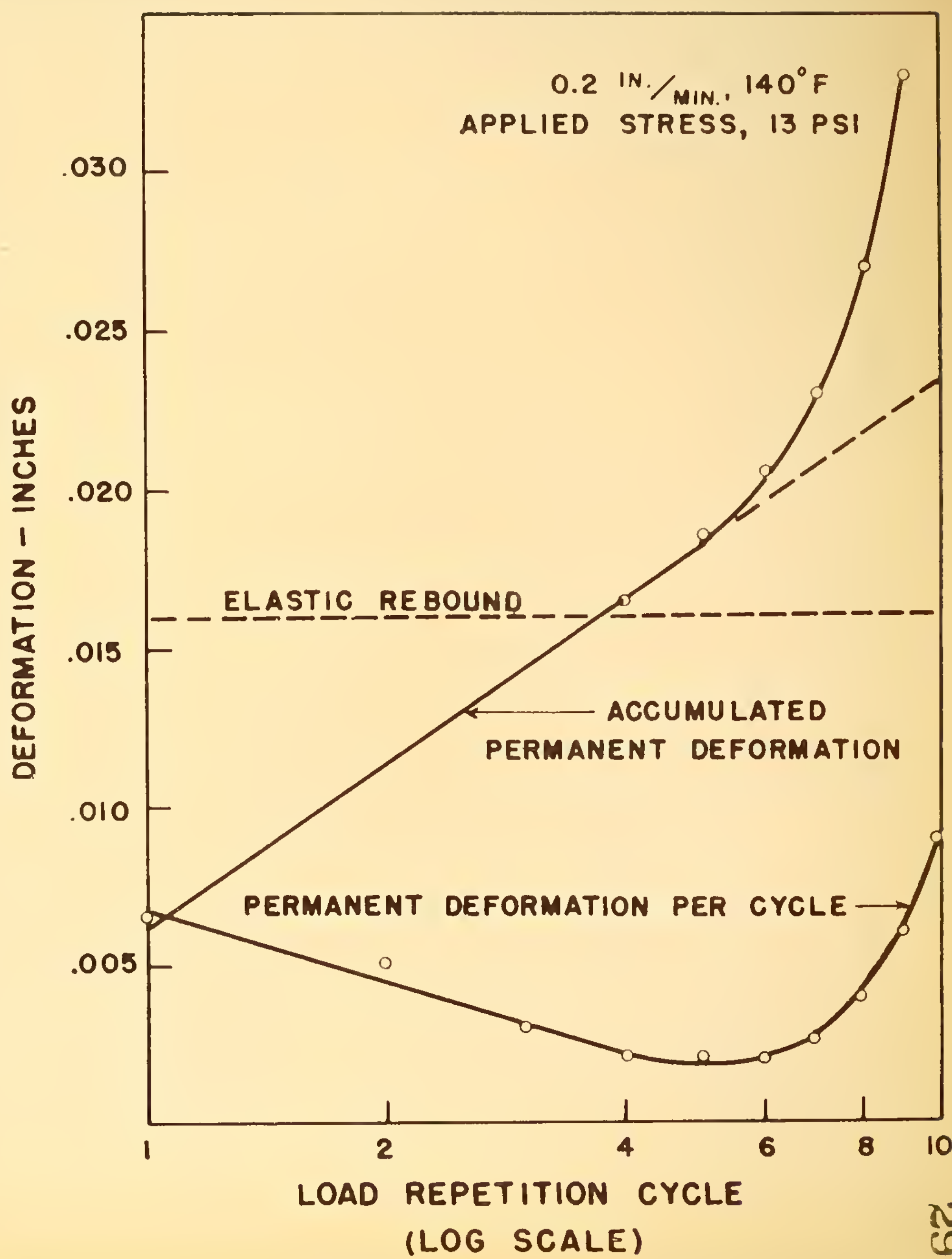
FIGURE 13



The curves shown in Figure 14 represent the break-down of the deformation experienced by a specimen for each load cycle in the repeated load test at 0.2 in/min. and 140°F using an applied stress of 13 psi. It shows the elastic rebound as a constant for the duration of the test. The amount of permanent deformation experienced by the specimen for each load cycle decreased to a minimum point and then increased rather sharply. It can be seen upon examination of the accumulated permanent deformation curve that this sharp increase in permanent deformation per cycle results in the deviation from a straight line plot. It was postulated that excessive shear deformations became important at this (failure) point.

Under each test condition there appeared to be an applied stress that could be cycled without excessive shear deformations occurring. At some level no further increase in permanent deformation was noted after a small number of load applications. This stress in each case was labeled as the endurance limit. At each test level the endurance limit was approximately 25 percent of the maximum compressive stress. The asphalt film must have been able to absorb these stresses since apparently no particle reorientation occurred. It can be noted that a large change in the rate of deformation did not bring about too great a change in the endurance limit. The results of this test series are shown in Table 7 in Appendix B and depicted graphically in Figure 15.

Figure 15 enables one to make a comparison of the relative effects of the rate of deformation and temperature upon the endurance limit. It can be seen that the temperature has a much greater effect upon the endurance limit than does the rate of deformation. For example,



RELATIONSHIP BETWEEN
ELASTIC AND PERMANENT
DEFORMATION
WITH REPETITIVE LOADING

FIGURE 14

APPLIED STRESS — ENDURANCE LIMIT RELATIONSHIPS AT VARIOUS TEMPERATURES AND RATES OF LOADING

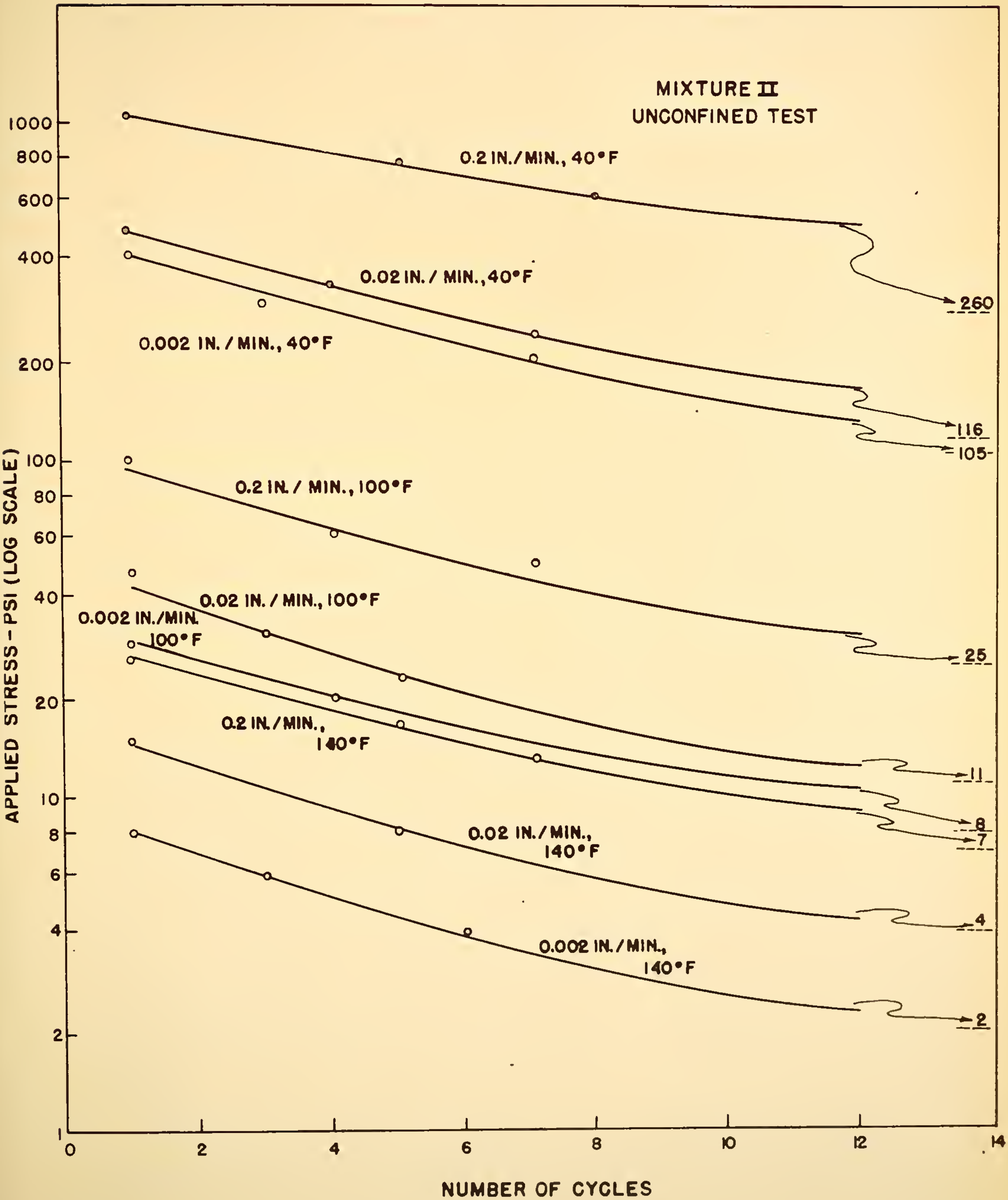


FIGURE 15

at a temperature of $40^{\circ}F$, changing the rate of deformation from 0.2 in./min. to 0.002 in/min. lowers the endurance limit from 259 psi to 105 psi. At a rate of deformation of 0.2 in./min., changing the temperature from 40 to $140^{\circ}F$ lowers the endurance limit from 259 psi to 7 psi.

The concept has been presented that the elastic part of the deformation takes place principally in the asphalt film surrounding the aggregate particles. The endurance limit and the elastic part of the deformation must be closely related. Under this hypothesis a certain film thickness must be present to resist the stress known as the endurance limit.

It would appear that the effective thickness of this asphalt film is associated with the viscous resistance which in turn is affected by temperature and rate of deformation. The viscous resistance of the asphalt film varies directly with the rate of deformation and inversely with the temperature.

Confined Repeated Load Test

The confined, repeated-load test series was performed on the sheet-asphalt mixture (Mixture II) to determine the effect of lateral support upon the relationship of applied stress, number of load applications necessary to reach failure, temperature, and rate of deformation. The inclusion of lateral support makes this study more realistic from the standpoint of actual field performance in bituminous pavements where some degree of confinement is known to exist.

Some typical results of these tests are shown graphically in Figures 16 and 17. The permanent deformation is plotted against the

RELATIONSHIP BETWEEN PERMANENT DEFORMATION AND NUMBER OF LOAD REPETITIONS FOR VARIOUS APPLIED STRESSES

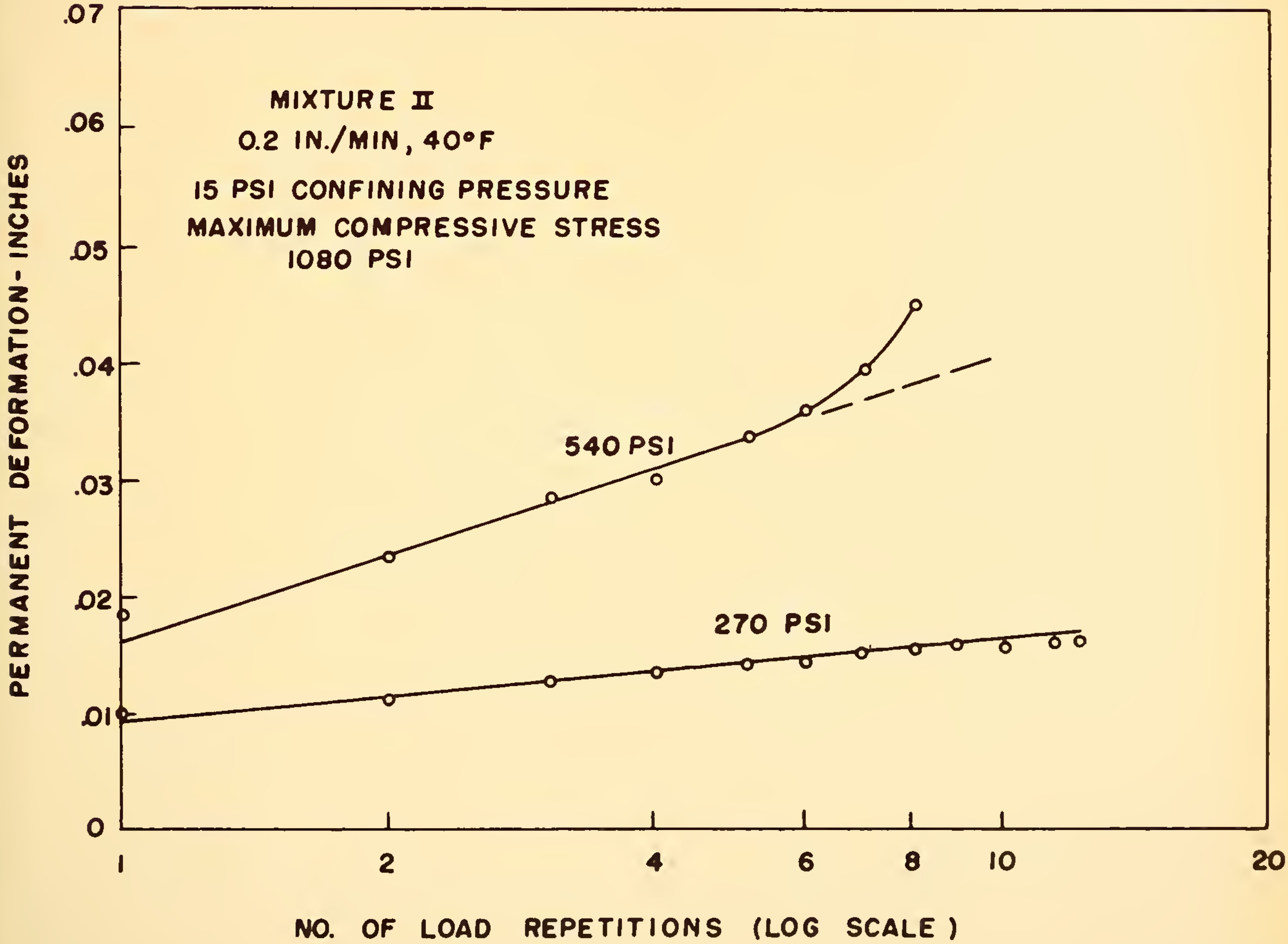


FIGURE 16

RELATIONSHIP BETWEEN PERMANENT DEFORMATION AND NUMBER OF LOAD REPETITIONS FOR VARIOUS APPLIED STRESSES

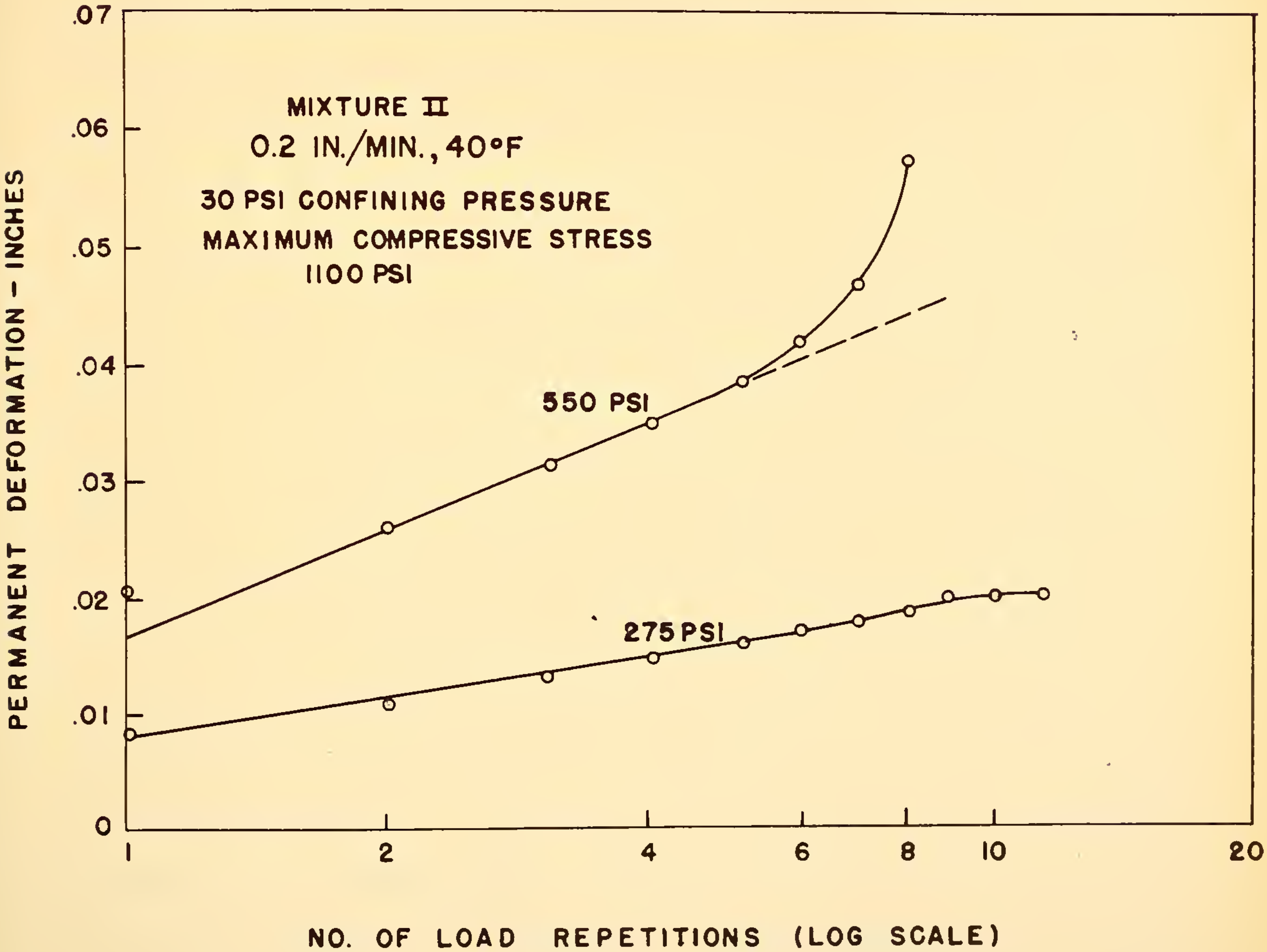


FIGURE 17

log of the number of load applications for different applied stresses. As in the case of the unconfined, repeated-load test the relationship starts out as a straight line. At some stage, dependent upon the stress condition, the plot deviates sharply from the straight line, as is shown in the upper curves of the two figures. This stress level was 50 percent of the maximum compressive stress. It was previously postulated what occurred at this point. When the applied stress was 25 percent of the maximum compressive stress, no deviation was observed for the number of load repetitions used in this sequence of observations. This stress was again labeled as the endurance limit.

The number of load repetitions necessary to cause failure under various confined test conditions is presented in Table 8 in Appendix B and shown graphically in Figure 18 where the applied stress (log scale) is plotted against the number of load applications necessary to cause failure. The endurance limit stress is shown as a limiting value on the extreme right of the plot.

From Figure 18 it would appear that an applied stress of 25 percent of the maximum compressive stress for each given test condition could be cycled a number of times without excessive shear deformations occurring. This value is that referred to in a previous section as the endurance limit. Thus, it can be seen that, for the mixture being tested (sheet asphalt), the endurance limit was approximately 25 percent of the maximum compressive stress determined at a given set of test conditions regardless of the lateral support applied.

This is important since it shows that the addition of lateral

APPLIED STRESS - ENDURANCE LIMIT
RELATIONSHIPS AT VARIOUS TEMPERATURES
AND RATES OF LOADING

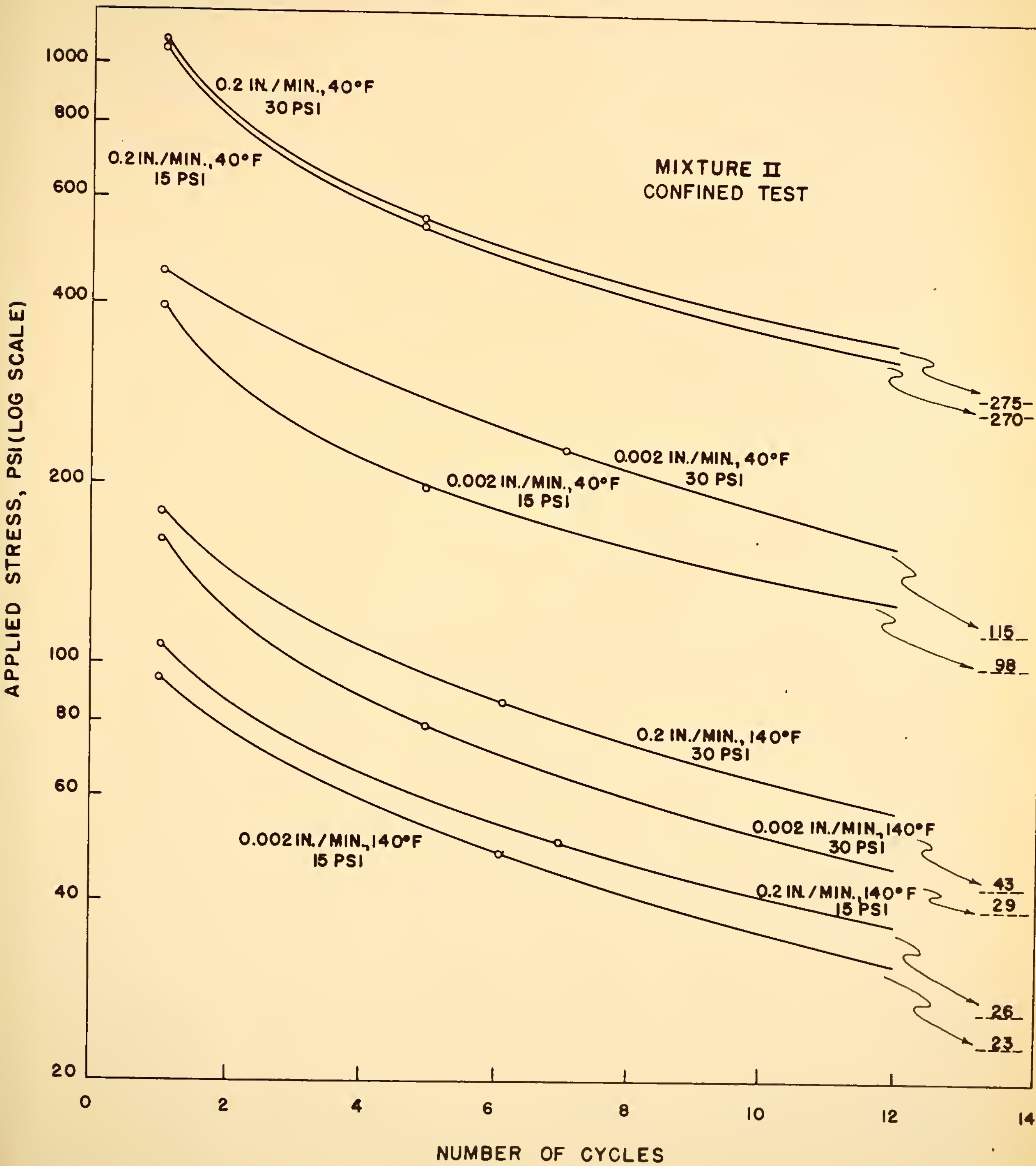


FIGURE 18

support does not change the basic behavior of the mixture in any unexpected manner. While it was stated that the endurance limit remained at about 25 percent of the maximum compressive stress, it must be remembered that at the lower rates of deformation and the higher temperatures, introducing lateral support materially increased the maximum compressive strength of the mixture. Thus, even though the endurance limit was approximately 25 percent of the maximum compressive stress for both the unconfined and the confined conditions, the endurance limit for the confined case was numerically much higher for the high temperatures and low rates of deformation than for the unconfined case. A comparison of Figures 15 and 18 will graphically show this.

Figure 18 shows the effect of lateral support upon the applied stress-endurance limit at various temperatures and rates of deformation. At a temperature of 40°F and a rate of deformation of 0.2 in./min., increasing the confining pressure from 0 psi to 15 psi and then to 30 psi increased the endurance limit from 260 psi, to 270 psi, and to 275 psi respectively. Thus, it can be seen that the introduction of lateral support at the test level did not affect the endurance limit appreciably. At that temperature and rate of deformation, the viscosity of the binder is such that the introduction of a confining pressure in the ranges of 0-30 psi had little effect upon the endurance limit. One would not expect the introduction of lateral support to affect the endurance limit at 40°F since there was little increase in the maximum compressive stress between 0 and 30 psi, at that temperature.

At the other extreme of test conditions, a temperature of 140°F and a rate of deformation of 0.002 in./min., increasing the confining pressure from 0 psi to 15 psi to 30 psi increased the endurance limit from 2 psi to 23 psi to 29 psi. This can be seen in Figure 18. In this case the confining pressure had a marked effect upon the endurance limit. At a temperature of 140°F and 0.002 in./min., the viscosity of the binder was so low that the introduction of the confining pressure "stiffened" the mix appreciably.

THE STRENGTH OF A THIN LAYER OF BITUMINOUS CONCRETE

The classical approach to 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 principle 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 the purpose of this portion of the investigation to estimate this confinement by means of laboratory tests.

Triaxial Tests

The first step in this part of the investigation was to obtain conventional triaxial test data for the bituminous-concrete mixture being studied. This mixture, containing six percent asphalt, was compacted into specimens four inches in diameter and 9-1/2 inches high by double-plunger static compaction. The mixture was studied at two density levels, 140 and 146 pounds per cubic foot. Density results from numerous pavement cores taken in the past, and current Indiana practice, indicated that the selected asphalt content and density values are realistic ones for the mixture employed. Some test results from pavement samples 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.

Duplicate specimens were tested in triaxial compression at a temperature of 80°F and at a rate of deformation of 0.02 inches per minute. Three lateral pressures were used: 15, 30 and 120 psi.

The results of the triaxial tests are shown in Table 9 in the Appendix B and are plotted as Compressive Strength versus Lateral Pressure in Figure 19. The relationship between this plot and the Mohr rupture envelope is such that if the lines of Figure 19 are straight, the Mohr envelopes will be also. Therefore, it is indicated that for these tests the Mohr rupture envelope is linear at least up to a confining pressure of 120 psi. Also it is shown that identical values for angle of internal friction were obtained for the two states of compaction.

An advantage of high pressure triaxial tests is illustrated by Figure 19. If one agrees that the relationship is linear to 120 psi and that knowledge of slope and intercept is desirable, it is evident that test results should be obtained 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.

Teste of Slab-Type Specimens

The second step in this part of the study was to determine the strength or load-carrying capacity of a thin slab of bituminous concrete when loaded over an area that was large with respect to specimen thickness but small with respect to specimen area. An essential feautre of this was the determination of a combination of specimen area and loaded area such that the strength of the specimen was independent of its area. The specimen thickness was fixed at two inches and the area to be loaded was chosen to be that of a four-inch

TRIAXIAL TEST RESULTS

COMPRESSIVE STRENGTH VS. LATERAL PRESSURE

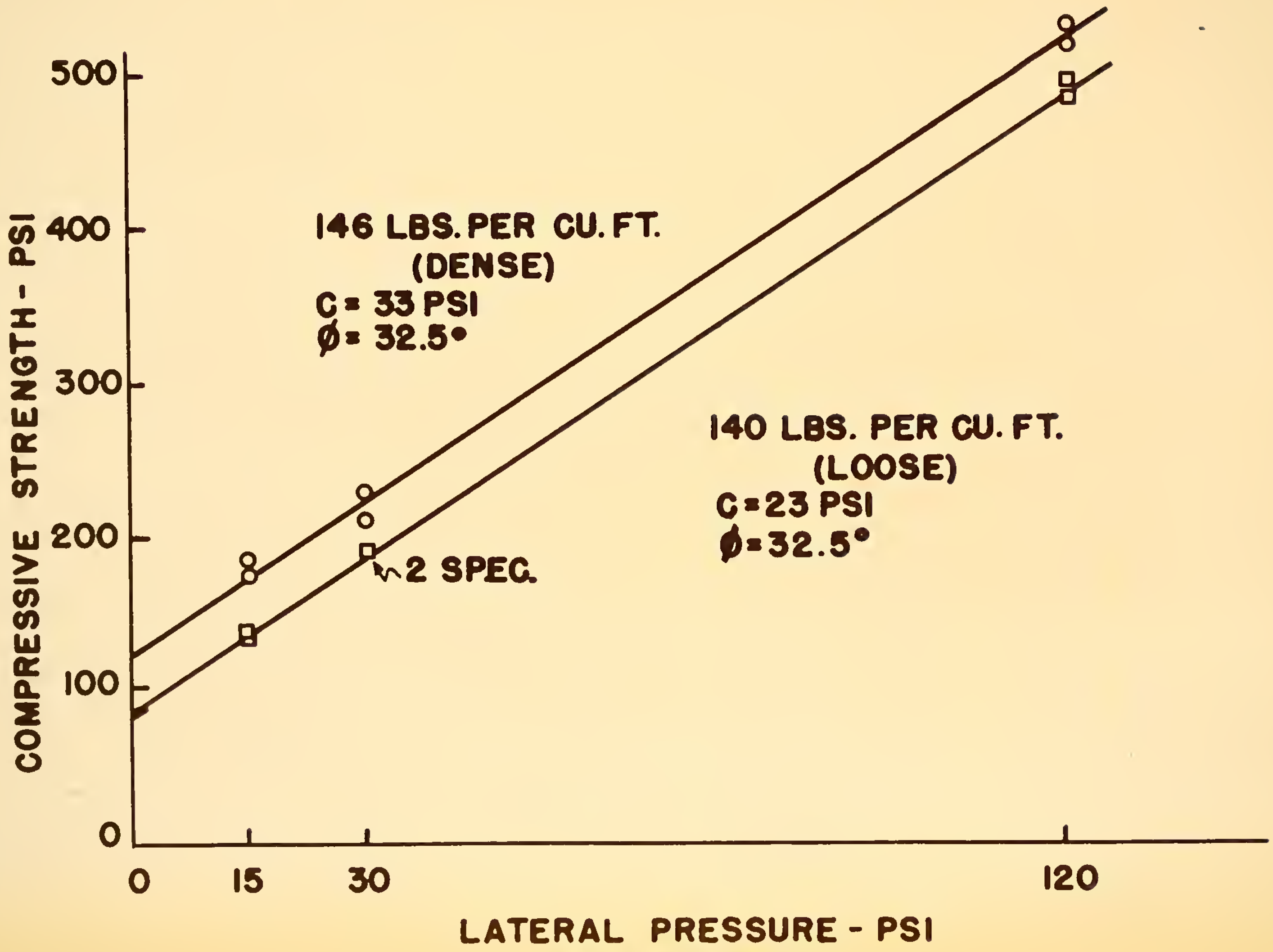


FIGURE 19

diameter plate. The bituminous concrete was identical in composition with that used for the triaxial test specimens.

In order to fabricate slab-specimens two-inches thick and of various sizes, a method of vibrational compaction was developed which employed a small (1-1/4-inch piston) pneumatic vibrator. The vibrator was operated at a line pressure of 60 psi and at this pressure the vibration frequency was approximately 3,170 cycles per minute. A foot plate or base plate 3-inches by 6 inches by 1/4 -inch in thickness was attched to the vibrator to serve as a compacting foot.

Three sizes of slabs were made, 8 by 8 inches, 11 by 11 inches, and 16 by 16 inches. For forming the two smaller size specimens, an adjustable steel mold was used. The 16 by 16-inch specimens were compacted into wooden forms. This equipment is illustrated in Figure 20. In this figure, the mold is set-up for making an 11 by 11-inch specimen.

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 $\pm$ 10°F, and asphalt, heated to a temperature of 275 $\pm$ 5°F, were brought together in the proper proporations and mixed for three minutes after which an amount of mixture sufficient to form a two-inch thick specimen of 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 redistribution of material in the mold in a sideways direction through the action of the vibrator.

FIGURE 20 VIBRATOR AND MOLD FOR SLAB SPECIMENS

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 of 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 operatre 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.

In addition to the three sizes of slab specimens, one additional size of specimen two inches thick and four inches in diameter was obtained for this test series. Specimens of this size were 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. 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 in which the tests were made. The four-inch diameter plate (a steel disc) was used to transmit load to the 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 were carried to failure. With the variety of specimen sizes used, strength results were obtained for specimen area to loaded area ratios of 1, 5.1, 9.7, and 20.4. From the complete series of tests, the effect of the specimen area to loaded area ratio on the strength of the mixture could be determined.

The test results are shown in Table 10 in Appendix B and are plotted in Figure 21. In this figure it may be seen that rather large increases in compressive strength of the bituminous concrete were observed for two-inch thick specimens having successively larger areas. 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 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 21 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 have 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

COMPRESSION TEST RESULTS - VARIABLE SPECIMEN AREA SERIES

COMPRESSIVE STRENGTH VS. RATIO OF SPECIMEN AREA TO LOADED AREA

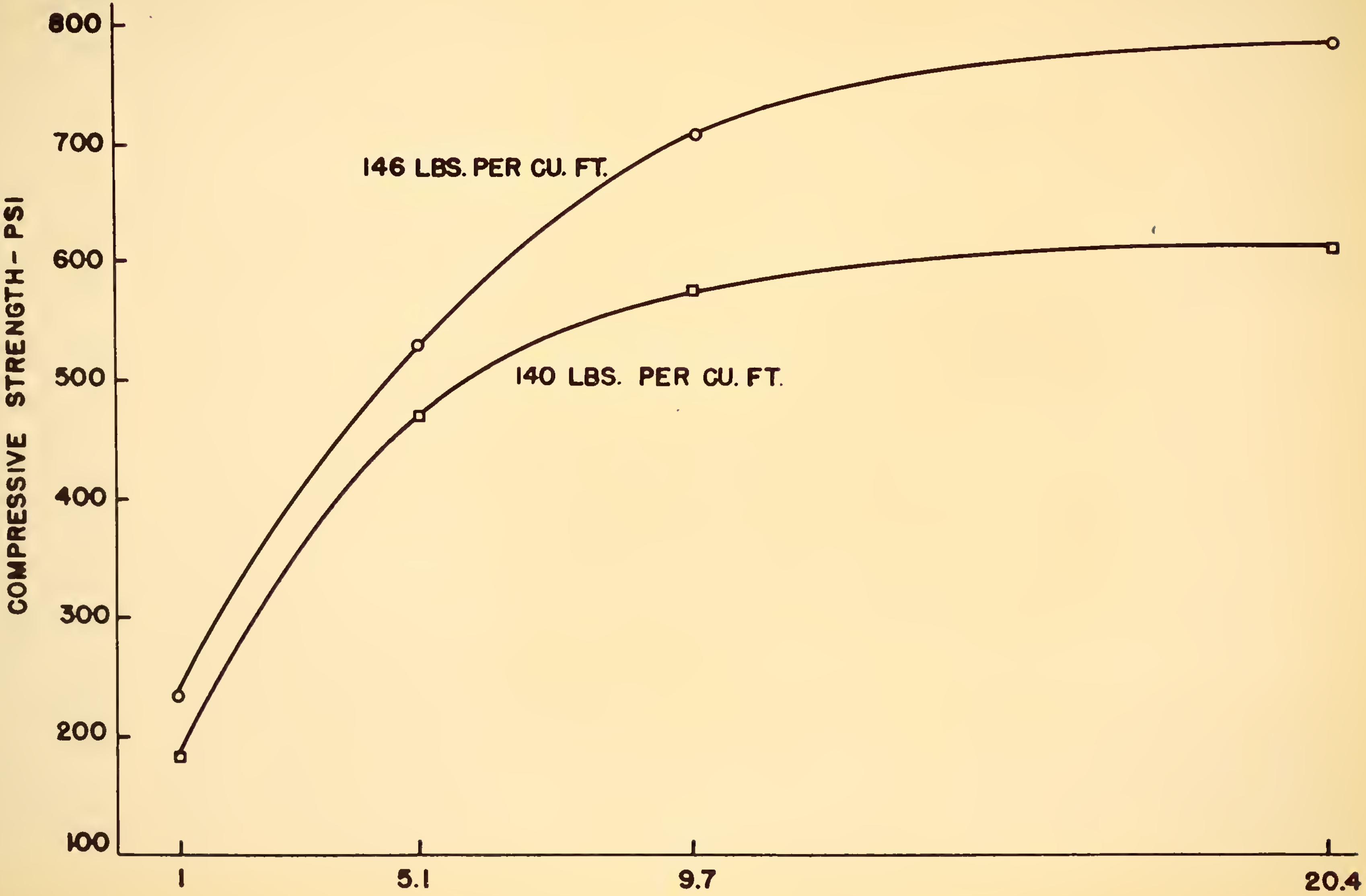


FIGURE 21

specimens 16 by 16 inches in size. That is, based on the shape of the curves shown in Figure 21, it is apparent 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 compressive strength 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 from the triaxial tests, at least an approximation of this confinement can be made.

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 trixial 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 test) and loaded over an area that was small relative to the area of the mixture? The results of the triaxial tests (Figure 19) are reproduced in Figure 22. 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 22 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 surprisingly high. They exceed the unconfined compressive strengths of the mixtures, the latter values being estimated from Figure 22 as 125 and 90 psi respectively. These results, however, appear to

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

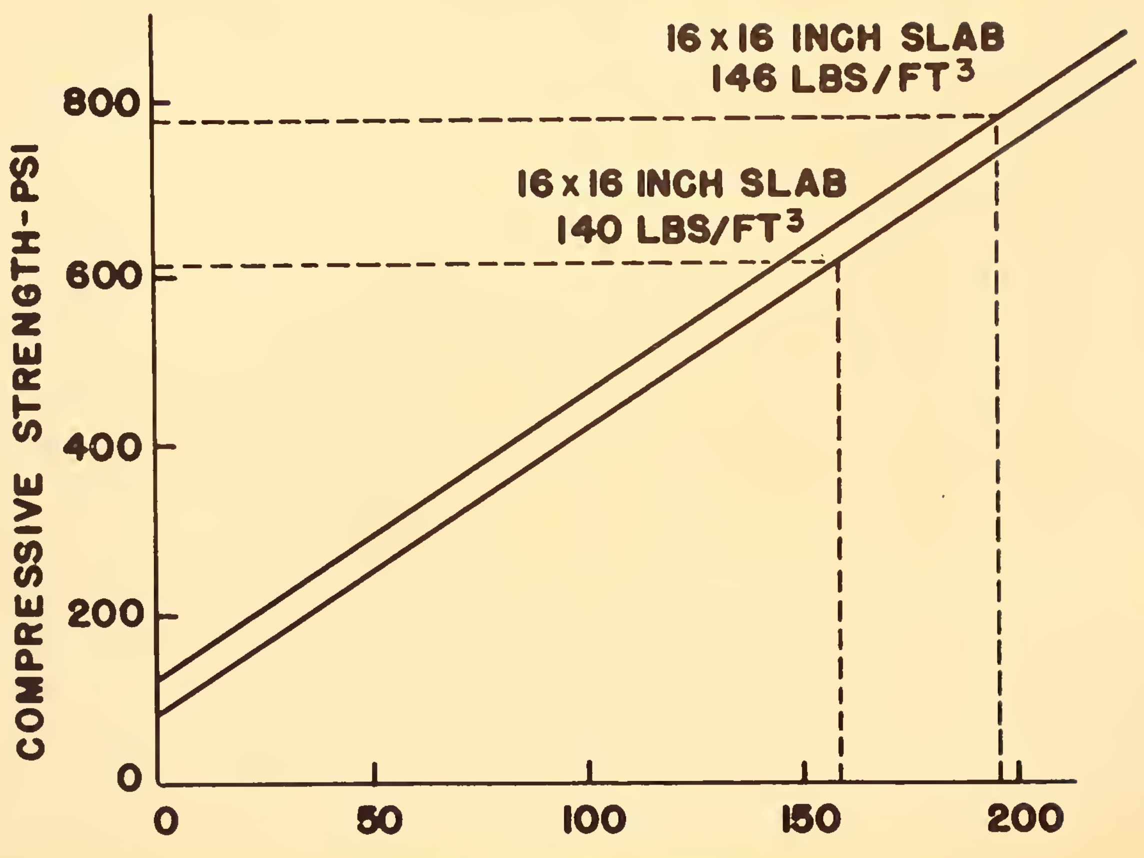


FIGURE 22

reinforce the hypothesis of McLeod[1] who has long maintained that the confinement in a bituminous layer is at least 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 several times the unconfined compressive strength of the mixture when tested at the same temperature and rate of deformation. In addition, 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 equal to or greater than the unconfined compressive strength of the mixture.

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[1] See references (12) and (13) in the Bibliography.

STRENGTH - RATE - TEMPERATURE RELATIONSHIPS FOR SHORT BITUMINOUS CONCRETE SPECIMENS

A previous section of this paper contained the results of an investigation in which a relationship was established between the strength of a bituminous mixture and the variables of rate of deformation and temperature. The tests were conducted on both sheet-asphalt and bituminous-concrete specimens all of which were of rational dimensions; that is, the specimen height was 2 or 2-1/2 times the diameter. It was felt that the validity of this basic relationship should be tested for other types of specimens, particularly for bituminous-concrete pavement cores and for laboratory-compacted specimens of "irrational" size.

Two such experiments were designed. In one, bituminous-concrete specimens four inches in diameter and two-inches thick were made in the laboratory at an asphalt content of six percent and compacted to a density of 146 pounds per cubic foot. Two methods of compaction were used in this series in order to study the effect of this variable. These methods were double-plunger static compaction and vibrational compaction. In the latter case, the specimens were obtained by cutting cores from 11 by 11-inch slabs made by methods previously described.

These laboratory-compacted specimens were tested in compression at two rates of deformation, 0.2 and 0.02 inches per minute, and at three temperatures, 40, 90, and 140°F. The temperature of the specimens was controlled by immersing them in a constant-temperature water bath for 1/2-hour before testing 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 an immersion heater, the voltage to which was regulated by a powerstat to maintain the desired temperature level in the bath.

These tests produced strength values for the bituminous-concrete specimens made by both methods of compaction and for all rate-temperature combinations. Duplicate determinations were made at all levels. The results of these tests are given in Table 11 in Appendix B.

The second experiment in this series involved the use of cores of bituminous concrete obtained from an overlay experiment in which thickness-of-lay was a variable. Three total thicknesses of specimen were represented in the experiment: 2, 3, and 4 inches. These specimens or cores were composed of both binder and surface material and the composition was as follows:

2-inch cores: one inch of binder and one inch of surface.

3-inch cores: 1-1/2 inches of binder and 1-1/2 inches of surface.

4-inch cores: 3 inches of binder and one inch of surface.

The cores were taken when the overlay had been in place for approximately five years. This overlay had been studied rather extensively with respect to its densification and cracking for several years and sufficient data were available to indicate that the mixture was homogeneous with respect to density within each section from which samples were obtained. It was concluded from this that the location of the core within a given section was a variable that could be excluded from consideration.

The cores when brought into the laboratory had rough, uneven bottom surfaces which necessitated their being capped with plaster of paris. The specimens were tested at three rates of deformation (0.2, 0.02, and 0.002 inches per minute) and at three temperatures (40, 90, and 140°F) in the same manner as previously described. A separate test series was made for each of the thicknesses so that a strength result was obtained for each size of core at all rate-temperature combinations. Duplicate determinations were made at all levels. These results are shown in Table 12 in Appendix B.

The data that were obtained from each test series were analyzed by multiple regression techniques to determine whether the general model that was found to be applicable to sheet asphalt mixtures could be applied to explain the relationship among the variables in the case of short specimens of bituminous concrete. The results of these analyses showed that this general relationship did hold true for the test conditions and specimen types employed. The equations derived by the regression analyses naturally differ with respect to the values of the constants because of differences in specimen compaction (for laboratory-compacted specimens) or because of differences in core thickness. These differences, however, did not change the underlying relationship which was determined for both the unconfined and confined tests on rational-size specimens of sheet asphalt and for the unconfined tests on all specimens of bituminous concrete.

We would conclude that the relationship between the strength of bituminous concrete and the variables of rate of deformation and temperature is of the form shown in equation 1.

REPEATED LOAD TESTS ON BITUMINOUS CONCRETE CORES

It was pointed out in the Introduction that field studies of the bituminous concrete commonly used in Indiana have shown the accumulation of permanent deformation in areas of the pavement subjected to repeated applications of a transient load. The use of an ultimate strength test as the sole criterion for stability might then be questioned when it is indicated that failure may occur by the repeated application of loads less than the ultimate strength of the material as measured by some conventional test.

In this part of the investigation an attempt was made to apply the concepts of a repeated load test to the strength evaluation of bituminous-concrete pavement cores. The cores tested were four inches in diameter and of three thicknesses, 2, 3, and 4 inches. The source and composition of these cores has been described briefly in the preceeding section.

The procedure followed in making the first series of repeated load tests on pavement cores was similar to that outlined for the tests which were made on the sheet-asphalt mixtures. We have referred to these tests as "slow-cycle" tests. It was originally intended to test each size of core at 25, 50, and 75 percent of its ultimate compressive strength, but before the full test series was completed it became obvious that the test method could be improved upon for the evaluation of bituminous-concrete cores. One of the shortcomings of the slow-cycle test as applied to the short pavement cores was the time required to obtain the number of cycles that appeared to be necessary to cause failure. The possibility of using a more rapid method of applying load repetitions was explored and from these explorations was developed the rapid-cycle repeated load test which is described in the following section.

Rapid-Cycle Repeated Load Test

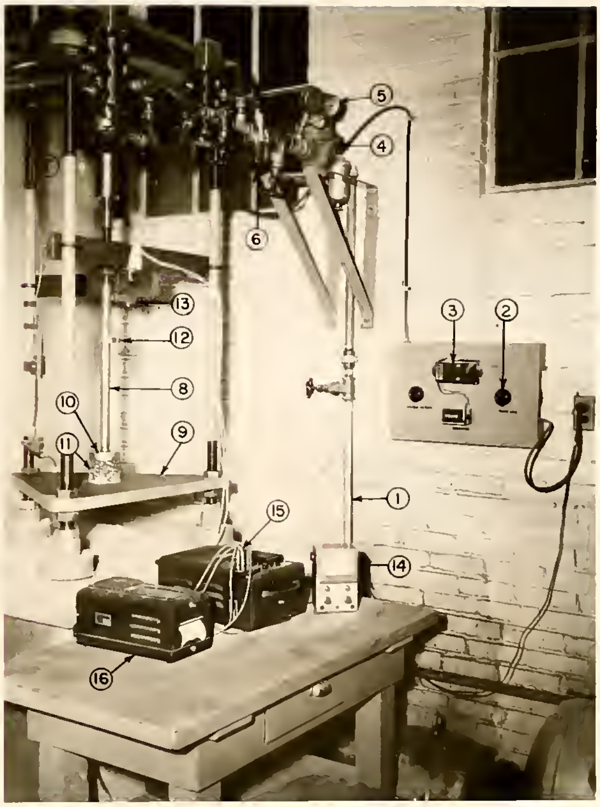
In the rapid-cycle repeated load test, a given stress (either 100, 150, or 200 psi) was instantaneously applied to a specimen and maintained for a brief (0.3 seconds) interval. At the end of this interval, the stress was instantaneously released and the specimen was permitted to rebound for another interval (3.7 seconds) after which the cycle was repeated. As in the case of the slow-cycle tests, the cumulative permanent deformation was the factor of primary interest.

Basically, the apparatus used for making the rapid-cycle repeated load tests 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 resting on the specimen. At the end of the pre-set time interval, during which the load was maintained constant, the inlet air valve 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.

To measure the deflections that were of interest in the study, movement of the loading piston was measured through the use of a Schaevitz Model .04-0-.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 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 permitted the useful range of the transformer to be extended 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 23. For clarity, the apparatus is shown without the water bath which was used to control the temperature of the specimen. The numbers on the photograph are explained in the key accompanying 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 later, or to 2,000 cycles. A few of the tests were continued somewhat beyond 2,000 cycles.


5
4
6
13
3
2
12
8
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1
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14
16

[illegible]

Key to Figure 23

1. Air line from air compressor
2. Main-line switch
3. Agastat
4. Grove GB 307-06 Poweractor Dome pressure regulator
5. Pressure gauge
6. Air line lubricator
7. Bellows 8002 FR 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

Discussion of Repeated Load Test Results

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." The first consideration given to the results of both the slow- and rapid-cycle repeated load tests was the determination of the nature of the relationship between cumulative permanent deformation and number of load repetitions.

The first approximation of this relationship with respect to the rapid-cycle tests was to test that which was found to be valid for the results of the slow-cycle tests on the sheet-asphalt mixtures. For those test results, it was found that a plot of permanent deformation versus the log of the number of load repetitions started out as a straight line. At some stage, which was dependent upon the applied stress, rate of loading, temperature and number of load applications, the plot would deviate upward from the straight line. This point of deviation was regarded as failure of the specimen and was selected as the failure criterion.

A semi-logarithmic plot of this sort implies that the underlying relationship between cumulative permanent deformation and number of load repetitions is of the form:

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

where y = cumulative permanent deformation

x = number of load repetitions, and

k and a = constants

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

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

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 on the sheet-asphalt mixtures, an analysis of the data by equation 6 appeared to be justified. However, it soon became evident that the relationship shown in equation 6 did not apply for the test conditions of the present part of the study in which the rapid-cycle load test was used. The results from several repeated load tests of both the slow-and rapid-cycle types were plotted on semi-log paper; some of these plots can be seen in Figures 24 and 25. Both of these figures show data points deviating 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 25). 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 the rapid-cycle part of the study, a relationship other than that expressed in equation 5 is operating. The whole question hinges on the definition of failure and it would appear as if a specimen that has accumulated deformation of 0.01 inches and shows no cracking or bulging has not "failed."

SLOW-CYCLE REPEATED LOAD TEST RESULTS

80°F

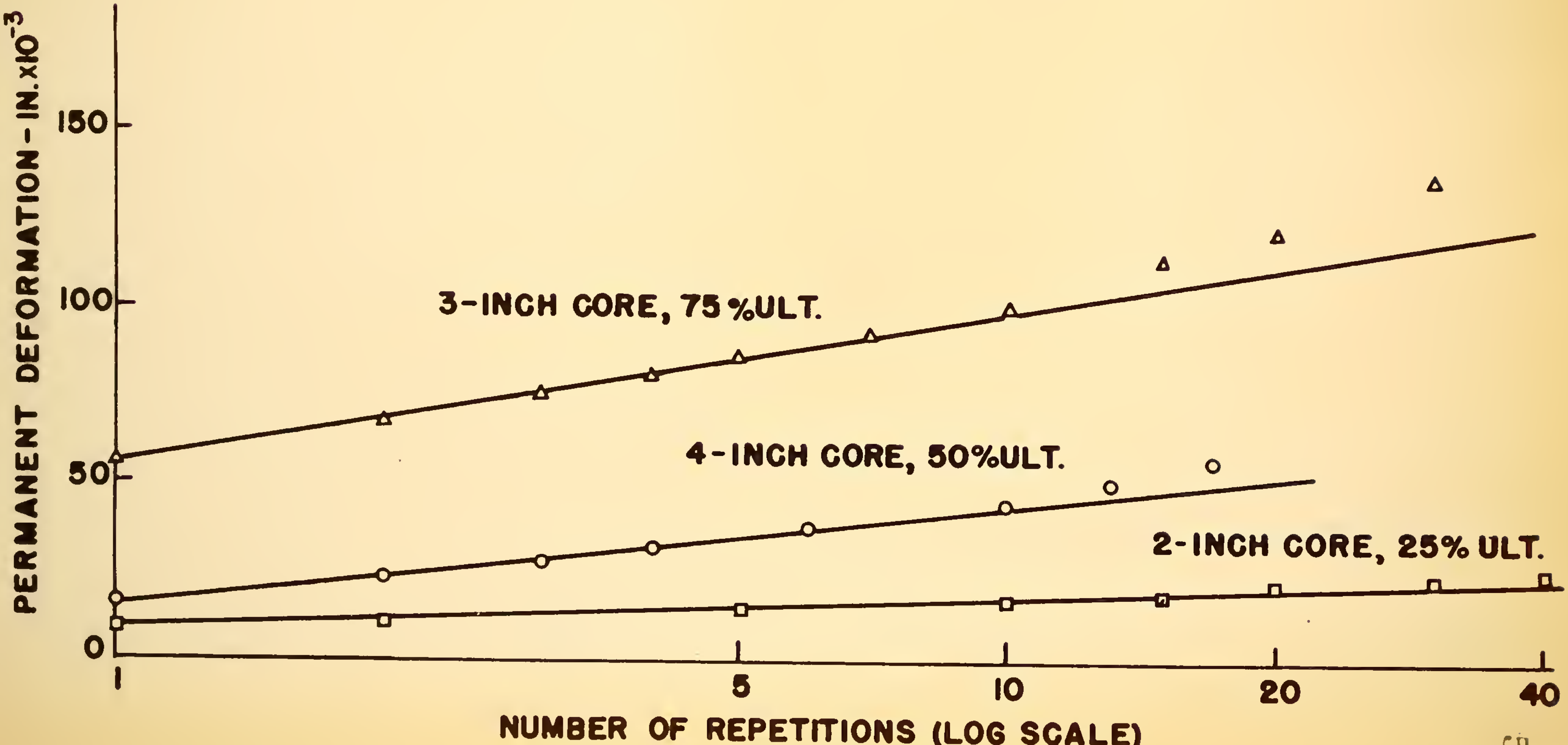

FIGURE 24

RAPID-CYCLE REPEATED LOAD TEST RESULTS

TWO-INCH PAVEMENT CORE

80°F - 100 PSI

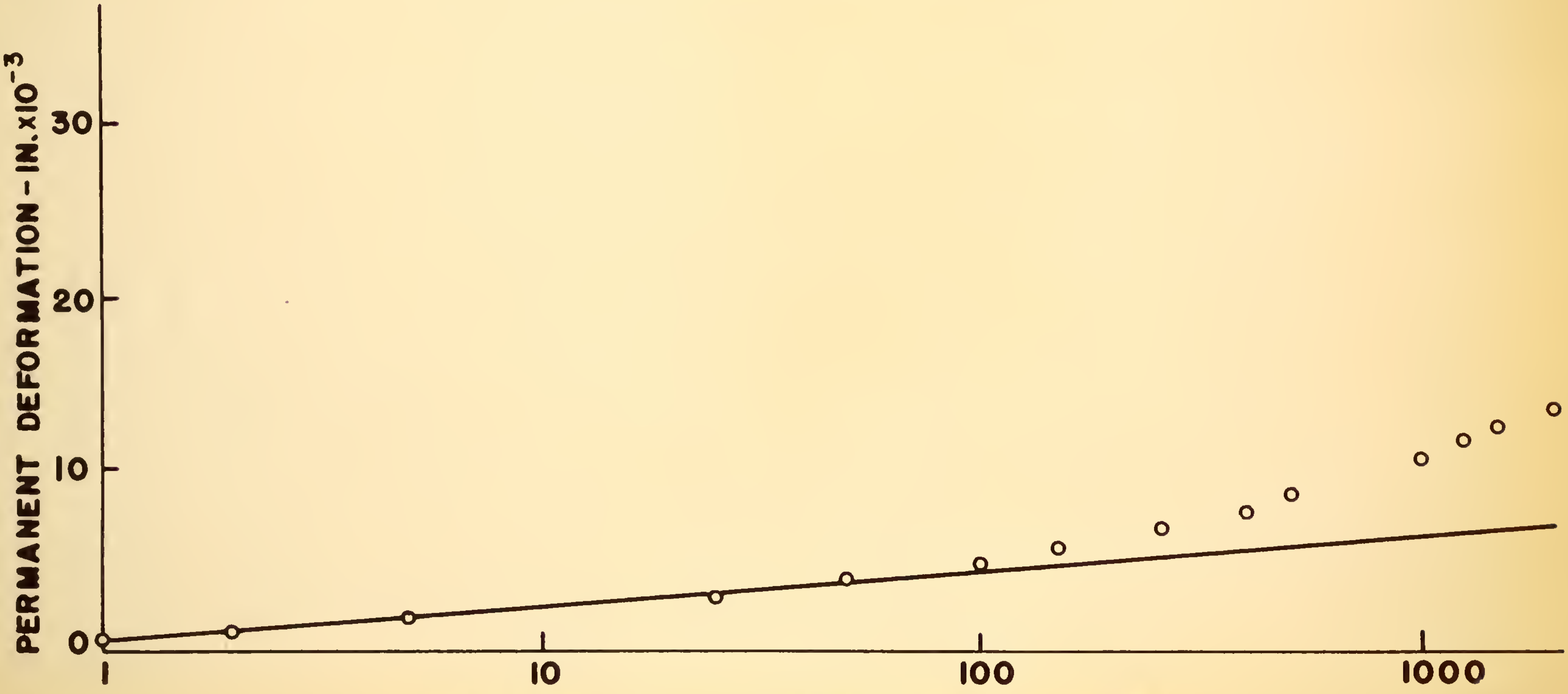

FIGURE 25

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, and

k and n = constants.

Taking the logarithm of each side of equation 7, it may be re-written:

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

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 several of the repeated load tests performed in this study were plotted to a log-log scale and are shown in Figure 26 for the slow-cycle tests and in Figures 27, 28, and 29 for the rapid-cycle tests.

The data for <u>all</u> of the repeated load tests have been tabulated and are included in Appendix B, Tables 13 through 19.

Figures 26 and 27 are log-log plots of the same data that are plotted to a semi-log scale in Figures 24 and 25 respectively. The objection to the interpretation of deviation from the straight line as failure is removed by plotting the data to a log-log scale or according to equation 7. The physical appearance of the specimens leads to the conclusion that a plot of log cumulative permanent deformation versus log of the number of load repetitions is the best method of interpreting the data. For this type of plot, deviation from the straight lines is again 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 1.

SLOW-CYCLE REPEATED LOAD TEST RESULTS
80° F

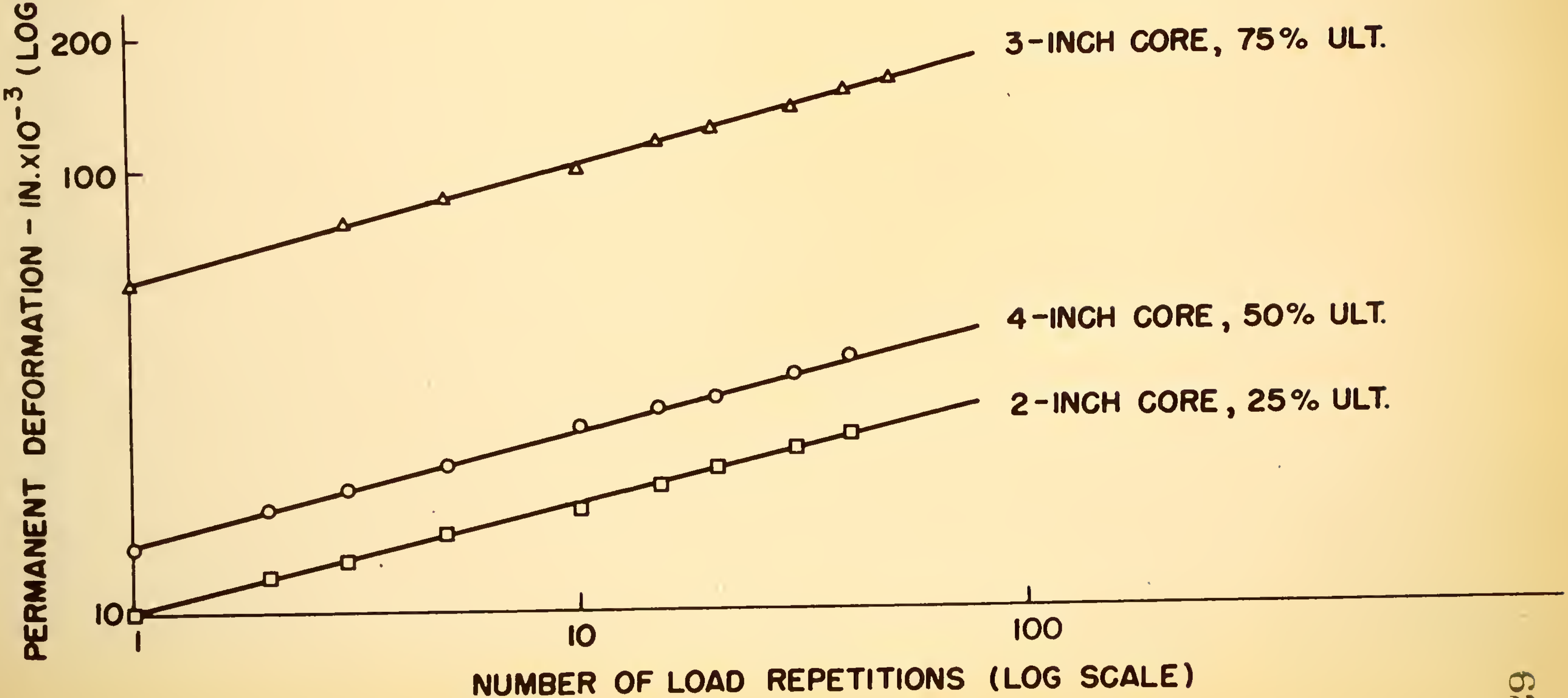

FIGURE 26

RAPID-CYCLE REPEATED LOAD TEST RESULTS

TWO-INCH PAVEMENT CORE

80°F – 100PSI

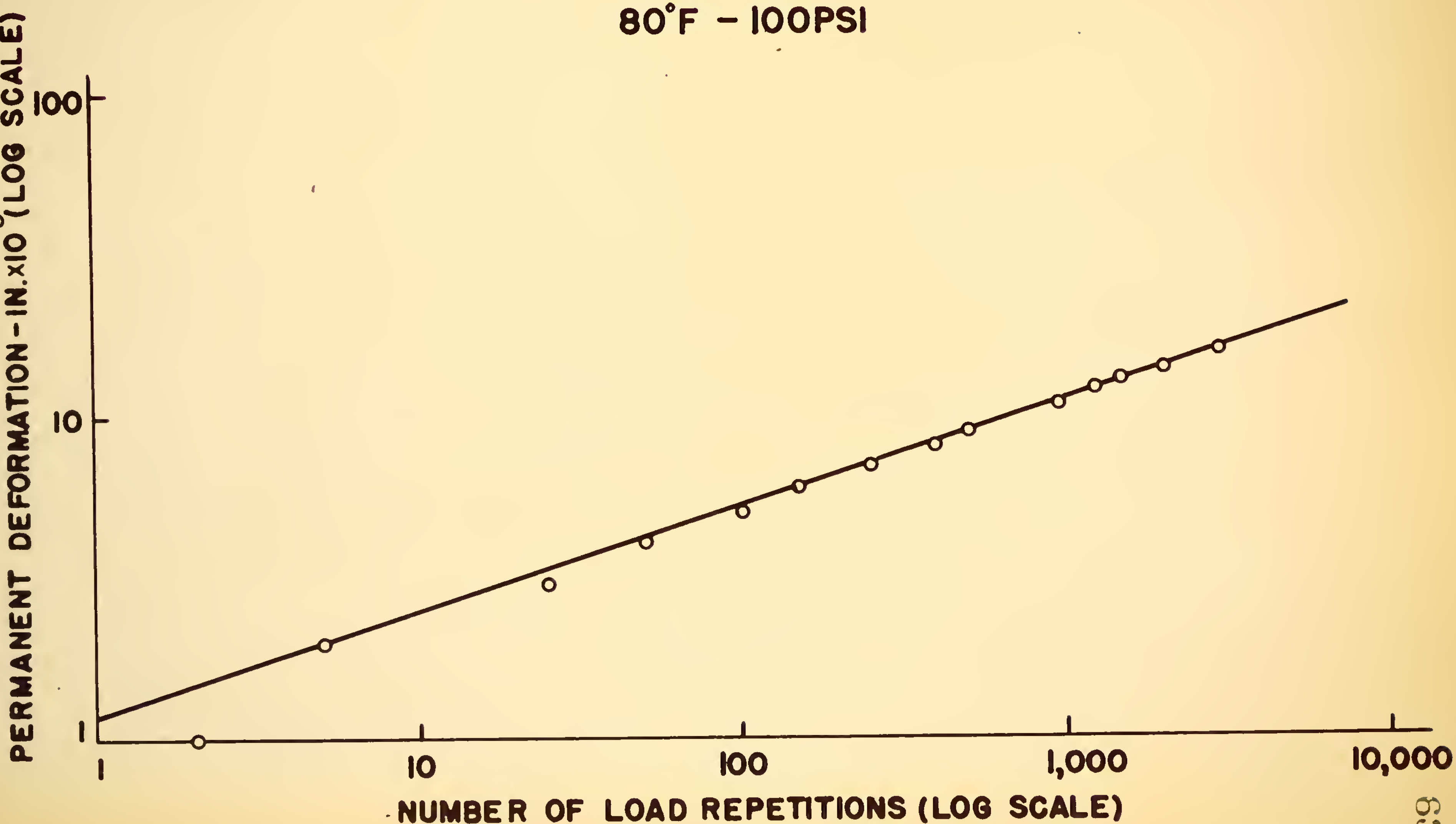

FIGURE 27

RAPID-CYCLE REPEATED LOAD TEST RESULTS

TWO AND FOUR-INCH PAVEMENT CORES

80°F - 200PSI

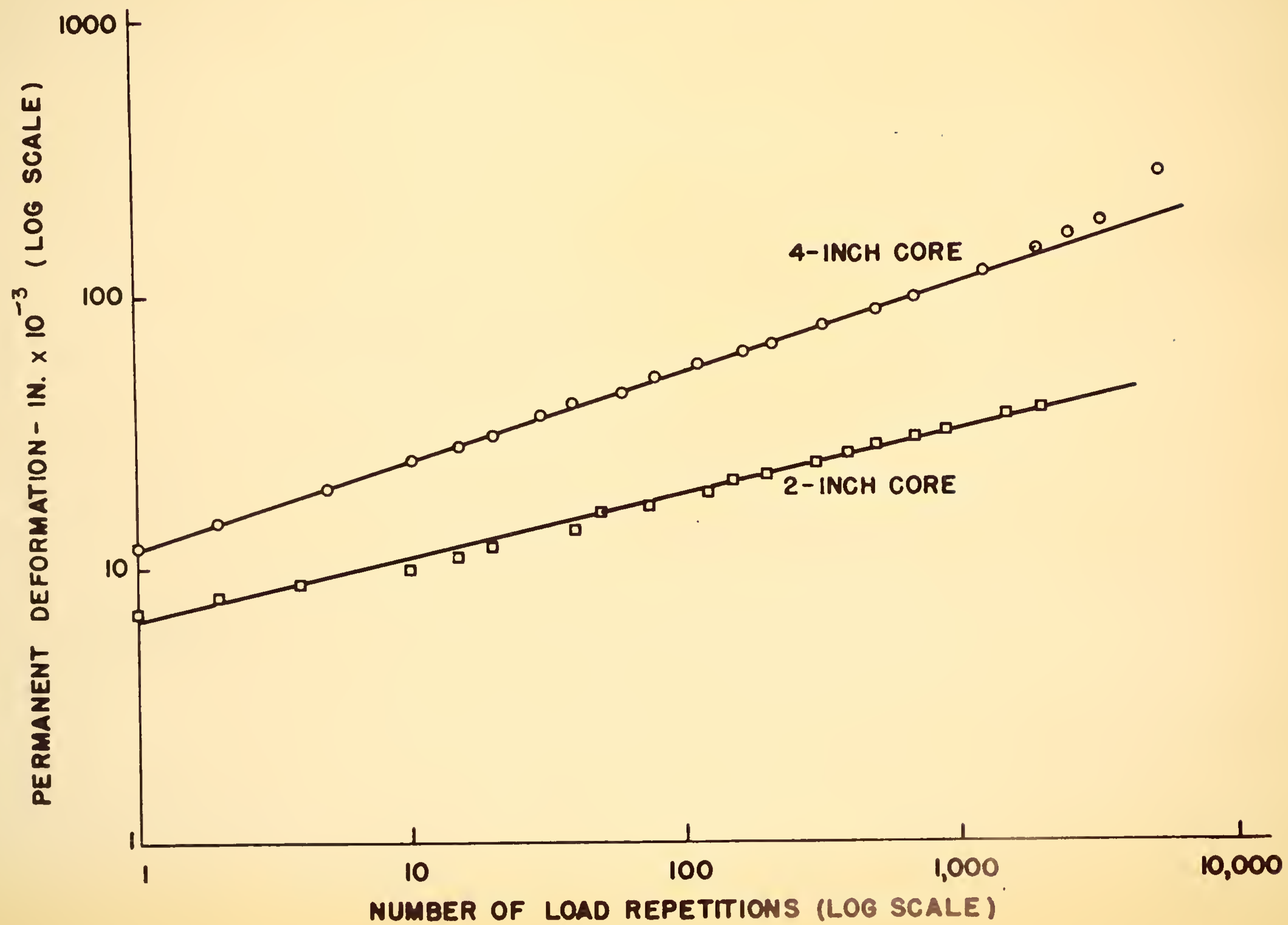

FIGURE 28

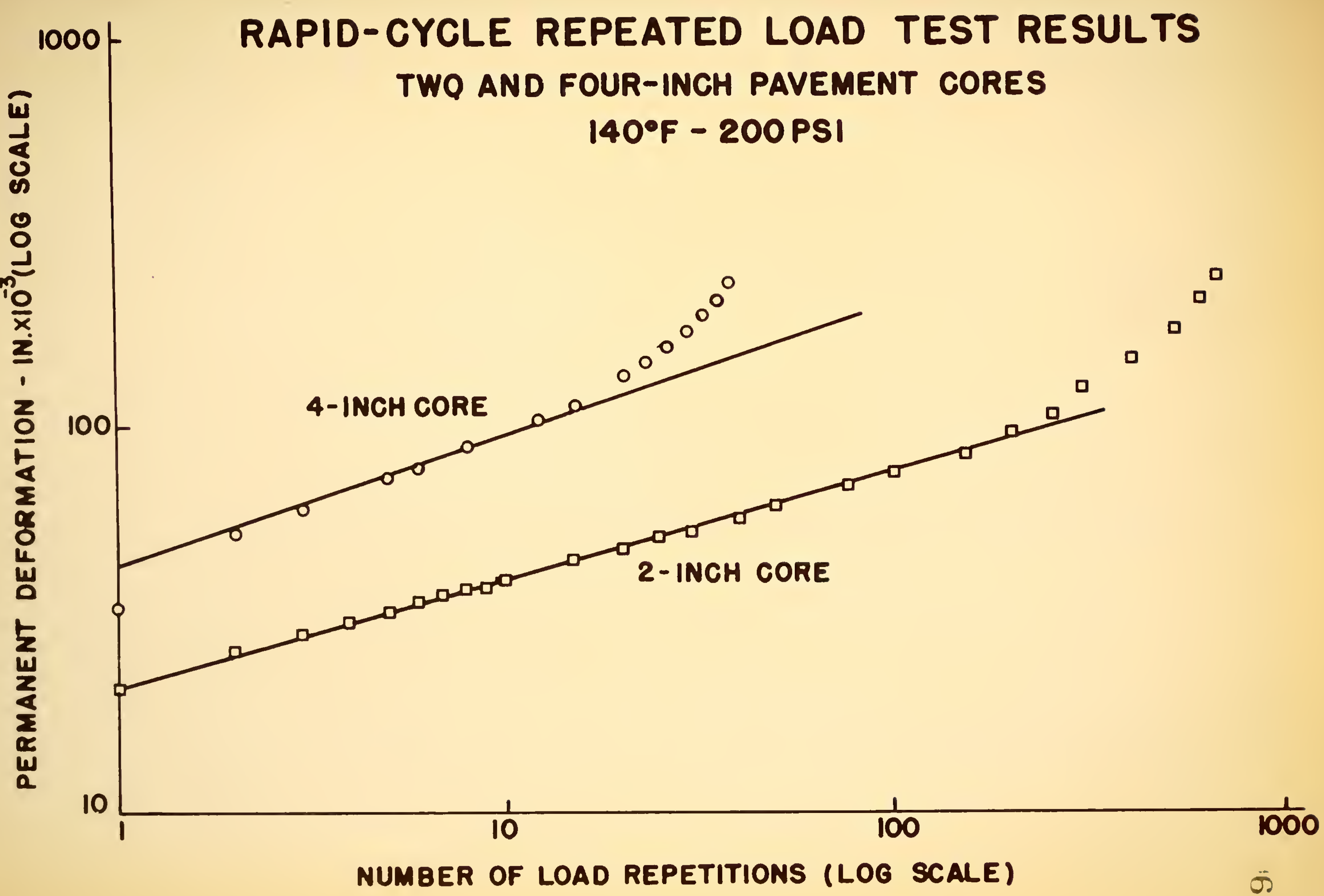
RAPID-CYCLE REPEATED LOAD TEST RESULTS
TWO AND FOUR-INCH PAVEMENT CORES
140°F - 200 PSI
4-INCH CORE
2-INCH CORE
PERMANENT DEFORMATION - IN.X10^-3 (LOG SCALE)
NUMBER OF LOAD REPETITIONS (LOG SCALE)
1000
100
10
1
10
100
1000

FIGURE 29

Table 1

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

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 mixtures of which these cores were composed may be about the same for the combinations of layer thickness 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 test 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, indicate that if the concept of an endurance limit for a bituminous-concrete mixture is considered 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 that a test in which the applied stress is of the order of that expected in service may be more realistic than one in which some percentage of the ultimate strength of the mixture, for a given set of test conditions, is 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. The regression equations that were derived in the section of the study on Strength-Rate-Temperature Relationships may be used 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 the percentage of the ultimate stress, for this type of test, that was being applied when using contact pressures of 100, 150, and 200 psi in the rapid-cycle repeated load tests can be estimated. This was done and the results are shown in Tables 2 and 3. It can be seen that some of the values listed in Table 3 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 the evaluation of the plastic nature and endurance properties of bituminous-aggregate mixtures. Much more work needs to be done with this test and we are continuing research along these lines. It is believed that the test and the method of analysis for the results that has been outlined contain the elements necessary to evaulate the plastic and load-carrying characteristics of a bituminous-concrete mixture.

Table 2

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

Core Height Inches	Ultimate Strength-psi	
	80°F	140°F
2	750	290
3	600	154
4	460	100

Table 3

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

Core Height Inches	Applied Stress psi	Percent Ultimate Strength 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

SUMMARY

In this presentation we have attemped to outline a problem existing in Indiana and one which is known to be present in other areas of the country. We have also attempted to show the laboratory apprcach to the solution of the problem, first from fundamental considerations of the stress-deformation characteristics of asphaltic mixtures as measured by various methods of test and second from the application of these test methods to the evaluation of bituminous concrete.

While the problem has not been solved in the rigorous technical sense, there have been discovered realtionships among the factors governing the load-carrying characteristics of bituminous mixtures that should provide a basis for the better understanding of the properties of the material. Specifically, we would like to point to the following:

A fundamental relationship between the compressive strength of a bituminous mixture and factors of rate of loading and temperature has been established for two basic types of bituminous mixtures and for a variety of test conditions.

The results of repeated load tests on bituminous mixtures suggest that this type of test might provide valuable information concerning the plastic nature of the mixture and, in addition, give a measure of its endurance limit.

Finally, the work on confinement in a thin layer indicates that the confining pressure that would be needed in a conventional triaxial test of bituminous concrete to produce a compressive strength equal to that which was developed by the mixture when tested in a relatively thin layer is equal to or greater than the unconfined compressive strength of the mixture.

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APPENDIX A

Regression Analyses

Unconfined Compression Test Data, Mixture I

Key to Coded and Transformed Data.

$X_0 = 10000 \log \log x_0$ $\qquad X_2 = x_2/20$

$X_1 = 1000 \log 1000\, x_1$ $\qquad X_3 = 50\, x_2 \log 1000\, x_1$

where x_0 = maximum unconfined compressive stress, psi

x_1 = rate of deformation, in./min.

x_2 = temperature, $^{\circ}F$

The multiple linear regression equation is as follows:

$$X_0 = B_0 + B_1 X_1 + B_2 X_2 + B_3 X_3 \quad \ldots \ldots \ldots \ldots (1)$$

where B_0, B_1, B_2, and B_3 are the unknown parameters.

Equation 1 may be written in terms of deviations from means.

$$(X_0 - \bar{X}_0) = B_1 (X_1 - \bar{X}_1) + B_2 (X_2 - \bar{X}_2) + B_3 (X_3 - \bar{X}_3). \quad \ldots (2)$$

If both sides of the equation are divided by the variance of X_0 and each term of the independent variables is multiplied and divided by its variance one gets the standard data form:

$$\frac{X_0 - \bar{X}_0}{\sigma X_0} = \frac{\sigma_1 B_1}{\sigma X_0} \frac{(X_1 - \bar{X}_1)}{\sigma_1} + \frac{\sigma_2 B_2}{\sigma X_0} \frac{(X_2 - \bar{X}_2)}{\sigma_2} + \frac{\sigma_3 B_3}{\sigma X_0} \frac{(X_3 - \bar{X}_3)}{\sigma_3} \quad \ldots (3)$$

Letting $\dfrac{(X_i - \bar{X}_i)}{\sigma X_i} = Z_i$ and $\dfrac{\sigma X_i}{\sigma X_0} B_i = B_i'$ one gets:

$$Z_0 = B_1' Z_1 + B_2' Z_2 + B_3' Z_3 \quad \ldots \ldots \ldots \ldots (4)$$

The problem now is to obtain the best sample estimates b_1', b_2', and b_3' of B_1', B_2', and B_3' such that the sum of squares of the deviations is a minimum. This is done by a least squares analysis using correlation coefficients r_{ij}'s (See reference (3) of the Bibliography) as a simplified means of calculation. The calculations themselves follow and equations used in the analysis are presented.

Observation		X_0	X_1	X_2	X_3
1		4233	301	2	602
2		1659	301	5	1505
3		72	301	7	2107
4		4486	1301	2	2602
5		2659	1301	5	6505
6		1316	1301	7	9107
7		4783	2301	2	4602
8		3222	2301	5	11505
9		2051	2301	7	16107
10		5072	3301	2	6602
11		3800	3301	5	16505
12		2848	3301	7	23017
13		5172	3987	2	7874
14		4036	3987	5	19685
n = 15		3131	3987	7	27559
Σu = sums		48540	33423	70	155974
$\bar{x}$ = means		3236	2228.2	4.666	10398.266
Σuv	X_0	188,185,390	123,208,144	190,298	506,740,174
	X_1		100,423,119	155,974	468,641,222
	X_2			390	868,998
	X_3				2,611,001,094
$\Sigma u \Sigma v$	X_0	157,075,440	108,156,828	226,520	504,731,864
	X_1		74,473,128.6	155,974	347,541,266
	X_2			326.667	727,878.67
	X_3				1,621,859.25
Sums of Product SP(u,v)	X_0	31,109,950	15,031,316	-36,222	2,008.310
	X_1		25,949,990	0	121,099,955
	X_2			63.33	141,119
	X_3				989,141,849

where $\Sigma u = \Sigma Xij$ $\qquad$ $\Sigma uv = \Sigma Xij \cdot Xij$

$i = 0, 1, 2, 3$ $\qquad$ $\frac{\Sigma u \Sigma v}{n} = \frac{(\Sigma Xij)\ (\Sigma Xij)}{n}$, $\quad SP\ (u, v) = \Sigma uv - \frac{(\Sigma u \Sigma v)}{n}$

$j = 1, \text{------} n$

The correlation coefficients, the r_{ij}'s, are calculated from the formula:

$$r_{ij} = \frac{SP\ (X_iX_j)}{\left[SP\ (X_iX_i)\ \ SP\ (X_jX_j)\right]^{1/2}}$$

For this analysis:

$$r_{01} = 0.5297 \qquad r_{12} = 0$$
$$r_{02} = -0.8182 \qquad r_{13} = 0.7559$$
$$r_{03} = 0.0114 \qquad r_{23} = 0.5653$$

From the above results one can establish a system of three linear equations and three unknowns. (See reference (18) of the Bibliography)

$$1\ b_1' + 0\ b_2' + 0.7559\ b_3' = 0.5297$$
$$0\ b_1' + 1\ b_2' + 0.5653\ b_3' = -0.8182$$
$$0.7559\ b_1' + 0.5653\ b_2' + 1\ b_3' = 0.0114$$

$$b_1' = 0.0205,\ b_2' = -1.1990,\ b_3' = 0.6737$$

Including those values in equation 4 one obtains:

$$Z = 0.0205\ \frac{(X_1 - \bar{X}_1)}{\sigma X_1} - 1.199\ \frac{(X_2 - \bar{X}_2)}{\sigma X_2} + 0.6737\ \frac{(X_3 - \bar{X}_3)}{\sigma X_3}$$

By making the proper substitutions in the above expression one can obtain equation 1 as follows:

$$X_0 = 5876.33 + 0.0224\ X_1 - 842.69\ X_2 + 0.1195\ X_3$$

Sum of squares due to regression $= b_1 SP\ (X_1\ X_0) + b_2 SP(X_2\ X_0) + b_3\ SP\ (X_3\ X_0)$

$$SSR = 0.0224\ (15{,}051{,}316) - 842.69\ (-36{,}222) + 0.1195\ (2{,}008{,}310)$$
$$= 31{,}101{,}059.7$$

Total Sum of Squares = 31,109,950

R^2 has been defined as the proportion of the sum of squares of the dependent variable which is explained by the multiple regression equation.

$$R^2 = \frac{\text{sum of squares attributable to regression}}{\text{total sum of squares}}$$

For this analysis: $R^2_{0.123} = \dfrac{31,101,059}{31,109,950} = 0.997$

Also $R^2_{0.123} = r^2_{01} + r^2_{02} + r^2_{03}$

This relationship was used to determine the effect of dropping out variables from the study. Thus:

$$R^2_{0.12} = (0.5297)\ (0.5297) + (0.8182)\ (0.8182) = 0.95$$

and $R^2_{0.1} = (0.5297)\ (0.5297) = 0.28$

Unconfined Compression Test
Indiana AH Type B Surface Mixture

All that was desired in this study was to obtain the raw data prediction equation from a limited number of results. The calculations and procedures are the same as for Mixture I.

Key to Coded and Transformed Data.

$X_0 = \log\log x_0$ $\qquad$ $X_2 = x_2$

$X_1 = \log 1000\, x_1$ $\qquad$ $X_3 = x_2 \log 1000\, x_1$

	X_0	X_1	X_2	X_3
	.42004	.301	40	12.04
	.17064	.301	140	42.14
	.47978	2.301	40	92.04
	.26661	2.301	140	322.14
Σu	1.33707	5.204	360	468.36
u	.33343	1.301	90	117.09

Σuv	X_0	.506821	1.89524	97.2078	142.29275
	X_1		10.7704	468.36	609.33636
	X_2			42400.	55162.40
	X_3				114,166.2824

$\frac{\Sigma u \Sigma v}{n}$	X_0	.446939	1.739528	120.3363	156.55753
	X_1		6.7704	468.36	609.33636
	X_2			32400.	42152.40
	X_3				54840.272

SP(u,v)	X_0	.059882	.155712	-23.1285	-14.26478
	X_1		4.0000	0	360.00
	X_2			10000.	13010.
	X_3				59326.01

This gives the following three equations and three unknowns:

$$4\, b_1 + 0\, b_2 + 360\, b_3 = 0.1557$$

$$0\, b_1 + 10{,}000\, b_2 + 13{,}010\, b_3 = -23.129$$

$$360\, b_1 + 13{,}010\, b_2 + 59{,}326.01\, b_3 = -14.265$$

Solving these equations, one obtains: $b_1 = .02264$, $b_2 = .002548$, $b_3 = .00018097$.

This gives: $X_0 = 0.33343 + 0.02264\ (X_1 - 1.301) - 0.002548\ (X_2 - 90) + 0.00018097\ (X_3 - 117.09)$.

Collecting terms: $X_0 = 0.51201 + 0.02264\ X_1 - 0.002548\ X_2 + 0.00018097\ X_3$.

Substituting in this equation, the maximum unconfined compressive stress is predicted for the following test conditions:

40°F	0.02 in./min.	= 648 psi.
100°F	0.002 in./min.	= 72 psi.
	0.02 in./min.	= 110 psi.
	0.2 in./min.	= 175 psi.
140°F	0.02 in./min.	= 45 psi.

Confined Compression Test Data, Mixture II - 15 psi

All that was desired in this study was to obtain the raw data prediction equation from a limited number of results. The calculations and procedures are the same as for Mixture I.

Key to Coded and Transformed Data.

$X_0 = \log \log x_0$ $\qquad X_2 = x_2$

$X_1 = \log 1000\, x_1$ $\qquad X_3 = x_2 \log 1000\, x_1$

	X_0	X_1	X_2	X_3
	.41348	.301	40	12.04
	.29515	.301	140	42.14
	.48193	2.301	40	92.04
	.30472	2.301	140	322.14
Σu	1.49528	5.204	360	468.36
$\bar{u}$	.37382	1.301	90	117.09

		X_0	X_1	X_2	X_3
Σuv	X_0	.58319004	2.023379	119.7982	159.93526
	X_1		10.7704	468.36	969.33636
	X_2			42400.	55162.04
	X_3				114,156.2824
$\frac{\Sigma u \Sigma v}{n}$	X_0	.558965	1.945359	134.5752	175.08234
	X_1		6.770404	468.36	609.33636
	X_2			32400.	42152.40
	X_3				54840.272
SP(u,v)	X_0	.024225	.07802	-14.777	-15.14708
	X_1		4.0000	0	360.00
	X_2			10000.	13010.0
	X_3				59326.01

This gives the following three equations and three unknowns:

$$4\, b_1 + 0\, b_2 + 360\, b_3 = 0.07802$$

$$0\, b_1 + 10{,}000\, b_2 + 13{,}010\, b_3 = -14.777$$

$$360\, b_1 + 13{,}010\, b_2 + 59{,}326.01\, b_3 = -15.14708$$

Solving these equations, one obtains: $b_1 = 0.046027$, $b_2 = -0.001094$, $b_3 = -0.0002263$.

This gives: $X_0 \cong 0.37382 + 0.0460277\ (X_1 - 1.301) - 0.0010941\ (X_2 - 90) -$

$0.000294697\ (X_3 - 117.09)$.

Collecting terms: $X_0 \cong 0.44515 + 0.0460277\ X_1 - 0.0010941\ X_2 -$

$0.000294697\ X_3$.

Substituting in this equation, the maximum confined compressive stress at 15 psi confing pressure us predicted for the following test conditions:

40°F	0.02 in./min.	≅ 620 psi
100°F	0.002 in./min.	≅ 155 psi
	0.02 in./min.	= 189 psi
	0.2 in./min.	≅ 232 psi
140°F	0.02 in./min.	≅ 97 psi

Confined Compression Test, Mixture II - 30 psi

All that was desired in this study was to obtain the raw data prediction equation from a limited number of results. The calculations and procedures are the same as for Mixture I.

Key to Coded and Transformed Data.

$$X_0 = \log \log x_0 \qquad X_2 = x_2$$

$$X_1 = \log 1000\, x_1 \qquad X_3 = x_2 \log 1000\, x_1$$

	X_0	X_1	X_2	X_3
	.42518	.301	40	12.04
	.34157	.301	140	42.14
	.48308	2.301	40	92.04
	.34988	2.301	140	322.14
Σu	1.59971	5.204	360	468.36
$\bar{u}$	.39993	1.301	90	117.09

Σuv	X_0	X_1	X_2	X_3
X_0	.653204	2.147432	133.1334	176.68595
X_1		10.7704	468.36	969.33636
X_2			42400.	55162.40
X_3				114,166.2824

$\frac{\Sigma u \Sigma v}{n}$	X_0	X_1	X_2	X_3
X_0	.63977	2.081210	143.9739	187.31004
X_1		6.770404	468.36	609.33636
X_2			32400.	42152.40
X_3				54840.272

SP(u,v)	X_0	X_1	X_2	X_3
X_0	.013434	.066222	-10.8405	-10.62409
X_1		4.0000	0	360.000
X_2			10000.	13010.
X_3				59326.01

This gives the following three equations and three unknowns:

$$4\, b_1 + 0\, b_2 \quad 360 \quad b_3 = 0.066222$$

$$0\, b_1 + 10{,}000\, b_2 + 13{,}010 \quad b_3 = -10.8405$$

$$360\, b_1 + 13{,}010\, b_2 + 59{,}326.01\, b_3 = -10.62409$$

Solving these equations, one obtains: $b_1 = 0.0388725$, $b_2 = -0.0007614$, $b_3 = -0.00024795$.

This gives: $X_0 = 0.39993 + 0.0388725 (X_1 - 1.301) - 0.0007614 (X_2 - 90) - 0.00024795 (X_3 - 117.09)$.

Collecting terms: $X_0 = 0.44542 + 0.0388725 X_1 - 0.0007614 X_2 - 0.00024795 X_3$.

Substituting in this equation, the maximum confined compressive stress at 30 psi confining pressure is predicted for the following test conditions:

40°F	0.02 in./min.	=	684 psi.
100°F	0.002 in./min.	=	230 psi.
	0.02 in./min.	=	276 psi.
	0.2 in./min.	=	332 psi.
140°F	0.02 in./min.	=	162 psi.

APPENDIX B

Tabulations of Data

Table 4

Unconfined Compression Test Results for Mixture I

Temperature °F	Rate of Deformation in./min.				
	0.002	0.02	0.2	2.0	8.65
	Maximum Compressive Stress (psi)				
40	445	660	1045	1705	2015
100	29	71	128	257	350
140	10	23	41	86	116

Table 5

Unconfined Compression Test Results for Indiana AH Type B Mixture

Temperature °F	Rate of Deformation - in./min.		
	0.002	0.02	0.2
		Maximum Compressive Stress (psi)	
40	427[1]	648[2] 708[3]	1043[1]
100	72[2] 64[3]	110[2] 96[3]	175[2] 145[3]
140	32[1]	45[2] 34[3]	70[1]

1 Values used in establishing mathmatical relationship for Indiana AH Type B Surface Mixture.

2 Calculated values.

3 Observed test values taken after the relationship was established.

Table 6

Confined Compression Test Results for Mixture II

Temperature °F	Lateral Pressure - psi	Rate of Deformation - in./min. 0.002	0.02	0.2
		Maximum Compressive Stress (psi)		
40	15	390	609	1080
	30	459	680	1100
100	15	103	114	177
	30	168	195	222
140	15	94	96	104
	30	157	170	173

Table 7

Unconfined, Repeated Load Test Results for Mixture II

Temperature °F	Nominal % max. Comp. Stress	Rate of Deformation - in./min. 0.002	0.02	0.2
		Load Repetitions Necessary to Cause Failure[1]		
	75	3	4	5
40	50	7	7	7
	25	**	**	28
	75	4[2]	3	4[4]
100	50	7	5	7
	25	**[3]	**	**
	75	3	2[4]	5[4]
140	50	6	5	7
	25	**	**	35

1 For definition of failure, see text.

** Number of load repetitions used during test did not cause failure.

2 67% maximum compressive stress was used.

3 33% of maximum compressive stress was used.

4 60% of maximum compressive stress was used.

Table 8

Confined Repeated Load Test Results for Mixture II

Temperature °F	Lateral Pressure - psi	% Max. Comp. Stress	Rate of Deformation - in./min. 0.002	0.02	0.2
			Load Repetitions Necessary to Cause Failure.[1]		
40	15	50	5		5
	15	25	**		**
	30	50	7		5
	30	25	**		**
100	15	75		5	
	15	50		9	
	15	25		**	
	30	75		4	
	30	50		3	
	30	25		**	
140	15	50	6		7
	15	25	**		**
	30	50	5		6
	30	25	**		**

[1] For definition of failure, see text.

**Number of load repetitions used during test did not cause failure.

Table 9

Triaxial Test Results for Indiana AH Type B Mixture

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

Table 10

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

Table 11

Results of Strength-Rate-Temperature Series
Laboratory-Compacted Specimens-Indiana AH Type B Mixture

Temp. °F	Specimen	Total Load At Failure - Pounds: Vibrated, Rate of Deformation - in/min. 0.2	Vibrated 0.02	Double-Plunger Compacted 0.2	Double-Plunger Compacted 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

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

Table 13

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

	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 14

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 15

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	23		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 16

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	34
200	27	21	46
300		26	
400	36	27	58
600			65
700	42	33	
1000	46	36	
1500		42	
2000	53	48	95
2800		53	

Table 17

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 18

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 19

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		