Fatigue Life and Permanent Deformation Characteristics of Asphalt Concrete Mixes

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New laboratory and field relationships between asphalt mix variables and their fatigue life and permanent deformation characteristics are presented and discussed. It is shown that the laboratory fatigue life and plastic deformation of laboratory specimens can be predicted from a knowledge of several parameters pertaining to the compacted asphalt mix, magnitude of the applied load, and test temperature. Predictions of the fatigue life and rut depth of asphalt pavements require, in addition to the above parameters, the compressive and tensile strains of the asphalt course, as well as the moduli of the different pavement layers. During the course of the laboratory investigation, a new indirect tensile test apparatus was designed, fabricated, and used to conduct cyclic load indirect tensile tests using Marshall-type specimens and various asphalt concrete mixes. The test results indicate that the indirect tensile test can be used to characterize the fatigue life of asphalt concrete mixes.

The design of flexible pavements has rapidly evolved from empirical and semi-empirical procedures to design methods based on elastic and/or viscoelastic theories (*1–7*). Today, many highway agencies use such methods in one form or another to design new, reconstructed, and/or overlaid asphalt pavements. This use requires a thorough knowledge of the basic structural properties (resilient characteristics, fatigue life, and plastic deformation) of the asphalt layer that are functions of the asphalt mix variables and the properties of the various layers in the pavements (*8–13*).

Several tests and test equipment have been developed and used in laboratories to evaluate the structural properties of asphalt mixes (*14–27*). It was found that (regardless of the complexity of the tests, test procedures, and test equipment) different tests yield different results and that the test results are difficult to reproduce (*12*). Further, existing asphalt concrete mix design procedures are based on parameters that do not necessarily have any relationship to the structural design of asphalt pavements and their fatigue lives (*15*,*12*).

EXPERIMENTAL PROGRAM

Recognizing the need to tailor the asphalt mix design procedures to the optimum structural properties of the mix and to be able to obtain these parameters from simple tests, a research project sponsored by FHWA was undertaken at the Department of Civil and Environmental Engineering at Michigan State University (MSU). The objectives of the program included the following:

1. The selection of a simple test and test procedure that would allow the highway engineer to determine the fundamental engineering properties required for the structural design of asphalt pavements.
2. A study of the repeatability of the test results and the number of tests required to reliably obtain the structural properties of the asphalt materials.

Several laboratory tests and test procedures were used, including triaxial (constant and variable cyclic loads), cyclic flexural tests, Marshall tests, indirect tensile tests (constant and variable cyclic loads), and creep tests. The initial test results indicate the following:

1. The repeatability of test results is poor.
2. The material properties obtained from the different tests are substantially different.
3. The results from the indirect tensile test are the most promising, although they are not consistent.

The last observation was made after examining the results of 24 tests (12 tests at a cyclic load of 500 lb and 12 tests at 100 lb) that were conducted, in triplicate, using existing (Schmidt) apparatus. It was found that the inconsistency of the test results was related to equipment problems rather than to the test mode (*28–31*). In recognition of these problems, a new indirect tensile test apparatus was designed at MSU and fabricated by the Michigan Department of Transportation (MDOT). The new apparatus was capable of measuring the specimen deformation in three directions (*30–31*). Further, test results obtained using the new apparatus were very consistent and reproducible (*27–31*). It was found that the maximum difference in the test results for any triplicate was 7 percent. Nevertheless, the new apparatus was used to characterize the structural properties and fatigue lives of various asphalt mixes.

LABORATORY TESTS

Three tests were conducted using the new indirect tensile test apparatus: indirect tensile tests, indirect constant peak cyclic load tests, and indirect variable peak cyclic load tests.

The indirect tensile tests (INTT) used a standard Marshall

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loading frame and deformation rate. In this test mode, some of the test specimens were conditioned as standard Marshall specimens. Other specimens were dry-conditioned and tested at 60, 25, and 5°C (140, 77, and 40°F).

The indirect constant peak cyclic load (INCCL) tests used an MTS hydraulic system. In this test, the specimens were subjected to a constant, sustained load followed by a constant peak cyclic load of 500 lb (i.e., stress-controlled tests) applied at a frequency of two cycles per second with a loading time of 0.1 sec and a relaxation period of 0.4 sec. Some of the test specimens were subjected to a maximum of 500,000 cycles. Measurements of the elastic, total, and plastic (permanent) deformations were collected along the vertical and horizontal diameters, and along the thickness of the specimen. The data were then analyzed to obtain the resilient and plastic characteristics of the specimens and their fatigue lives.

Indirect variable peak cyclic load (INVCL) tests used an MTS hydraulic system in the stress-controlled mode. Basically, the test procedure was the same as that of the INCCL except that, after the application of the sustained load, the specimen was subjected to 100-, 200-, and 500-lb peak cyclic loads, with each load being applied for only 1,000 cycles. It should be noted that a magnitude of the cyclic load of 500 lb implies that the cyclic stresses at the center of the specimen were 239 psi in compression and 80 psi in tension; for 200 lb, cyclic stresses were 96 and 32 psi; and for 100 lb, stresses were 47.8 and 16 psi. Also, for all three magnitudes (100, 200, and 500 lb), the shear stresses at the center of the specimen were zero.

One hundred and fifty specimens were tested in the INTT, 75 in the INCCL, and 75 in the INVCL methods. A complete tabulation of the test results has been given by Baladi (*31*).

It should be noted that for the remainder of this paper, the term *sample* is used to describe a cylindrical sample (8.5 in. high and 4 in. in diameter) that was prepared at a certain density and percent air voids. After preparation, each cylindrical sample was sawed into three (triplicate) test specimens (each 2.5 in. high and 4 in. in diameter).

TEST SPECIMENS

One hundred and twenty-five samples (375 specimens) were made and tested using the new indirect tensile test apparatus (75 for each of the INCCL and INVCL tests, and 150 for the INTT). The samples were made using the following materials:

1. Three types of aggregate (crushed and angular limestone, relatively rounded river-deposited natural aggregate, and a mix of 50 percent by weight per sieve of the crushed limestones and natural aggregates),
2. Fly ash mineral filler,
3. Two aggregate gradations (see Figure 1), and
4. Three viscosity-graded asphalt cements (AC-10, AC-5, and AC-2.5).

For each material combination, a constant percent asphalt content was used (the percent asphalt content at 3 percent air voids as determined from the standard Marshall mix design). The test samples were compacted near three values of the percent air void (3, 5, and 7) by varying the foot pressure and the number of tampings of a kneading compactor. For each material combination and percent air void, a cylindrical sample 10.16 cm in diameter and 22 cm high (4 in. by 8.5 in.)

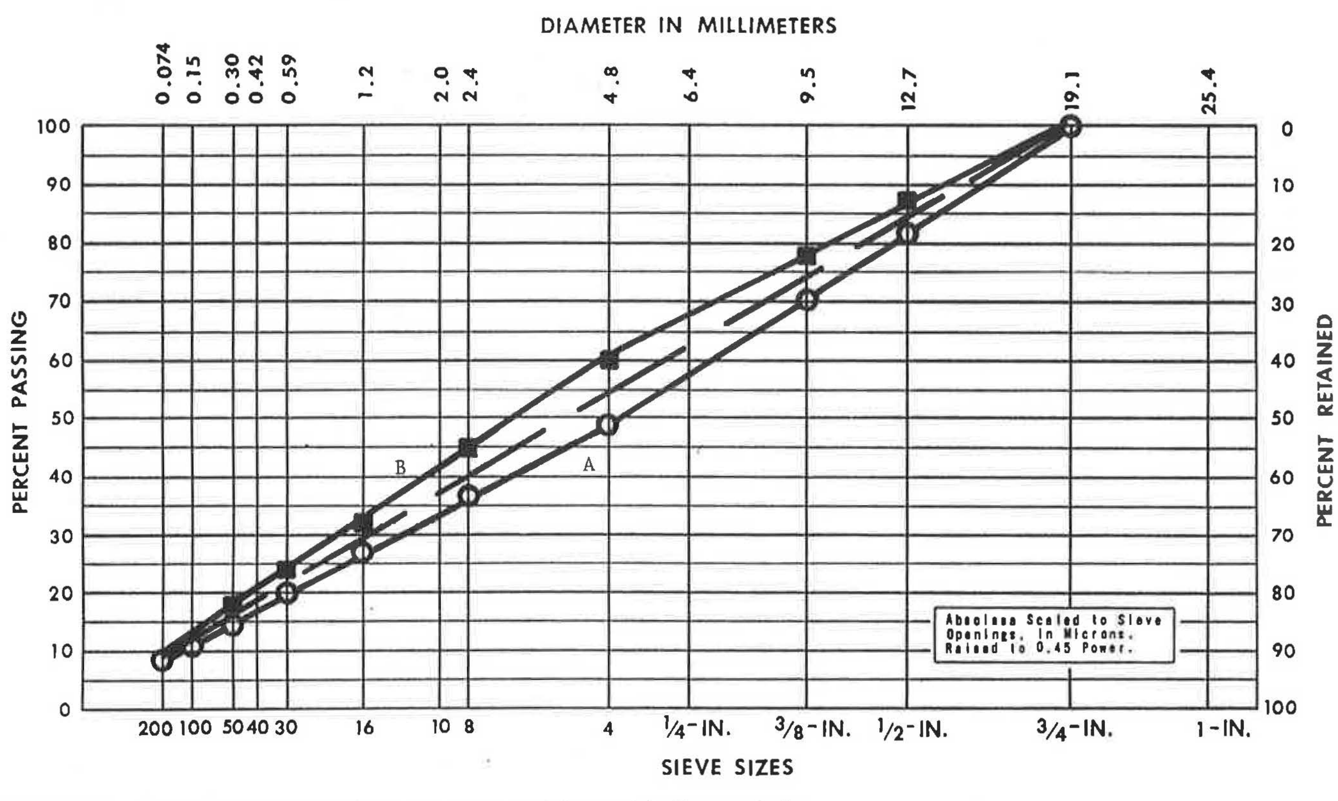

FIGURE 1 Gradations A and B of the aggregates and the straight-line gradation.

was made. Later, the sample was sawed into three 6.3 cm high (2.5 in.) Marshall-size specimens. The three specimens (a triplicate) were then tested under the same conditions (test temperature, cyclic load, and test type) using the new indirect tensile test apparatus.

TEST RESULTS AND ANALYSIS

Indirect Tensile Tests

As noted previously, 150 Marshall-type specimens were tested to failure using the new indirect tensile test apparatus and the standard Marshall loading frame and deformation rate. During the tests, the applied load and the total deformations along the vertical and horizontal diameters, and along the thickness of the specimen, were measured. The maximum values of the applied load at failure (*P*) were used to obtain the compressive and tensile strengths of the specimens (*24–26*). The measured values of the deformation at failure (*DH*) along the horizontal diameter represented the maximum tensile deformation to which the specimen could be subjected to prior to failure. Figure 2 depicts typical plots of the horizontal deformation at failure versus the percent air voids of the INTT specimens tested at 77°F. For all tests at 40, 77, and 140°F, these values were analyzed and statistically correlated to the specimen, mix, and test variables. The following equation was obtained:

$$\ln(DH) = 5.887 + (0.06931)(\mathrm{AV}) + (0.0003901)(\mathrm{KV}) + (0.03382)(\mathrm{ANG})$$

$$R^2 = 0.925,\ \mathrm{SE} = 0.003 \qquad (1)$$

where

ln = natural logarithmic operator,
AV = percent air voids (AV = 3 to 7),
KV = kinematic viscosity at 275°F (AASHTO T-201) (centistokes),
ANG = aggregate angularity,
R^2 = coefficient of correlation, and
SE = standard error.

Figure 3 shows the measured and calculated *DH* values using Equation 1. The line of equality between measured and calculated values is also shown in the figure. It should be noted that the maximum percent difference between the arithmetic values of the calculated (using Equation 1) and measured data is 37. The majority of the estimated values, however, are within 7 percent. The significance of this equation is discussed later. Two important aspects can be deduced from Equation 1:

1. The total horizontal deformation at failure of a Marshall-type specimen is a function of AV, KV, and ANG. Increasing AV from 3 to 7 percent results in an increase in *DH* by a factor of 1.49. Increasing KV from 159 to 270 centistokes yields an increase in *DH* by a factor of 1.04. Finally, using angular aggregate causes an increase in the value of *DH* by a factor of 1.07 relative to rounded aggregates.
2. The total deformations at failure are independent of the test temperature. However, the magnitude of the applied loads at failure for the higher temperature specimens was much lower than that at the lower temperatures.

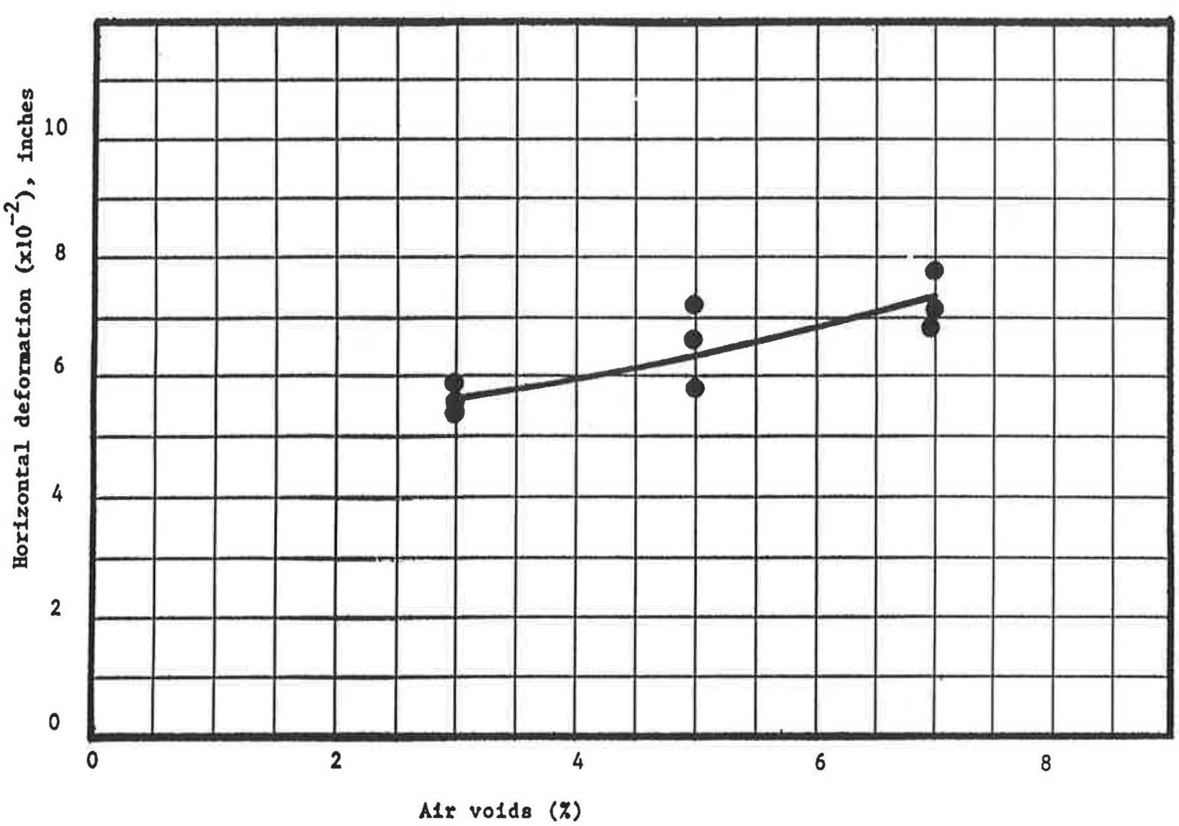

FIGURE 2 Typical plots of the horizontal deformation of INTT specimens versus the percent air voids in the compacted asphalt mix.

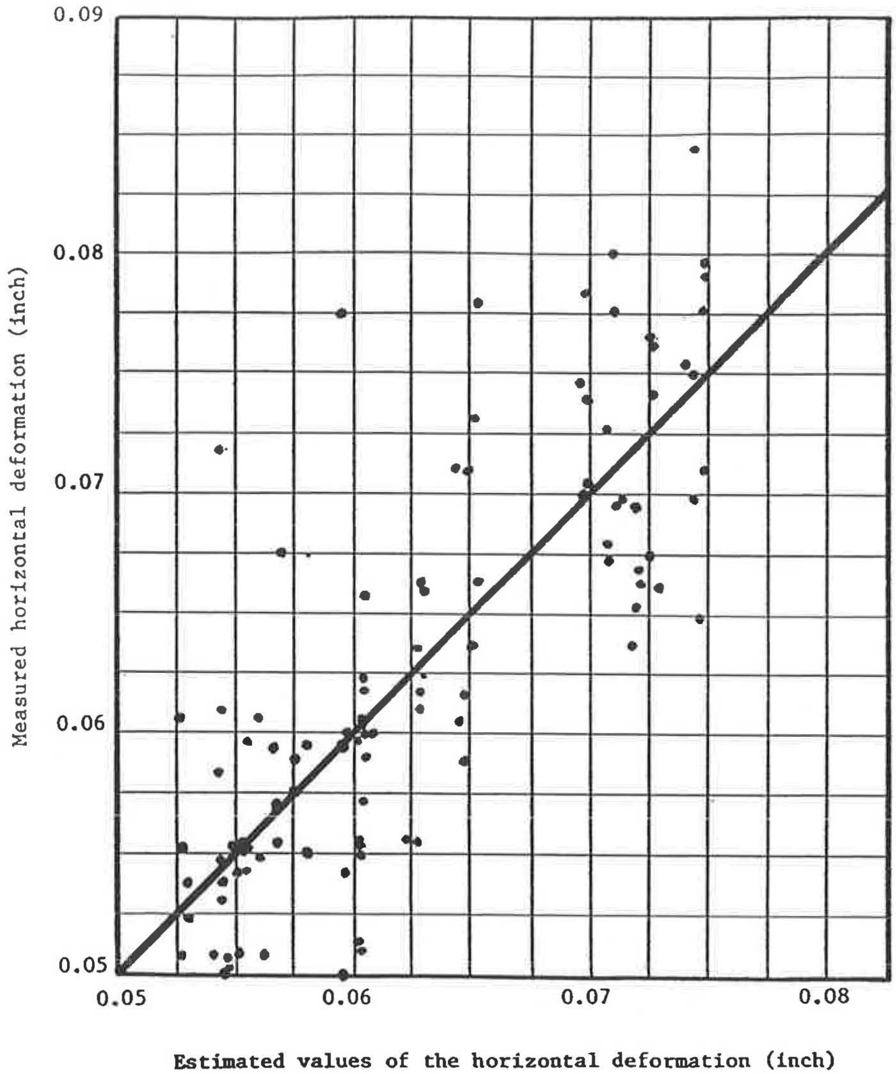

FIGURE 3 Measured and estimated values of the horizontal deformation at failure for the INTT specimens.

Indirect Constant and Variable Cyclic Load Tests

Seventy-five Marshall-type specimens were tested to failure in each of the INCCL and INVCL tests using the new indirect tensile test apparatus. During the tests, the applied sustained and cyclic loads; the number of load applications; and the elastic, total, and plastic deformations (CD,*s*) along the horizontal diameter (CD1), the vertical diameter (CD2), and the thickness of the specimens (CD3) were measured. The test results are reported in Baladi (*31*). Typical plots of CD2 and CD1, versus the number of load applications for three values of the percent air voids of the INCCL test specimens that were tested at 77°F, are shown in Figures 4 and 5, respectively. It should be noted that each data point in these figures represents a triplicate.

The cumulative plastic deformations (CD2 and CD1) measured (respectively) along the horizontal and vertical diameters of both the INCCL and INVCL test specimens were analyzed and statistically correlated to the mix, specimen, and test variables. The objective of the analysis was twofold:

1. Study the tensile plastic deformations (CD2) along the horizontal diameter and determine their relationships to the fatigue lives of the test specimens.
2. Study the compressive plastic deformations (CD1) along the vertical diameter and determine their relationships to the rut depth of actual pavements.

Nevertheless, Equations 2 and 3 are the resulting statistical equations.

$$\begin{aligned}\ln(\mathrm{CD2}) = {} & -12.789 + (0.07034)(\mathrm{TT}) \\ & + (0.5018)[\ln(N)] + (1.154)[\ln(\mathrm{CL})] \\ & + (0.3215)(\mathrm{AV}) + (0.001026)(\mathrm{KV}) \\ & R^2 = 0.967,\ \mathrm{SE} = 0.299 \qquad (2)\end{aligned}$$

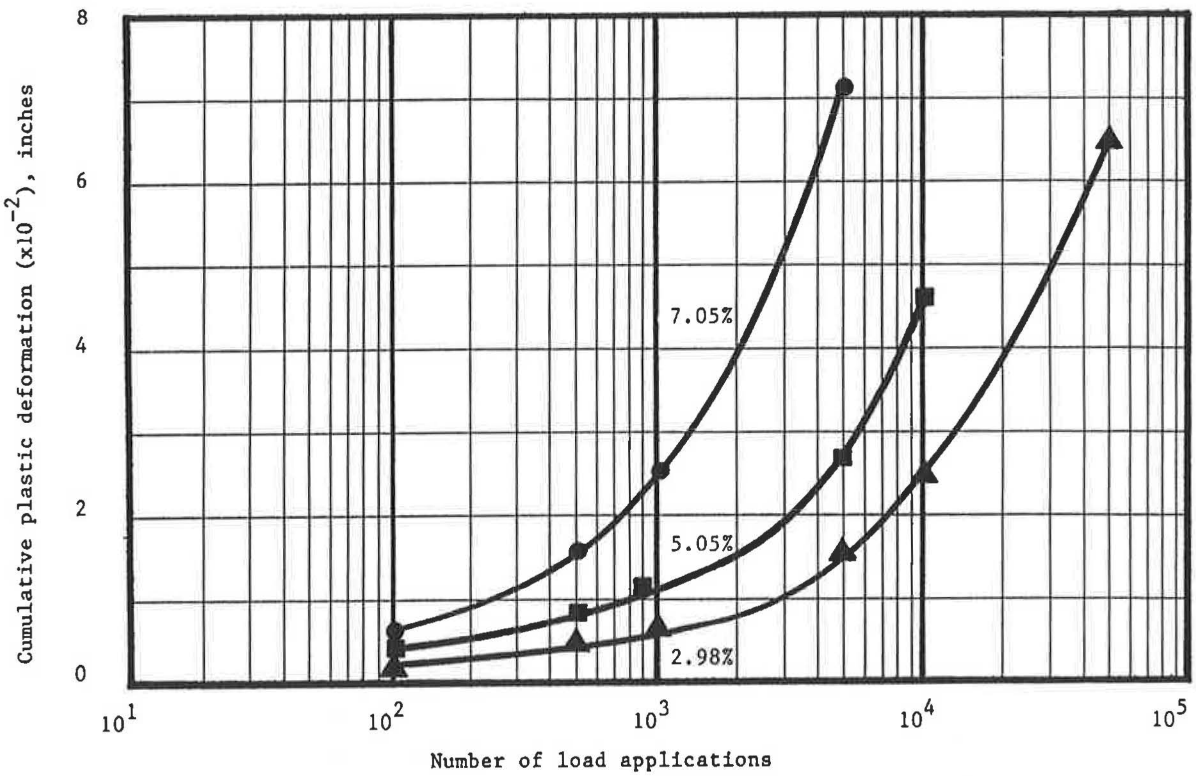

FIGURE 4 Typical plots of the horizontal cumulative plastic deformations of the INCCL test specimens versus the number of load applications for three values of the percent air voids.

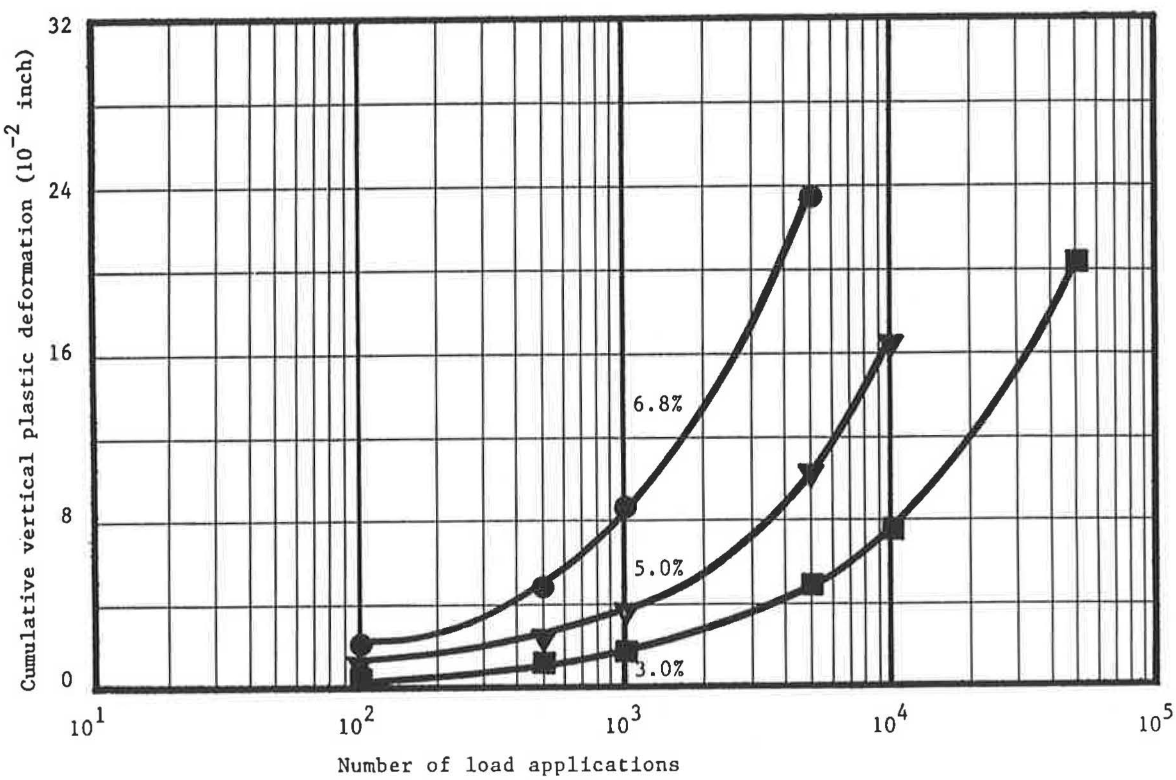

FIGURE 5 Typical plots of the vertical cumulative plastic deformations of the INCCL test specimens versus the number of load applications for three values of the percent air voids.

$$\ln(\mathrm{CD1}) = -11.615 + (0.07028)(\mathrm{TT}) + (0.5000)[\ln(N)] + (1.148)[\ln(\mathrm{CL})] + (0.3326)(\mathrm{AV}) - (0.001007)(\mathrm{KV})$$

$$R^2 = 0.96,\ \mathrm{SE} = 0.36 \qquad (3)$$

where

CD2 = plastic deformation along the horizontal diameter (inch $\times 10^{-4}$),
CD1 = plastic deformation along the vertical diameter (inch $\times 10^{-4}$),
TT = test temperature (°F),
N = number of load applications, and
CL = cyclic load (lb).

It should be noted that the independent variables in Equations 2 and 3 are listed in the order of their significance (e.g., TT is the most significant variable, while KV is the least significant). A sensitivity analysis of the arithmetic (not logarithmic) values of CD2 and CD1 to the range of the independent variables was made, and the results are reported by Baladi (*31*). For completion, a summary of this analysis is presented below.

Increasing TT from 40 to 77°F increases the values of CD2 and CD1 by a factor of about 13.5. Higher number of load applications yield higher values of CD2 and CD1. Increasing the magnitude of the cyclic load from 100 to 500 lb increases the values of CD2 and CD1 by a factor of about 6.4 (i.e., nonlinear material). Increasing AV from 3 to 7 percent causes both CD2 and CD1 to increase by a factor of about 3.8. Increasing KV from 159 to 270 centistokes decreases the values of CD2 and CD1 by a factor of about 1.12. Finally, aggregate angularity has an insignificant effect on the values of CD2 and CD1.

FATIGUE LIFE

Fatigue life data for asphalt mixes have been accumulated in large quantities since the early 1950s. Traditionally, these data are plotted as stress or strain amplitude versus the resulting life, commonly known as *S-N curves*. For asphalt mixes, as for most other materials, fatigue life increases steadily with decreasing stress or strain amplitude until the stress or strain level of the fatigue limit is reached; below which, the life apparently becomes infinitely long. In general, stresses at or below the fatigue limit cause only elastic strains. It should be emphasized that cyclic plastic strains are ultimately responsible for fatigue damage and the consequent fatigue failure. A perfectly elastic material will never experience fatigue damage regardless of the number of load applications.

Because asphalt mixes are weaker in tension than in compression, it was thought that measuring the plastic deformation along the horizontal diameter of the Marshall-type specimens subjected to cyclic loads may yield a measure of the fatigue life of the specimens. In doing so, a problem was encountered: a proper definition of the fatigue life of the test specimen. Currently, no standard definition of fatigue life of asphalt mixes is universally adopted although, several definitions are used by different investigators (*21–23*). Thus, a new definition was introduced prior to the analysis.

For the purpose of this study, the fatigue life of the INCCL test specimens is defined as follows: "Fatigue lives of the INCCL and INVCL test specimens are defined herein by the number of load applications at which the cumulative horizontal plastic deformation (CD2) along the horizontal diameter reaches a value equal to 95 percent of the total horizontal deformation failure (*DH*) of a compatible specimen tested to failure using the INTT mode."

It should be noted that this definition was established after visually examining the conditions of the INCCL and INVCL test specimens during and after each test. It was noted that a hair crack (along the vertical diameter) was initiated in the specimen (and failure occurred shortly after) when the value of CD2 reached a value equal to about 95 percent of the total horizontal deformation at failure of a compatible specimen tested using the INTT mode.

Using the above definition of fatigue life, the arithmetic version of the right-hand side of Equation 1 was multiplied by 0.95 and the results were substituted for CD2 in Equation 2. After arranging terms and substituting N with N_{FL} (the number of load applications at fatigue life), the following equation was obtained.

$$\ln(N_{FL}) = 36.631 - (0.1402)(\mathrm{TT}) - (2.300)[\ln(\mathrm{CL})] - (0.5095)(\mathrm{AV}) - (0.001306)(\mathrm{KV}) + (0.06403)(\mathrm{ANG}) \qquad (4)$$

Sensitivity analysis of Equation 4 yielded the following points:

1. The fatigue life of indirect tensile test specimens increases as the temperature decreases. Decreasing temperatures from 77 to 40°F increases the fatigue life by a factor of 179. This does not necessarily mean that higher temperatures cause more cracks. It simply means that, at lower temperatures (stiffer asphalt), the magnitude of the plastic deformation per one load application is lower than that at the higher temperatures. Lower plastic deformations result in a higher number of load applications to fatigue failure. It should be noted that lower temperatures cause shrinkage cracks in flexible pavements that are not load related. Temperature cracks were not investigated in this study.
2. Increasing the magnitude of the cyclic load from 100 to 500 lb causes a decrease in the fatigue life by a factor of about 40.
3. The fatigue life decreases as the percent air voids in the mix increases. Increasing AV from 3 to 4 percent yields a decrease in N_{FL} by a factor of 7.7.
4. Decreasing KV from 270 to 159 centistokes increases fatigue life by a factor of about 1.16; that is, harder asphalts cause lower fatigue life.
5. Angular aggregate increases fatigue life by a factor of 1.14, relative to rounded aggregate.
6. For the limited range of the aggregate gradation used in this study, gradation has no significant effect on the fatigue life.

In a qualitative sense, the above observations (except Item 4) are compatible to those found in the literature for the controlled-stress tests. Quantitatively, the magnitude of the effects of the test and specimen variables on the fatigue life is different. In addition, the fatigue lives of the indirect tensile test specimens were found to be much shorter than those obtained from flexural beam tests (see Figure 6). This difference could be attributed to the stress distribution within the test specimens, specimen configuration, and the boundary conditions of both tests.

Equation 4 was used to calculate the fatigue lives of various INCCL specimens for several combinations of the specimen and test variables. Figure 7 depicts the fatigue curves at 77°F for three values of the percent air voids and three values of the kinematic viscosity. Similar plots at 40°F can be obtained.

The applicability of Equation 4, or any fatigue equation, to field conditions must be examined. Recall that fatigue life is affected by the stress distribution in the materials as well as the environmental conditions. Field conditions cannot be duplicated exactly in the laboratory. Hence, any prediction of pavement fatigue life based on laboratory test results is problematic. Equation 4, on the other hand, can be used to assess the effects of different materials, specimen variables, and temperatures on fatigue life. Equation 4 is not recommended, however, to be used in predicting fatigue life of in-service pavements without a proper calibration using field data. A limited field calibration is presented below.

Field Observations

Limited field observations of in-service flexible pavements in Michigan have indicated that the actual fatigue lives of the pavements were about 20 times higher than those estimated using Equation 4. Incorporating this factor into the equation, arranging terms, and designating the fatigue life of an in-service pavement by (FL) yields:

$$\begin{aligned}\ln(FL) = 42.63 &- (0.1402)(TT) \\ &- (2.3)[\ln(CL)] - (0.5095)(AV) \\ &- (0.001306)(KV) + (0.06403)(ANG) \qquad (5)\end{aligned}$$

It should be noted that Equation 5 is based on a limited number of field observations (10 pavement sections). Consequently, the accuracy of the equation should be checked as more field data become available. Further, because fatigue life of an asphalt pavement is a function of not only the properties of the AC surface but also those of the underlying materials, and because Equation 5 does not account for the properties of the various layers, it is difficult, physically, to support the results of Equation 5. This problem, however, was solved using a recent development at MSU and the MDOT.

MICH-PAVE

Recently, a nonlinear finite element computer program, MICH-PAVE, for designing flexible pavements was developed at MSU for the MDOT. The program is capable of calculating stresses and strains and the surface deflection developed in the pavement section due to a wheel load. Using the output of the MICH-PAVE program for existing 10 pavement sections (see Table 1) with known fatigue life and cross sections, Equation 5 was recalibrated. In the calibration process, the following information was used:

1. The actual fatigue lives of 10 pavement sections located in Indiana and Michigan.
2. The strains (calculated using the MICH-PAVE pro-

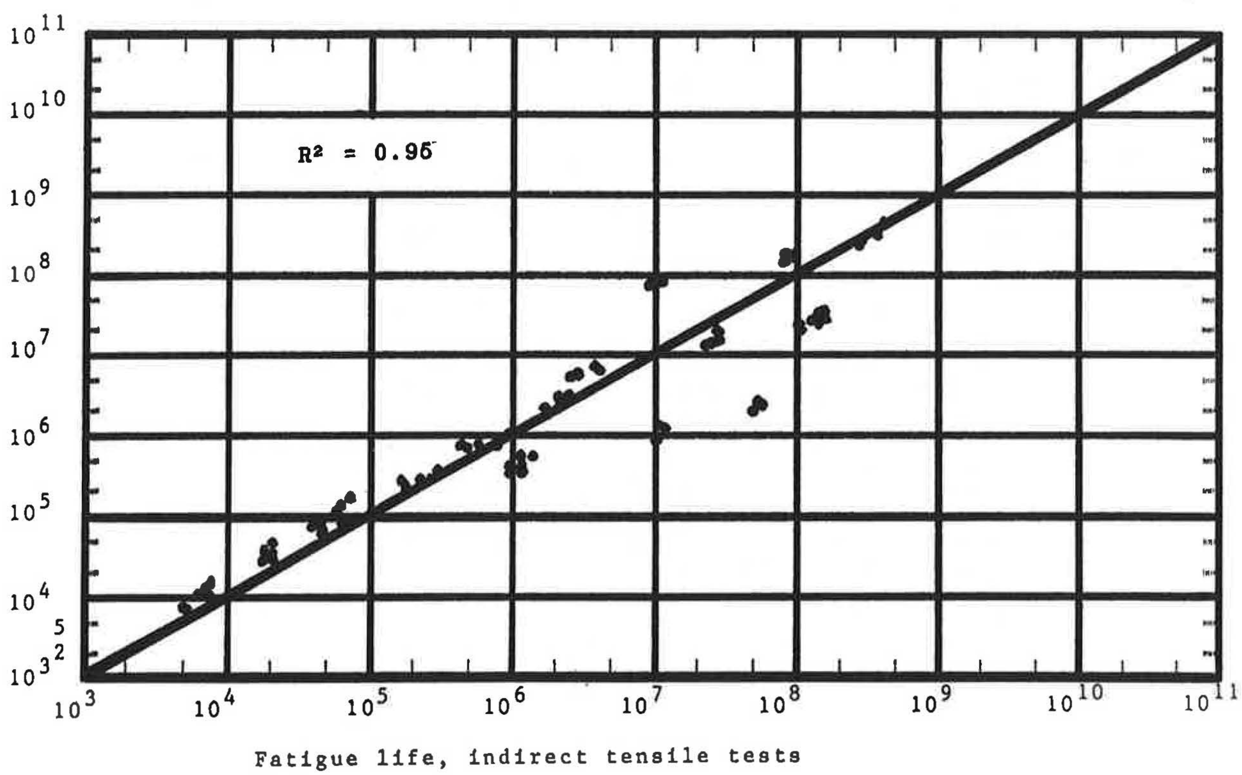

FIGURE 6 Fatigue lives of the INCCL test specimens versus those of flexural beam test specimens.

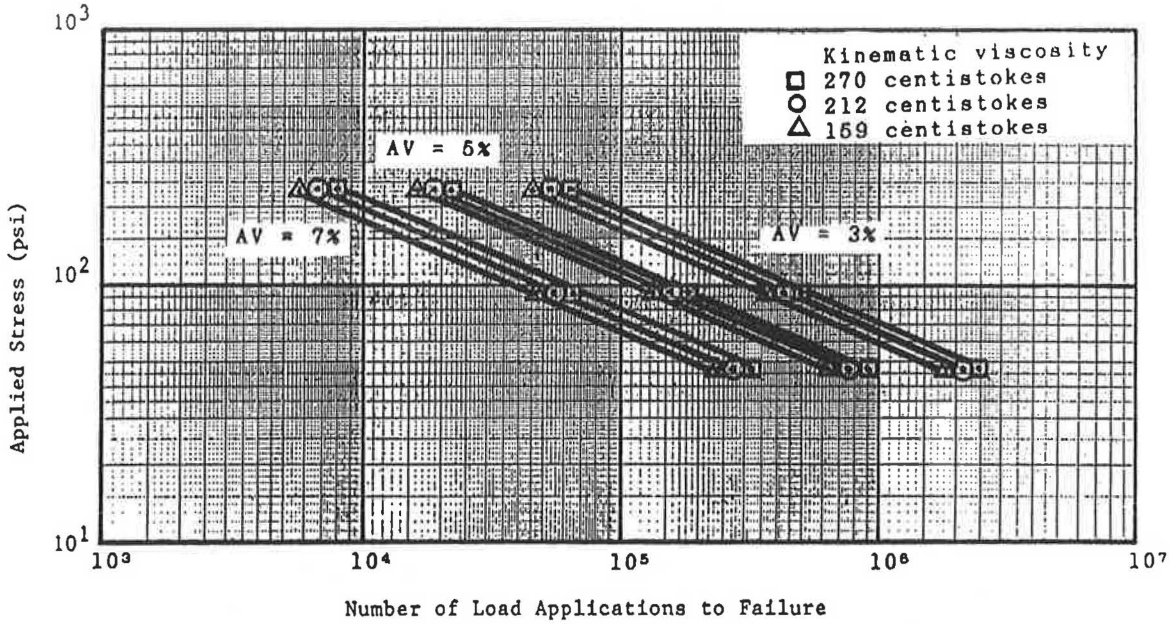

FIGURE 7 Fatigue curves of the INCCL test specimens.

TABLE 1 FATIGUE LIVES AND RUT DEPTHS OF 10 IN-SERVICE PAVEMENT SECTIONS

Site	AC TAC/MR in/ksi	Base TB/MR in/ksi	Subbase TSB/MR in/ksi	Roadbed MR ksi	Fatigue life (10^6 ESAL) FL(1)	FL(2)	AASHTO	Rut depth (in) RD(1)	RD(2)
IN1	4.0/500	12/60	6/12	6	1.9000	2.20000	0.7500	0.3	.123
IN1	5.0/500	12/60	6/12	6	5.5000	5.30000	1.7500	0.2	.101
IN1	6.0/500	12/60	6/12	6	9.7000	11.50000	3.7000	-	.087
IN2	3.0/150	12/25	-	4	0.0090	0.00425	0.0135	0.2	.303
IN3	3.0/150	6/30	-	6	0.0122	0.00206	0.0052	-	.347
IN4	3.0/150	10/25	-	4	0.0040	0.00226	0.0070	-	.316
MIU1	4.0/150	10/20	36/9	3	0.0820	0.06659	0.6200	<0.1	.047
MIU2	2.5/150	20/20	10/9	3	0.0410	0.02689	0.2800	<0.1	.086
MIU3	6.0/350	16/20	13/9	3	0.5000	0.53555	1.0000	0.5-1	.029
MIU4	4.5/350	12/20	70/9	3	5.4750	4.96000	45.0000	<0.1	-

IN1 = a pavement which was overlayed twice, each overlay consist of one inch thick AC;
AC = asphalt concrete;
TAC = thickness of the asphalt concrete course (inch);
TB = thickness of the base layer (inch);
TSB = thickness of the subbase (inch)
MR = resilient modulus of the designated layer (ksi); MRb : resilient modulus of the base layer (ksi); MRsb : resilient modulus of the subbase layer (ksi);
FL(1) = actual fatigue life of the pavement section (ESAL);
FL(2) = fatigue life calculated using equation 6 (ESAL);
AASHTO = fatigue life calculated using the 1986 AASHTO guide (ESAL);
RD(1) = measured rut depth (inch); and
RD(2) = rut depth calculated using equation 7 (inch).

Note: for all pavement sections located in Indiana (IN1 through IN4) an average annual air temperature of 75 °F was used, for Michigan sections (MIU1 through MIU4), 66° F was used.

gram) developed at the bottom fiber of the asphalt course due to an 18-kip axle load with 100 psi tire pressure.

3. The peak surface deflection (deflection under the center of the load) of the pavements, which was calculated using the MICH-PAVE program.

4. The thicknesses and properties of the various pavement layers.

5. Equation 5 with an average temperature of 66°F (TT in the equation), the actual (as-designed) air voids of the AC surface, the actual kinematic viscosity of the original AC binder, and 200 pounds (CL term in the equation) that correspond to a tire pressure of 100 psi.

The calibration process yielded the following fatigue life equation.

$$\begin{aligned}\log(\text{ESAL}) = & -2.544 + (0.154)(\text{TAC}) \\ & + (0.0694)(\text{TB}_{\text{EQ}}) \\ & - (2.799)[\log(\text{SD})] \\ & - (0.261)(\text{AV}) \\ & + (0.917)[\log(MR_{\text{B}})] \\ & + (0.0000269)(M_{\text{RB}}) \\ & - (1.0964)[\log(\text{TS})] \\ & + (1.173)[\log(\text{CS})] \\ & - (0.001)(\text{KV}) + (0.064)(\text{ANG}) \qquad (6)\end{aligned}$$

where

log = base 10 logarithmic operator,
ESAL = number of 18-kip equivalent single-axle loads (ESALs) to fatigue life,
TAC = thickness of AC course (in.),
TB_{EQ} = equivalent thickness of base material (in.), which is the actual thickness of the base layer plus the equivalent thickness of the subbase layer; equivalent thickness of the subbase layer is equal to the actual thickness of the subbase layer reduced by the ratio of the modulus of the subbase to that of the base material,
SD = surface deflection (in.),
MR_{B} = resilient modulus of the base material (psi),
MR_{RB} = the resilient modulus of the roadbed soil (psi),
TS = tensile strain at the bottom of the *AC* fiber,
CS = compressive strain at the bottom of the *AC* layer, and
ANG = aggregate angularity (4 for 100 percent crushed material, 2 for rounded river-deposited material, and 3 for a 50-percent mix of crushed and rounded aggregate).

Equation 6 seems to be very accurate, and it reasonably predicts the actual fatigue lives of the 10 pavement sections. In comparison, the 1986 AASHTO equation predicted the fatigue lives of the same sections to within a factor of 10. Nevertheless, the actual fatigue lives of the 10 pavement sections and their calculated fatigue lives using Equation 6 and the 1986 AASHTO design guide are listed in Table 1. It should be noted that, relative to Equation 6, the pavement surface deflection and the tensile and compressive strains at the bottom fiber of the asphalt course are functions of the properties of all layers in the pavement section as well as the applied load, hence, these terms are not independent variables.

In addition, a comparison between Equation 6 and the 1986 AASHTO equation was made using 30 arbitrarily selected pavement sections with various layer thicknesses and material properties (see Table 2). First, all the sections were analyzed using the MICH-PAVE computer program to obtain the various parameters (CS, TS, and SD due to an 18-kip single-axle load with 100 psi tire pressure) needed for inputs to Equation 6. The fatigue lives of all 30 sections were then calculated using Equation 6, as well as the 1986 AASHTO equation. The results are listed in Table 2 and are plotted in Figure 8. It should be noted that (for all sections) the ratio of the fatigue lives calculated using the AASHTO equation to those obtained using Equation 6 varied from 0.45 to 1.46.

RUT DEPTH

Plastic deformations along the vertical diameter (CD1) of the Marshall-size specimen (see Equation 3) were related to the rut depth of flexible pavements using field data obtained from seven pavement sections in Michigan and Indiana (see Table 1). First, each section was analyzed using the MICH-PAVE computer program. The resulting stresses, strains, surface deflection, and layer properties were then used to calibrate Equation 3. This resulted in the following equation:

$$\begin{aligned}\log(\text{RD}) = & -1.6 + (0.067)(\text{AV}) \\ & - (1.4)[\log(\text{TAC})] + (0.07)(\text{AAT}) \\ & - (0.000434)(\text{KV}) \\ & + (0.15)[\log(\text{ESAL})] \\ & - (0.4)[\log(MR_{\text{RB}})] \\ & - (0.50)[\log(MR_{\text{B}})] \\ & + (0.1)[\log(\text{SD})] \\ & + (0.01)[\log(\text{CS})] \\ & - (0.7)[\log(\text{TB}_{\text{EQ}})] \\ & + (0.09)\{\log[50 - (\text{TAC} + \text{TB}_{\text{EQ}})]\} \qquad (7)\end{aligned}$$

where

RD = rut depth (in.),
ESAL = the number of 18-kip ESALs at which the rut depth is being calculated,
SD = pavement surface deflection (in.), and
AAT = average annual temperature (°F).

It should be noted that the values of the moduli of the base and roadbed soil of Equations 6 and 7 are the yearly average values (effective values). These effective values are normally influenced by the freezing index and the seasonal variations of the moisture content of the base and roadbed soil. A procedure for calculating the effective moduli values can be found in the 1986 AASHTO guide for the design of pavement structures. Nevertheless, measured and calculated rut depths of

TABLE 2 FATIGUE LIVES AND RUT DEPTHS OF 30 ARBITRARILY SELECTED PAVEMENT SECTIONS

SEC NO.	TAC	TB	AV	MR (ksi) AC	MR (ksi) BASE	MR (ksi) RB	STRAINS TS	STRAINS CS	SD (in)	FATIGUE LIFE AASHTO	FATIGUE LIFE EQUATION 6	RUT DEPTH (in)
1	2	6	7	200	25.9	3	343	143	.1074	212	134	0.58
2	4	6	7	200	19.2	3	602	302	.0836	778	631	0.32
3	8	6	7	200	11.9	3	371	315	.0548	8767	9970	0.22
4	2	6	7	200	73.6	3	82	57	.0826	1261	*	*
5	8	6	7	200	24.8	3	298	317	.0516	23340	30622	0.18
6	8	6	7	200	38.0	3	247	318	.0495	45325	66288	0.16
7	2	6	5	350	24.5	3	457	121	.1012	446	328	0.50
8	2	6	3	500	23.2	3	476	98	.0954	835	933	0.44
9	8	6	5	350	10.0	3	232	163	.0459	47196	32677	0.21
10	8	6	3	500	8.8	3	156	100	.0408	210720	114230	0.19
11	2	9	7	200	24.2	3	298	240	.0953	784	629	0.57
12	2	12	7	200	22.7	3	295	293	.0874	1865	1606	0.54
13	4	9	7	200	56.3	3	184	322	.0594	26190	28509	0.24
14	4	12	7	200	54.4	3	174	353	.0539	87556	69653	0.22
15	4	6	7	200	58.0	3	222	274	.0677	6051	8318	0.26
16	2	6	7	200	24.2	6	340	244	.0685	1351	1190	0.58
17	2	6	7	200	28.7	9	328	292	.0531	3631	3891	0.56
18	8	6	5	350	11.9	9	204	167	.0230	680635	467143	0.17
19	8	12	3	500	10.5	9	141	102	.0193	5310000	4790000	0.11
20	8	12	3	500	12.6	25	130	105	.0120	73000000	66900000	0.10
21	8	12	3	500	35.2	9	108	108	.0170	34400000	29600000	0.08
22	8	12	3	500	38.9	25	98	111	.0097	539000000	512000000	0.07
23	8	18	3	500	10.5	9	140	102	.0196	8410000	11900000	0.10
24	8	18	3	500	11.8	25	133	104	.0127	95000000	132000000	0.08
25	8	18	3	500	33.6	9	104	109	.0165	95400000	84500000	0.07
26	8	18	3	500	49.1	25	87	116	.0097	2092600000	1286000000	0.06
27	8	18	5	350	23.4	9	161	184	.0202	11300000	11800000	0.09
28	8	18	5	350	26.3	25	150	188	.0128	193000000	148000000	0.08
29	8	18	5	350	37.3	9	132	191	.0185	44300000	30100000	0.08
30	8	18	5	350	40.2	25	124	195	.0114	640000000	399000000	0.07

TAC = THICKNESS OF THE AC COURSE (in) TB = THICKNESS OF THE BASE (in)
AV = PERCENT AIR VOIDS OF THE AC MR = RESILIENT MODULUS (ksi)
AC = ASPHALT CONCRETE RB = ROADBED SOILS
TS = TENSILE MICROSTRAIN (BOTTOM OF AC) CS = COMPRESSIVE MICROSTRAIN
SD = SURFACE DEFLECTION * = SHEAR FAILURE
NOTE: THE RUT DEPTH IS CALCULATED AT THE FATIGUE LIFE OF THE PAVEMENT SECTION AND AN AVERAGE ANNUAL TEMPERATURE OF 75 °F.

the seven in-service pavement sections are listed in Table 1. In addition, the rut depths due to a number of ESALs equal to the fatigue lives of the 30 arbitrarily selected pavement sections were calculated using Equation 7 and are listed in Table 2.

Two important points should be noted.

1. Equation 7 is based on limited field data. Hence, it should be used with extreme caution and the equation should be calibrated as more field data become available.
2. For each arbitrarily selected pavement section of Table 1, the surface deflections and the compressive and tensile strains due to several axle loads (3,000 to 32,000 lb) and for various trucks (single-axle, semi, and tandem) were also calculated. It was noted that the AASHO equivalent load factors provided in the 1986 AASHTO design guide for terminal serviceability of 2.5 are related to the pavement surface deflection (SD in Table 1) calculated under the center of the load. The relationship can be expressed as follows:

$$ELF = (SD_i/SD_{18})^{4.25} \quad (8)$$

where

ELF = AASHO equivalent load factor obtained for a terminal serviceability index of 2.5 using the 1986 AASHTO design guide,
SD_{18} = surface deflection due to 18-kip single-axle load, and
SD_i = surface deflection for load i.

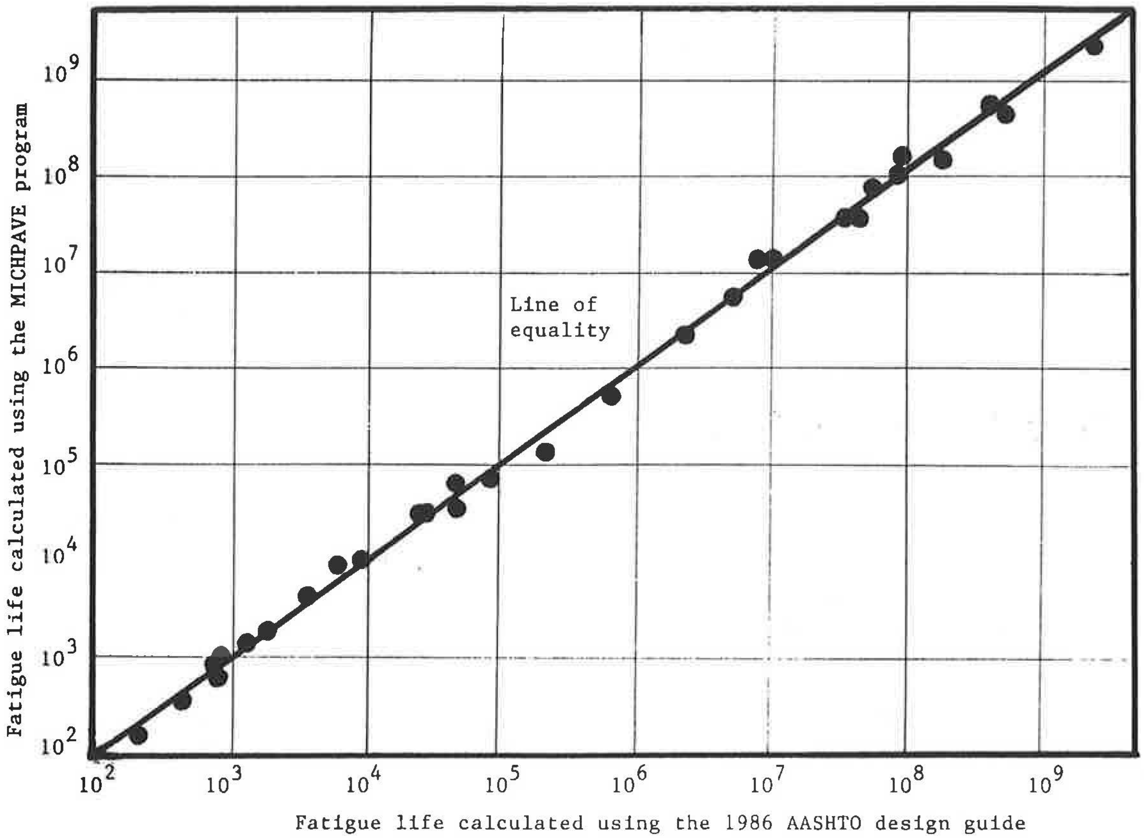

FIGURE 8 Fatigue lives of 30 arbitrarily selected pavement sections calculated using the MICH-PAVE program versus those calculated using the 1986 AASHTO Design Guide.

The significance of Equation 8 is that, because the surface deflection of a pavement section varies with time (seasonal variations) as well as with the pavement conditions, the ELF (a measure of relative load-related damage) of any vehicle including the 18-kip single-axle load is not constant. For example, the load-related damage of an 18-kip single-axle load traveling a pavement section in the spring (thaw condition) is higher than that delivered during summer conditions. Similarly, the load-related damage delivered to a pavement section increases as the pavement develops more cracks. The values of ELF provided in the AASHTO design guide do not account for seasonal variations and/or deteriorating pavement conditions. For example, the ELF value of an 18-kip single-axle load is one, regardless of the value of the structural number (SN) of the pavement. The value of SN for a given pavement section changes and it is a function of time (seasonal variation) and pavement conditions. Hence, the ELF value for an 18-kip single-axle truck is not constant. Equation 8 can be used to assess the ELF for the load-related damage of any vehicle, including the 18-kip single-axle load. This can be accomplished using the following steps:

1. Establish a standard design value of the surface deflection (standard design deflection; SDD) for the pavement section in question due to 18-kip single-axle load. This design value (constant with time and deteriorating pavement conditions) is the denominator of Equation 8.
2. Measure the surface deflection (SD) of any vehicle including an 18-kip single-axle load at any time and for any pavement condition. This value is the numerator of Equation 8.
3. Calculate the ELF.

The advantages of the above procedure and Equation 8 include the following:

1. The ELF of an 18-kip single-axle load calculated immediately after construction can be used to determine construction quality and/or as a part of the construction specifications.
2. A maximum threshold value of ELF for an 18-kip single-axle load can be established, above which the structural capacity of the pavement is rendered unacceptable. Thus, the critical time for rehabilitation can be established.
3. The values of ELF with time can be used to estimate past and predict future structural performance of the pavements.

IMPLEMENTATION

The advantage of Equations 6 and 7 is that, for most mechanistic pavement design procedures (e.g., CHEV5, ELSYM, MICH-PAVE, ILLI-PAVE, VESYS), the parameters (e.g., CS, TS, SD, and *MR*) of Equations 6 and 7 are typically calculated. The calculated values can be used as inputs into the equations to predict the fatigue life and rut depth of the pavement that is being designed.

Equations 6 and 7 can be used to design a pavement cross section such that the limiting acceptable value of the rut depth (established by the state highway agency) and the fatigue life of the pavement section are reached at the same time (the intended design life of the pavement). That is, the equations will allow the engineer to balance the pavement design between fatigue life and rut depth.

It is of utmost importance to note that temperature cracks, stripping, and other distresses should also be considered in the design of flexible pavements. Further, Equations 6 and 7 are based on limited field data, hence, further calibration and/or verification may be required.

SUMMARY AND CONCLUSIONS

The capability of predicting the fatigue life and rut depth of asphalt pavements is essential for the structural design of pavement structures. These predictions become increasingly important as more highway engineers use pavement design systems based on elastic and/or viscoelastic theories that require estimates of the fatigue life. Further, a proper asphalt concrete mix design procedure should be based on the fundamental properties and fatigue life of the mix. A new indirect tensile test apparatus was designed and used in this study to obtain the asphalt mix properties and fatigue life. The indirect tensile test is simple and yields consistent results. Fatigue lives calculated using Equation 6 were compatible to those obtained for 10 pavement sections and to a certain extent were compatible to those obtained using the AASHTO equation for 30 arbitrarily selected pavement sections.

Considering the test results and the analytical and statistical analyses, the following conclusions were drawn:

1. The indirect tensile test can be used to study the effects of the test, specimen, and asphalt mix variables on its fatigue life and plastic deformation.
2. Test results obtained using the new indirect tensile test apparatus are consistent and reproducible.
3. The test temperature and the percent air void in the mix have the greatest influence on the fatigue life and rut depth of the mix.
4. The fatigue life of asphalt concrete increases as the temperature decreases, the percent air void of the AC mix decreases, the stiffness of the AC binder decreases, and/or the aggregate angularity increases.
5. Most existing mechanistic design procedures are capable of calculating the parameters needed for inputs to Equations 6 and 7.
6. Equations 6 and 7 represent two steps in the right direction for the prediction of fatigue life and rut depth of asphalt pavements. The equations can be programmed as a part of almost all existing mechanistic design procedures.
7. Further calibration and verification of Equations 6 and 7 should be accomplished on a continuing basis as more data become available.

RECOMMENDATIONS

Existing asphalt mix design procedures, such as Marshall and Hveem, possess no relationship to the structural properties and fatigue life of asphalt concrete mixes. These properties are needed, and they are essential to the proper design of flexible pavement structures. A proper asphalt mix design procedure should be tailored to optimize the values of the structural properties of the mix. The results and analyses presented in this paper and those presented elsewhere (*28,29*) indicate that the structural properties of asphalt concrete mixes can be obtained using cyclic load tests and the new indirect tensile test apparatus. Consequently, it is highly recommended that the new indirect tensile test apparatus be used to establish a new asphalt mix design procedure, whereby the mix design parameters can be obtained based on the structural properties of the mix.

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REFERENCES

1. *AASHTO Interim Guide for Design of Pavement Structures*. AASHTO, Washington, D.C., 1981.
2. G. Y. Baladi. *Characterization of Flexible Pavement: A Case Study*. Special Technical Paper 807. American Society for Testing and Materials, 1983, pp. 164–171.
3. W. J. Kenis. *Material Characterizations for Rational Pavement Design*. Special Technical Paper 561. American Society for Testing and Materials, 1973, pp. 132–152.
4. J. S. Miller, J. Uzan, and M. W. Witczak. Modification of the Asphalt Institute Bituminous Mix Modulus Predictive Equation. In *Transportation Research Record 911*, TRB, National Research Council, Washington, D.C., 1983, pp. 27–36.
5. L. W. Nijboer. Mechanical Properties of Asphalt Materials and Structural Design of Asphalt Roads. *Proc., 33rd Annual Meeting*, HRB, National Research Council, Washington, D.C. Vol. 33, 1954, pp. 185–200.
6. J. F. Shook and B. F. Kallas. Factors Influencing Dynamic Modulus of Asphalt Concrete. *Proc. AAPT*, Vol. 38, 1969, pp.140–178.
7. E. J. Yoder and M. W. Witczak. *Principles of Pavement Design*, 2nd ed., John Wiley and Sons, Inc., New York, 1975.
8. The Asphalt Institute. *Asphalt Overlays for Highway and Street Rehabilitation*. The Asphalt Institute, Manual Series No. 17, College Park, Md., June 1983.
9. S. F. Brown and K. E. Cooper. A Fundamental Study of the Stress-Strain Characteristics of a Bituminous Material. *Proc. AAPT*, Vol. 49, 1980, pp. 476–499.
10. F. N. Finn. *NCHRP Report 39: Factors Involved in the Design of Asphaltic Pavement Surfaces*. HRB, National Research Council, Washington, D.C., 1967, pp. 1–112.
11. B. F. Kallas and J. F. Shook. Factors Influencing Dynamic Modulus of Asphalt Concrete. *Proc. AAPT*, Vol. 38, 1969, p. 140.
12. H. L. Von Quintus, J. B. Rauhut, and T. W. Kennedy. Comparisons of Asphalt Concrete Stiffness as Measured by Various Testing Techniques. *Proc. AAPT*, Vol. 51, 1982, pp. 35–52.
13. M. W. Witczak and R. E. Root. *Summary of Complex Modulus Laboratory Test Procedures and Results*. American Society for Testing and Materials, Special Technical Paper No. 561, 1974, pp. 67–94.
14. *AASHTO Test and Material Specifications*. Parts I and II, 13th edition, AASHTO, Washington, D.C., 1982.
15. *Mix Design Methods for Asphalt Concrete and Other Hot-Mix Types*. Manual Series No. 2, The Asphalt Institute, College Park, Md., 1979.
16. M. H. Farzin, R. J. Krizek, and R. B. Corotis. Evaluation of Modulus and Poisson's Ratio From Triaxial Tests. In *Transpor-*

tation Research Record 537, TRB, National Research Council, Washington, D.C., 1975, pp. 69–80.
17. W. H. Goetz. Comparison of Triaxial and Marshall Test Results. *Proc. AAPT*, Vol. 20, 1951, pp. 200–245.
18. G. Gonzalez, T. W. Kennedy, and J. N. Anagnos. *Evaluation of the Resilient Elastic Characteristics of Asphalt Mixtures Using the Indirect Tensile Test*. Report No. CFHR 3-9-72-183-6, Transportation Planning Division, Texas State Department of Highways and Public Transportation, Austin, Tex., Nov. 1975.
19. W. O. Hadley and H. Vahida. A Fundamental Comparison of the Flexural and Indirect Tensile Tests. Presented at the Annual Meeting, Transportation Research Board, January 1983.
20. J. F. Hills and W. Heukelom. The Modulus and Poisson's Ratio of Asphalt Mixes. *Journal of the Institute of Petroleum*, Vol. 55, Jan. 1969, pp. 27–35.
21. T. W. Kennedy. Characterization of Asphalt Pavement Materials Using the Indirect Tensile Test. *Proc. AAPT*, Vol. 46, 1977, pp. 132–150.
22. C. L. Monismith. Flexibility Characteristics of Asphalt Paving Mixtures. *Proc. AAPT*, Vol. 27, 1958, pp. 74–106.
23. P. S. Pell and S. F. Brown. The Characteristics of Materials for the Design of Flexible Pavement Structures. *Proc., 3rd International Conference on the Structural Design of Asphalt Pavements*, University of Michigan, Ann Arbor, Vol. 1, 1972, pp. 326–342.
24. S. P. Timoshenko and J. N. Goodier. *Theory of Elasticity*, McGraw Hill Book Company, New York, 1970.
25. M. A. Young and G. Y. Baladi. *Repeated Load Triaxial Testing, State of the Art*. Michigan State University, Division of Engineering Research, 1977.
26. G. Y. Baladi. *Linear Viscosity*, U.S. Army Corps of Engineers, Waterways Experiment Station, Vicksburg, Miss., Oct. 1985, pp. 1–6.
27. G. Y. Baladi. *Numerical Implementation of a Transverse-Isotropic, Inelastic, Work-Hardening Constitutive Model*. Soil Dynamics Division, Soils and Pavement Laboratory, U.S. Army Corps of Engineers, Waterways Experiment Station, Vicksburg, Miss., pp. 1–12, 1983.
28. G. Y. Baladi, R. S. Harichandran, and R. W. Lyles. New Relationships Between Structural Properties and Asphalt Mix Parameters. In *Transportation Research Record 1171*, TRB, National Research Council, Washington, D.C., 1988, pp. 168–177.
29. G. Y. Baladi, R. W. Lyles, and R. S. Harichandran. Asphalt Mix Design: an Innovative Approach. In *Transportation Research Record 1171*, TRB, National Research Council, Washington, D.C., 1988, pp. 160–167.
30. G. Y. Baladi and J. H. De Foe. *The Indirect Tensile Test: A New Horizon*. Interim Report FHWA, U.S. Department of Transportation, March 1987.
31. G. Y. Baladi. *Integrated Material and Structural Design Method For Flexible Pavements*. Final Report No. FHWA/RD-88/109, 110 and 118, Sept. 1987.

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