

Effect of Temperature and Mixture Variables on Fatigue Life Predicted by Diametral Fatigue Testing

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One of the major objectives in the Strategic Highway Research Program Project A-003A, Performance Related Testing and Measuring of Asphalt-Aggregate Interactions and Mixtures, is to develop accelerated performance-related tests for asphalt aggregate systems that successfully model construction and service conditions. Controlled-stress diametral fatigue testing was conducted at North Carolina State University to evaluate the influence of temperature and mixture variables on fatigue life of asphalt concrete mixtures with RB (MRL code) aggregates. The resilient modulus testing fixture developed by MTS Systems Corporation in accordance with ASTM D4123 was used with the MTS closed-loop servohydraulic system. Diametral fatigue test results plotted on a logarithmic scale of recoverable horizontal deformation and number of cycles to failure showed a good agreement with the historic trend of the fatigue data. The analysis of variance test results indicated that all the factors investigated in this research significantly affected the fatigue performance of asphalt concrete. In summary, the fatigue resistance was improved by increasing asphalt content and reducing air void contents. The effect of temperature on the fatigue performance of asphalt concrete determined by using recoverable horizontal strain was opposite to that determined by using maximum total horizontal strain. Asphalt concrete with a softer binder resulted in longer fatigue life. Furthermore, the temperature susceptibility of asphalt binder had a pronounced effect on the fatigue life of asphalt concrete mixtures.

Fatigue cracking is one of the major distresses occurring in asphalt concrete pavements. A significant amount of effort has been made to predict the fatigue life of asphalt concrete pavements by laboratory testing under simulated environmental conditions. This effort has produced various types of test configurations and a considerable amount of laboratory data evaluating effects caused by different environmental conditions and mixture variables.

One of the major objectives of the Strategic Highway Research Program Project A-003A, Performance Related Testing and Measuring of Asphalt-Aggregate Interactions and Mixtures, is to develop accelerated performance-related tests for asphalt-aggregate systems that successfully model construction and service conditions. As one of the subcontractors of this project, North Carolina State University is responsible for evaluating the feasibility of the diametral fatigue test as a standardized fatigue testing method.

The advantages of diametral fatigue test are as follows:

1. The test is relatively simple to conduct;
2. Sample fabrication is easy;
3. The same test configuration has been standardized by ASTM to determine indirect tensile strength and resilient modulus of asphalt concrete;
4. Failure is initiated in a region of relatively uniform tensile stress; and
5. Stress and strain solutions are readily available.

Regardless of these advantages, the sensitivity of test results to environmental conditions and mixture variables must be proved in order for the test to be a standardized method. The test results also must be consistent with historically available data from laboratory testing, test tracks, and field trials.

In this research, diametral fatigue testing was performed on dense-graded asphalt concrete samples at varying temperatures and mixture conditions. The mixture variables investigated in this research include temperature susceptibility of asphalt cement, asphalt content, air void content, and asphalt type.

MATERIALS

Asphalt Cement

Two asphalts used in this research were AAK-1 and AAG-1 asphalts (MRL codes). These asphalts were selected because of their vastly different compositional and temperature-susceptibility characteristics. From the penetration-viscosity-temperature plot, AAG-1 asphalt was found to be more temperature-susceptible than AAK-1 asphalt (1).

Two levels of asphalt content for each asphalt (by weight of aggregate) were determined at the University of California, Berkeley, as follows:

AAK-1 (%)		AAG-1 (%)	
Low	High	Low	High
5.1	5.7	4.9	5.5

The lower asphalt content was determined using a modification of standard Hveem (ASTM D1560) procedure, and the higher asphalt content was determined by the U.S. Army Corps of Engineers (Marshall) 75-blow procedure (ASTM D1559) (1). The higher asphalt content exceeded the lower

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asphalt content by about 0.6 to 0.7 percent (by weight of aggregate).

Aggregates

The aggregates used in this research were RB aggregates (MRL code). The selected gradation and ASTM D3515 specification limits are shown in Figure 1. Wet-sieve analysis was conducted to meet this gradation. The detailed sieve analysis procedure can be found elsewhere (1).

SPECIMEN PREPARATION AND COMPACTION

Mixing and Curing

The optimum mixing temperatures for two asphalts were selected from a Bitumen Test Data Chart as follows (1) [$t^{\circ}\text{F} = (t^{\circ}\text{C} \div 0.55) + 32$]:

Asphalt Type	Mixing Temperature ($^{\circ}\text{F}$)
AAG-1	284
AAK-1	300

The optimum temperatures correspond to a viscosity of 170 ± 20 cSt (1).

The asphalt was heated for distribution to small cans (first heating). The asphalt in the small cans to be used for mixing (second heating) was heated at the appropriate mixing temperature. It was stirred periodically to ensure uniform heating. The sample was discarded if it was not used within 3 to 3.5 hours from the start of heating. The heating of the asphalt was done once and was continuous.

Once the asphalt and aggregate were heated at the mixing temperature, they were mixed for 4 min inside a steel bowl. After mixing, the asphalt concrete was placed in an oven

maintained at 140°F (60°C) for 15 hr. This curing operation allows any asphalt absorption by the aggregate to take place before compaction or any other use of the mix. More detailed information on mixing and curing can be found elsewhere (1).

Compaction

Cured mixtures were compacted at 240°F (116°C) using the gyratory testing machine designed by the U.S. Army Corps of Engineers. One degree of gyratory angle was used to produce briquets 2.5 in. (6.35 cm) high and 4 in. (10.16 cm) in diameter. Two levels of compaction effort were introduced to mixtures in order to produce specimens with two air void contents, 4 and 8 percent, determined in accordance with ASTM D1188.

TEST METHOD

Test Apparatus

The 810 Materials Testing System (MTS) with the 458.20 MicroConsole was used in this research. This servohydraulic system has a closed-loop control. The testing apparatus was provided by MTS Systems Corporation (2) and was originally designed for the determination of resilient modulus of asphalt concrete in accordance with ASTM D4123. The schematic presentations of this fixture are shown in Figures 2–4. The fixture was installed inside an environmental chamber in which temperature could be maintained within $\pm 1^{\circ}\text{F}$ for extended periods.

Horizontal and vertical displacements were measured by extensometers designed by MTS Systems Corporation with full-scale travels of 0.15 in. (0.381 cm) and 0.16 in. (0.4064 cm), respectively. Ranging the transducers to 10 percent of

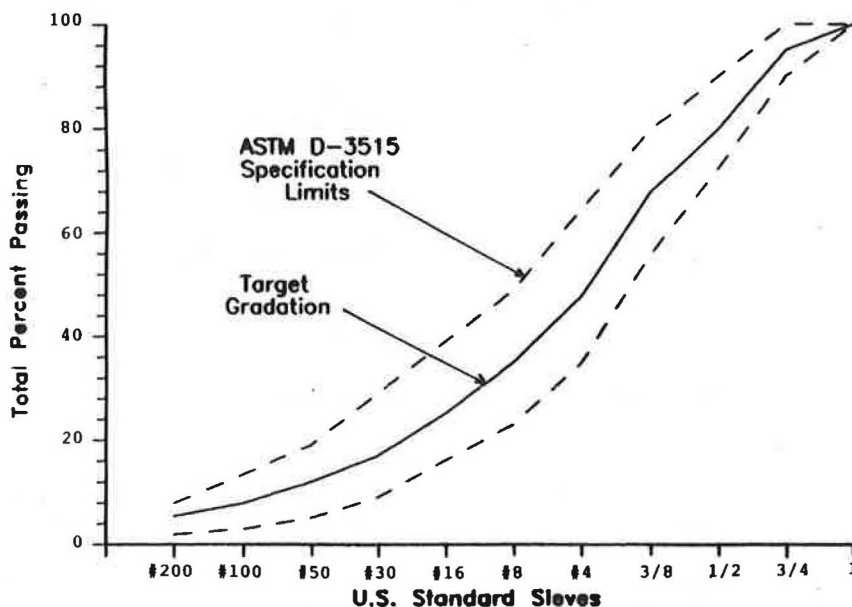


FIGURE 1 Target gradation for RB aggregates after wet-sieve analysis.

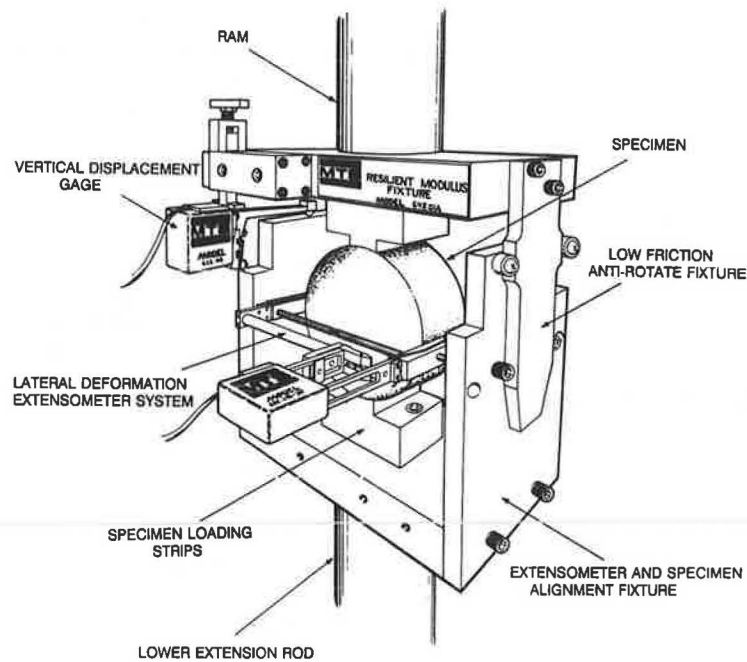


FIGURE 2 Diametral fatigue-testing fixture and extensometers.

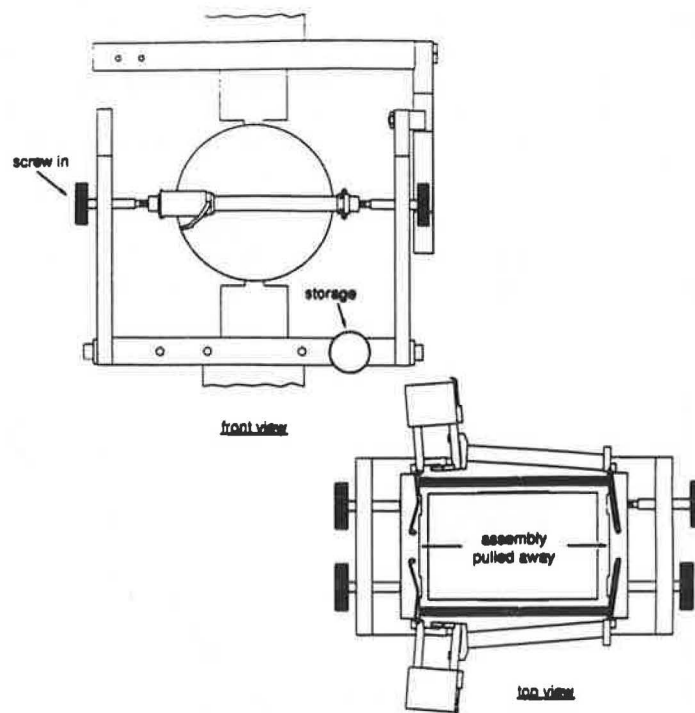


FIGURE 3 Positioning of side brackets using thumbscrews.

full scale calibrates the output to a finer-scale travel and allows higher-resolution measurements of small deformations.

The diametral device consists of two extensometers with gauge-length extenders and two specimen-adapter brackets as shown in Figure 2. The brackets are machined to the same radius as the specimen and remain in contact with the spec-

imen all along the thickness. This design allows each bracket to measure the maximum deformation instead of measuring local deformation caused by point contact with a linear variable differential transducer, which is used in some diametral tensile test fixtures. The device is located on the specimen by guiding pins with the same lengths that extend from the side

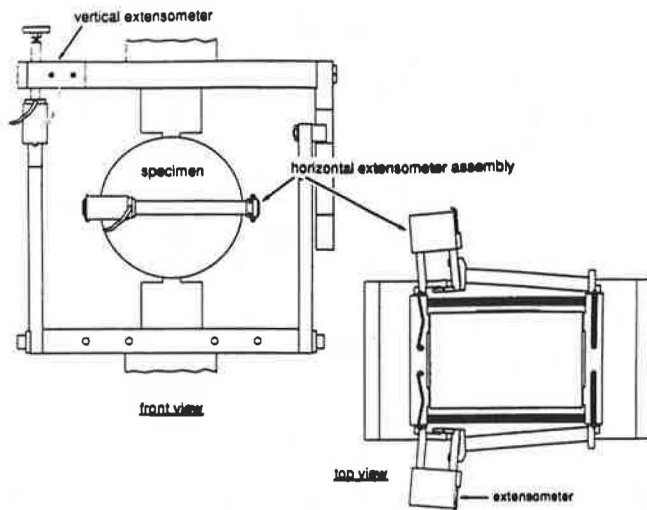


FIGURE 4 Setup ready for testing with spring-loaded extensometers.

of the fixture (Figure 3). When the brackets are pulled away completely, the equal distance between the brackets and the specimen ensures that the center line of the specimen is parallel to the direction of the upper and lower loading strips. Also, because the brackets are placed at the sides of the specimen and guided by pin screws through the holes in fixed side walls, this fixture gives more precise control over the vertical locations of the brackets.

Once the test starts, the low-friction antirotate bar in the upper fixture (Figure 2) prevents possible rotation of the actuator due to repetitive loading. Another advantage of this fixture is that improper measurements caused by "rocking" of the specimen can be minimized. Because the extensometers are attached to the specimen by springs (Figure 4), extraneous deformation caused by rigid-body rotation cannot affect the deformation measurements.

Test Protocol

Four factors were investigated in this research: asphalt type, asphalt content, air void content, and testing temperature. The experimental design is shown in Table 1.

Haversine load with a 0.1-sec load duration and 0.5-sec rest period was repeated until the specimen "failed." The stress amplitude was kept constant throughout testing, and horizontal and vertical deformations were recorded at the 200th cycle. Two load levels, low and high, were used; these load amplitudes induce the failure of the specimen under a specific condition at about 100,000 and 10,000 cycles, respectively.

Horizontal tensile strain (ϵ_x) in the indirect tensile specimen under the line loading can be determined from

$$\epsilon_x = \frac{2P}{E\pi td} \left[\frac{(1 + 3\mu)d^4 - 8x^2d^2(1 + \mu) + 16x^4(1 - \mu)}{(d^2 + 4x^2)^2} \right] \quad (1)$$

where

P = load amplitude applied (lb),

E = elastic modulus of specimen (psi),

t = thickness of specimen (in.),

d = diameter of specimen (in.),

μ = Poisson's ratio of specimen, and

x = distance from center line of specimen (in.).

Integrating Equation 1 along the diameter of the specimen obtains the following equation for the horizontal deformation (δ_H):

$$\delta_H = \frac{P}{tE} \left(\frac{4}{\pi} - 1 + \mu \right) \quad (2)$$

To find the largest horizontal strain that occurs in the middle plane, x in Equation 1 is replaced with 0 to get:

$$\epsilon_0 = \frac{2P(1 + 3\mu)}{E\pi td} \quad (3)$$

where ϵ_0 is horizontal strain in the middle plane.

In spite of the temperature and stress-level dependency of Poisson's ratio, this study used a value of 0.35 as suggested in ASTM D4123. Equating Equations 2 and 3 obtains the following equation for a 4-in.-diameter specimen:

$$\epsilon_0 = 0.5235\delta_H \quad (4)$$

Therefore, the recoverable horizontal strain in the middle plane can be determined from

$$\Delta\epsilon_0 = 0.5235\Delta\delta_H \quad (5)$$

Failure Criteria

The fatigue life of the specimen predicted from the laboratory testing is significantly influenced by the definition of the "failure." According to the earlier study by Scholz (3), the failure

TABLE 1 EXPERIMENTAL DESIGN FOR DIAMETRAL FATIGUE TESTS

Asphalt Type	Asphalt Content	Air Voids	Temperature
0	0	0	0
1	0	0	0
0	1	0	0
1	1	0	0
0	0	1	0
1	0	1	0
0	1	1	0
1	1	1	0
0	0	0	1
1	0	0	1
0	1	0	1
1	1	0	1
0	0	1	1
1	0	1	1
0	1	1	1
1	1	1	1

Asphalt Type: 0 = AAK-1
1 = AAG-1

Asphalt Content: 0 = Low
1 = High

Air Voids: 0 = 4 ± 0.5%
1 = 8 ± 0.5%

Temperature: 0 = 32°F (0°C)
1 = 68°F (20°C)

in diametral fatigue testing occurs when the permanent horizontal deformation reaches between 0.28 in. (0.71 cm) and 0.36 in. (0.91 cm). It was recommended that the lead-based foil tape attached on the specimen and the fatigue shut-off wiring for the MTS could automatically stop the testing when the permanent horizontal deformation reached the failure criteria. Preliminary testing was performed under various conditions to determine unified failure criteria for the controlled-stress diametral fatigue tests conducted in this research project.

Throughout the preliminary testing, horizontal deformation was monitored and plotted against the number of cycles (Figures 5 and 6). Irrespective of the test temperature and the applied load, it was found that the horizontal deformation increased dramatically after a value of 0.1 in. of horizontal deformation. Even though the dramatic increase in horizontal deformation occurred earlier than a value of 0.1 in. (0.25 cm) in some cases [e.g., tests done at 32°F (0°C), shown in Figure 5], the difference in the number of cycles between the point where dramatic increase in horizontal deformation occurred

and the point with the horizontal deformation equal to 0.1 in. (0.25 cm) was relatively small. It was thus concluded that the failure of the material in the controlled-stress diametral fatigue testing can be described as the moment when the total horizontal deformation reaches 0.1 in. (0.25 cm).

The greatest advantage of having failure criteria at 0.1 in. (0.25 cm) of horizontal deformations is that the fatigue test can be automatically stopped using the MTS limit detector without any additional devices or mechanisms. The limit detector automatically turns off the hydraulic pressure when the horizontal extensometer reading becomes larger than the preset limit, horizontal deformation of 0.1 in.

DISCUSSION OF TEST RESULTS

The recoverable horizontal strain during unloading of 200th cycle was plotted against the number of cycles to failure on a log-log scale. A linear behavior between logarithms of the recoverable horizontal strain and the number of cycles to

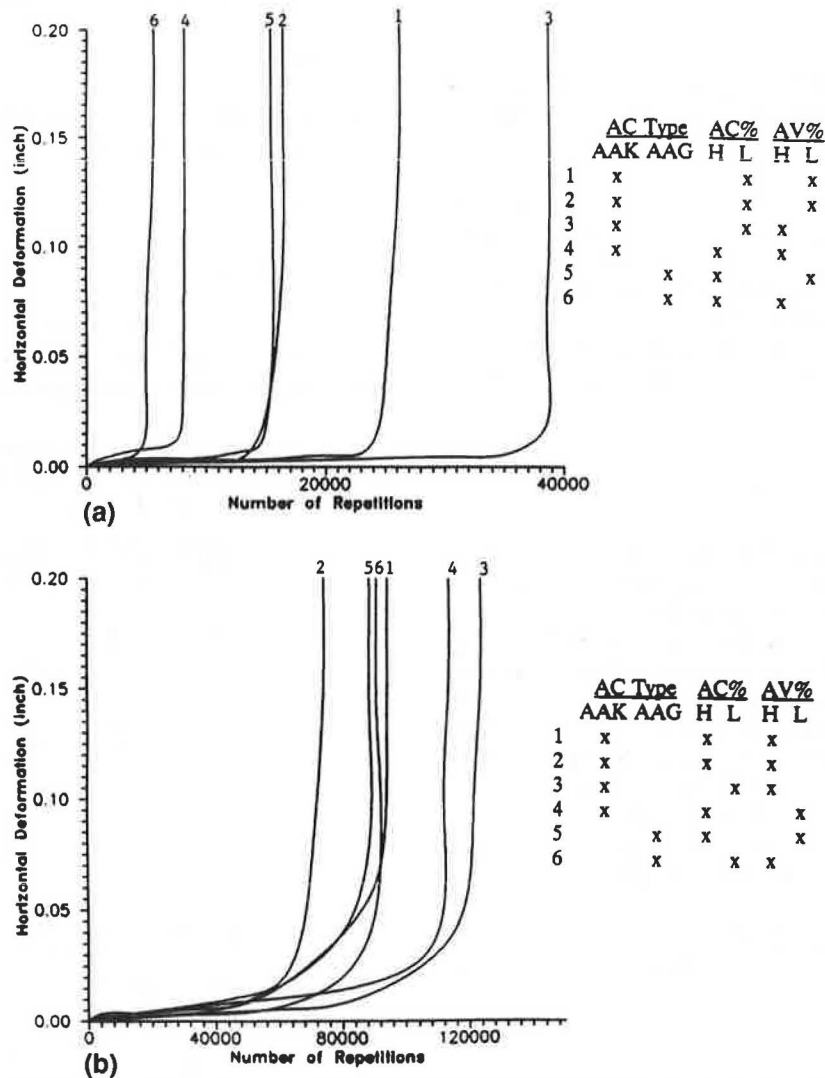


FIGURE 5 Growth of horizontal deformation at 32°F (0°C) at (a) high stress level and (b) low stress level.

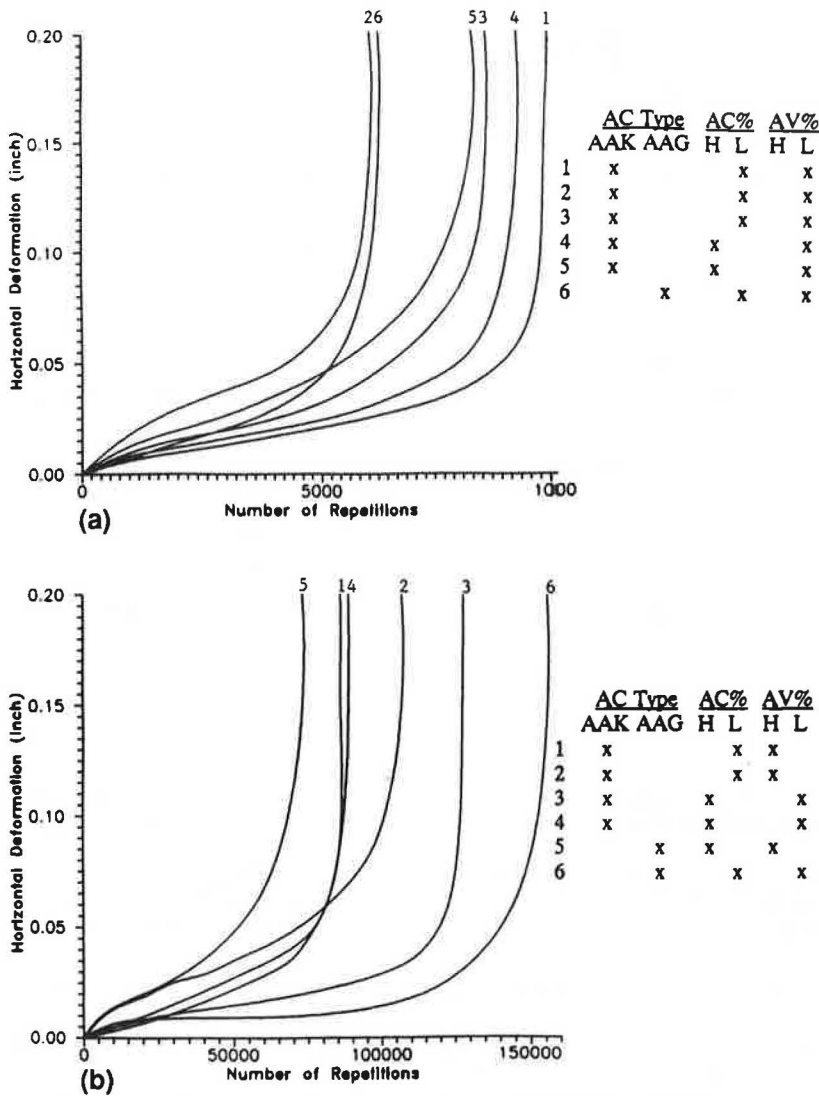


FIGURE 6 Growth of horizontal deformation at 68°F (20°C) at (a) high stress level and (b) low stress level.

failure confirmed the following power form of the fatigue model:

$$N_f = K_1 \left(\frac{1}{\epsilon_t} \right)^{K_2} \quad (6)$$

where

- N_f = number of cycles to failure,
- ϵ_t = tensile strain in asphalt concrete, and
- K_1, K_2 = regression constants.

The entire test results in accordance with the experimental design in Table 1 are plotted in Figures 7 and 8, and a summary of K_1 and K_2 values are shown in Table 2. Generation of each data point required one test on one sample; for each test condition, four to six data points were required to fit a straight line using a regression analysis.

The same approach was followed using the maximum total horizontal strain at the 200th cycle in lieu of using recoverable horizontal strain in the ordinate of the plots. However, the figures with the maximum total horizontal strain in the or-

ordinate did not demonstrate the effect of temperature and mixture variables as distinctively as Figures 7 and 8 did with the recoverable horizontal strains. Also, as shown in Figure 9, the variation of fatigue data from the best-fit line in the figures was generally smaller when the recoverable horizontal strain was used. The reason for the better role of recoverable horizontal strain is probably because the recoverable horizontal strain does not consider the plastic behavior of materials. Therefore, it was decided to use the recoverable horizontal strain to evaluate the effect of temperature and mixture variables on fatigue life of asphalt concrete.

The analysis of variance (ANOVA) tests for equality of slopes and intercepts of lines were conducted to statistically support the observations from the figures on the significance of different factors on fatigue performance. Test results indicated that all the interactions among factors were statistically insignificant. Therefore, a summary of ANOVA test results is shown in Table 3 for only four independent factors. The intercept in Table 3 is the intercept at the number of cycles to failure equal to 1,000 cycles. The use of intercept at

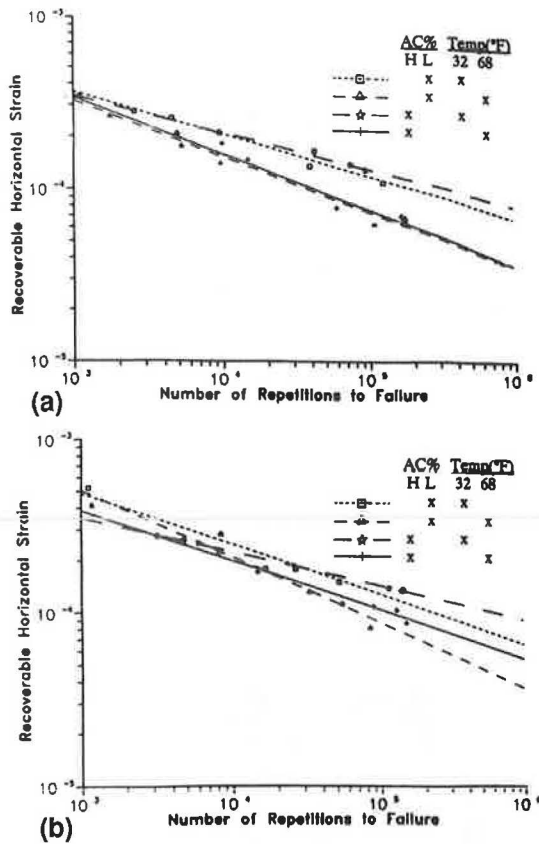


FIGURE 7 Diametral fatigue test results of AAK-1 asphalt mixtures with (a) high air void content and (b) low air void content.

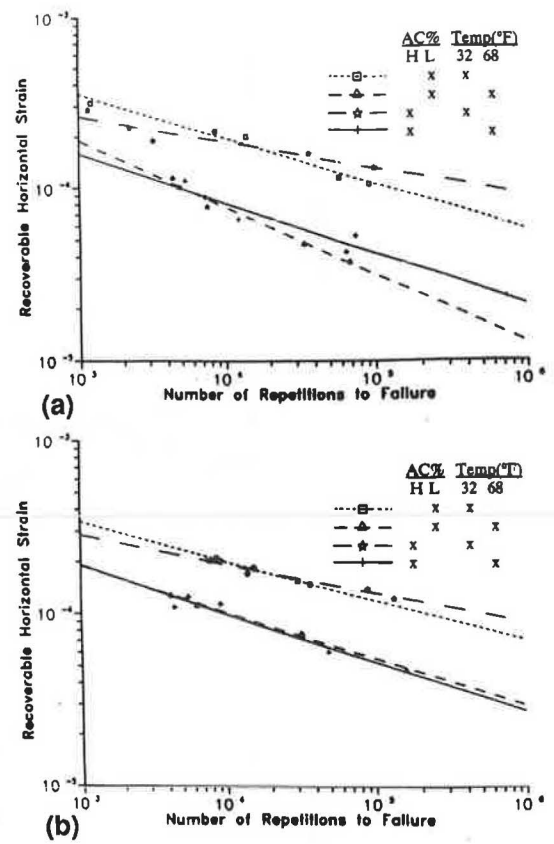


FIGURE 8 Diametral fatigue test results of AAG-1 asphalt mixtures with (a) high air void content and (b) low air void content.

1,000 cycles was necessary to compare findings from the figures and from ANOVA test results.

The ANOVA test procedure uses the *F*-value as the test statistic to test the null hypothesis that all the slopes or intercepts are the same. The level of significance for this test is the probability of having an *F*-value larger than the calculated *F*-value from a data set for the factor in question. A smaller value of this probability implies the heavier weight of the sample evidence for rejecting the null hypothesis. For example, in Table 3, a level of significance of $p = 0.0299$ on slope from asphalt content shows more evidence for the rejection of the null hypothesis, the slopes are the same, than does the statistical test on air voids with $p = 0.5725$. That is, a lower *p*-value means that the effect of the tested factor on a dependent variable (slope or intercept) is more significant.

The statistical results in Table 3 indicate that all four factors investigated in this study had a significant effect on slope or intercept or both. The following sections discuss the effect of individual factor on fatigue performance by using the engineering judgment from the figures and by comparing statistical test results. Normally, factors with a *p*-value less than 0.05 are considered to have a very significant effect on a dependent variable.

Effect of Temperature

The effect of temperature on the fatigue life of asphalt concrete with different mixture variables is shown in Figures 7

and 8. A significant effect of temperature on the fatigue performance was observed and was well supported by the *p*-values of 0.0041 and 0.0043 for slope and intercept in Table 3.

In Figures 7 and 8, the lines representing test data at 32°F (0°C) fall above the lines representing test data at 68°F (20°C), which contradicts a typical trend found from the plots of logarithms of initial bending strain versus number of cycles to

TABLE 2 SUMMARY OF K_1 AND K_2 VALUES

Sample ID abcd'	K_1	K_2
0000	3.00×10^9	3.484
0001	1.90×10^9	2.646
0010	3.82×10^{10}	4.184
0011	1.00×10^9	3.145
0100	1.01×10^{11}	5.208
0101	7.71×10^{10}	3.546
0110	2.07×10^{14}	4.808
0111	1.40×10^9	3.125
1000	5.40×10^{10}	4.405
1001	1.64×10^{11}	3.704
1010	7.51×10^{11}	3.802
1011	2.85×10^7	2.564
1100	7.95×10^{10}	5.952
1101	5.76×10^{11}	3.559
1110	2.72×10^{11}	6.579
1111	8.70×10^{11}	3.436

Note: 'a = Asphalt Type b = Asphalt Content
c = Air Voids d = Temperature

Levels of each factor are presented in Table 1.

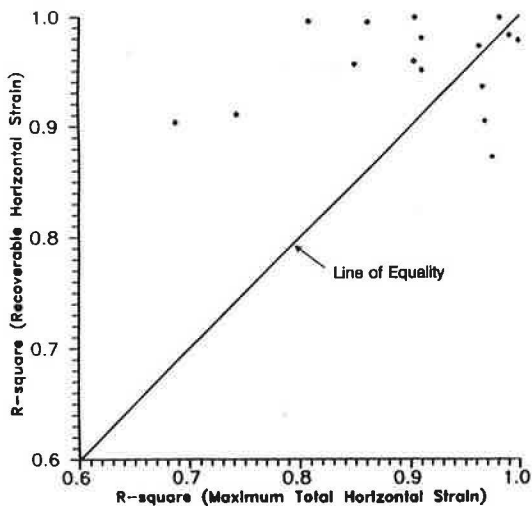


FIGURE 9 Comparison of R^2 values based on recoverable horizontal strain and maximum total horizontal strain.

TABLE 3 SUMMARY OF ANOVA TEST RESULTS

Factor	Level of Significance (p-value)	
	Dependent Variable	
	Slope	Intercept*
Asphalt Type	0.3044 ^b	0.0004
Asphalt Content	0.0299	0.0303
Air Voids	0.5725 ^b	0.0485
Temperature	0.0041	0.0043

* The values are the intercepts at the number of cycles to failure equal to 1000 cycles.

^b These values are considered to be insignificant.

failure from beam flexural tests. When the fatigue test data in Figure 7a were replotted in terms of maximum total horizontal strain and number of cycles to failure as shown in Figure 10, the lines for 32°F (0°C) and 68°F (20°C) test data fell in the reverse order of Figure 7a. The reason for this contradiction is that the major portion of total deformation at 32°F (0°C) is elastic (i.e., recoverable) while the elastic portion of total deformation at 68°F (20°C) is relatively small compared to that at 32°F (0°C).

Therefore, it was concluded that the asphalt concrete layer at 68°F (20°C) fails earlier than that at 32°F (0°C) for a given recoverable horizontal strain and vice versa for a given maximum total horizontal strain. These findings are in agreement with the research work by Ruth and Olson (4) in which they determined the elastic strain from incremental stress test and dynamic fatigue test using indirect tensile mode and concluded that the asphalt concrete has a greater resistance to larger elastic strains at lower temperatures. Also, the comparison between flexural fatigue data and indirect tensile fatigue data showed contradictory results. Ruth and Olson (4) stated that the reason for this contradiction was related to the method of testing. Because the beam specimen was returned to its

original position after application of each loading cycle, the measured elastic strain was excessive and not indicative of the actual dynamic elastic response of the material.

Kennedy (5) demonstrated that the fatigue characteristics obtained using the repeated-load indirect tensile test were comparable to the results obtained from other tests only if the biaxial state of stress in the indirect tensile test was accounted for. Therefore, care must be exercised in using the fatigue data presented in this paper as input to performance prediction models based on the data from beam flexural fatigue tests, such as the fatigue-cracking model used in the VESYS analysis. If the slopes and intercepts determined from Figures 7 and 8 were used in the VESYS analysis, the prediction would result in shorter fatigue life at the higher temperature. An effort was made to develop relationships between the fatigue constants based on these two strains so that the fatigue constants (K_1 and K_2) can be applied based on the recoverable horizontal strain to the fatigue-cracking model based on the maximum total horizontal strain. These relationships are shown in Figures 11 and 12. The correction factors for K_1 and K_2 were developed as follows based on the entire data set excluding one outlier shown in Figures 11 and 12:

$$K_{1,TOT} = 5245.5 K_{1,REC}^{0.821} \quad R^2 = 0.601$$

$$K_{2,TOT} = 0.0039 + 0.747 K_{2,REC} \quad R^2 = 0.596 \quad (7)$$

where

$$K_{i,TOT} = K_i \text{ based on maximum total horizontal strain,}$$

$$K_{i,REC} = K_i \text{ based on recoverable horizontal strain, and}$$

$$i = 1, 2.$$

These relationships must be used with great caution because of poor R^2 values and sensitive influence of these parameters on fatigue life of asphalt concrete.

Effect of Temperature Susceptibility

Figures 7 and 8 show the effect of temperature susceptibility of asphalt binder on the fatigue life of asphalt concrete mixture. AAK-1 asphalt is less temperature-susceptible than AAG-1 asphalt, and comparison between Figures 7a and 7b for AAK-1 asphalt mixtures and Figures 8a and 8b for AAG-1 asphalt mixtures with varying other conditions proves that the fatigue life of mixtures with AAK-1 asphalt is less temperature-susceptible than that of mixtures with AAG-1 asphalt. That is, the lines for 32°F (0°C) and 68°F (20°C) in Figure 8 are further apart than those in Figure 7, which shows greater temperature susceptibility of fatigue performance of mixtures with AAG-1 asphalt.

In order to statistically support this observation, fatigue data were divided into two sets by asphalt type, and the ANOVA test was performed on each data set. The results are as follows:

Asphalt Type	Level of Significance	
	Slope	Intercept
AAK-1	0.0116	0.8762
AAG-1	0.0185	0.0007

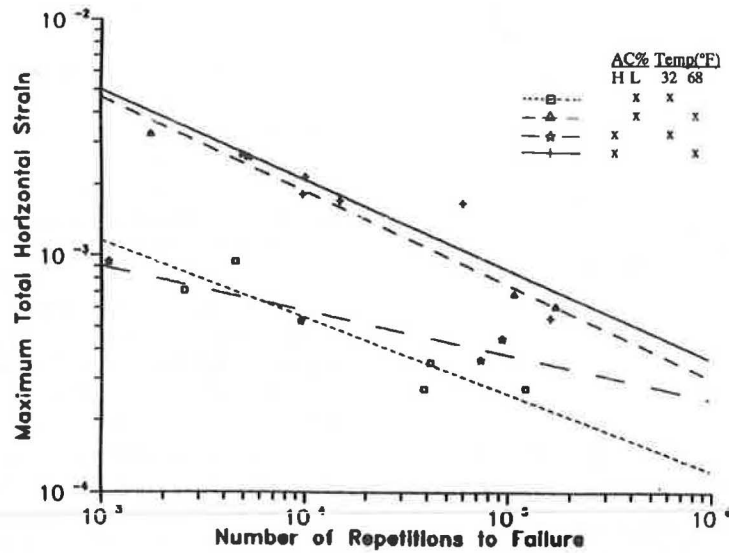


FIGURE 10 Diametral fatigue test results corresponding with Figure 7a (maximum total horizontal strain versus number of cycles to failure).

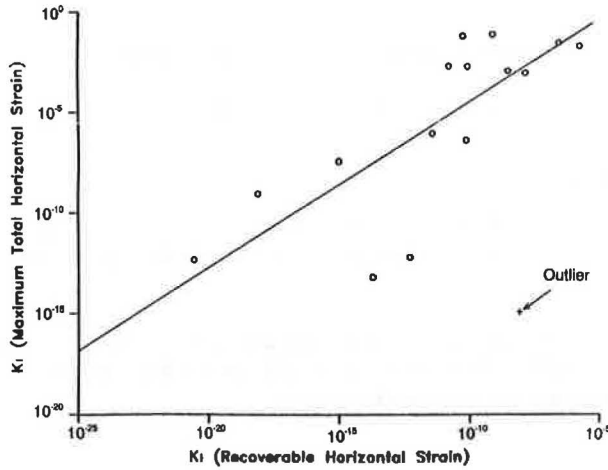


FIGURE 11 Relationship of K_1 values based on recoverable horizontal strain and maximum total horizontal strain.

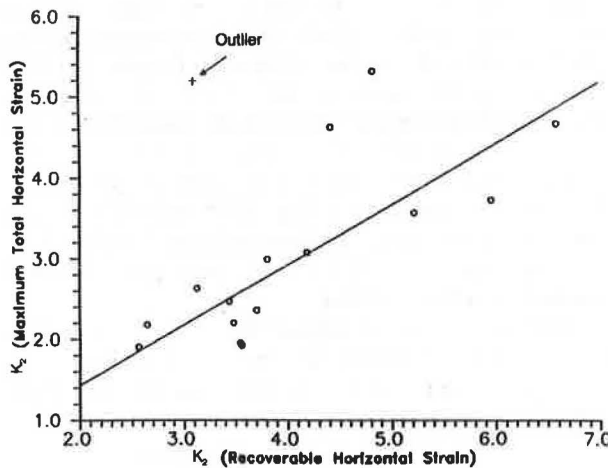


FIGURE 12 Relationship of K_2 values based on recoverable horizontal strain and maximum total horizontal strain.

The level of significance (p -value) of temperature on the slope remains almost the same for both asphalts; the level of significance of temperature on the intercept becomes much greater with AAG-1 asphalt than with AAK-1 asphalt. That is, the effect of temperature on the shift of fatigue lines is more significant with AAG-1 asphalt, which verifies the observation made from Figures 7 and 8.

Effect of Asphalt Content

The ANOVA test results shown in Table 3 indicate that the effect of asphalt content was significant for both slope and intercept. It was observed from Figures 7b and 8a that the increase in asphalt content reduced the slope of the fatigue line. However, the 0.6 percent increase in asphalt content did not make a significant difference in Figures 7a and 8b. This result might be explained by the test results reported by Barksdale (6). The fatigue properties of asphalt concrete base course mixes used by the Georgia Department of Transportation were measured using a rectangular beam specimen supported on an elastic rubber subgrade. The influence of asphalt content was evaluated for the mixture with 4.8 percent optimum asphalt content based on the Marshall mix design method. It was found that an increase in asphalt content from 4.25 to 4.5 percent increased the fatigue life by 350 percent and that an increase in asphalt content from 4.5 to 4.75 percent increased the fatigue life by 95 percent. Barksdale concluded that the improvement in fatigue resistance by increasing a specific percentage of asphalt content became less significant as the higher asphalt content was used. Because the asphalt contents used in this study are optimum contents from the modified Hveem mix design method (5.1 percent for AAK-1 asphalt and 4.9 percent for AAG-1 asphalt) and high asphalt contents, which were 0.6 percent higher than the optimum, the effect of increased asphalt content was not significant in some test conditions.

In order to identify the significance of the factors at different levels of temperature, fatigue data were divided into

two sets based on test temperature. The ANOVA tests were then conducted on each data set to investigate the significance of the factors (including asphalt type, asphalt content, and air voids) on slope or intercept at different temperatures. The results are shown in Table 4. According to the results in Table 4, the effect of asphalt content on both slope and intercept is much more significant at 32°F (0°C) than at 68°F (20°C). That is, a 0.6 percent increase in asphalt content in the mixture will provide more resistance to fatigue cracking at 32°F (0°C), which is the temperature range where fatigue cracking is of great concern.

Effect of Air Void Content

To clearly demonstrate the effect of air void content on fatigue life, the test data in Figures 7 and 8 were regrouped as shown in Figures 13 and 14. Figure 13 shows the data at 32°F (0°C), and Figure 14 shows them at 68°F (20°C). It was concluded that the effect of air void content on the fatigue life of the dense-graded asphalt concrete at 32°F (0°C) in Figure 13 was not as pronounced as at 68°F (20°C) in Figure 14. This observation was well supported by the statistical test results in Table 4 because the *p*-values of air voids on both slope and intercept were much smaller at 68°F (20°C) than at 32°F (0°C).

This behavior can be explained on the basis of crack initiation and propagation processes. As shown in Figure 5, the growth in horizontal deformation at 32°F (0°C) is almost negligible until a certain point, and then the growth becomes suddenly chaotic—which results in a complete failure of the specimen. Meanwhile, the growth in horizontal deformation at 68°F (20°C) in Figure 6 is more gradual. Therefore, one can easily draw a conclusion that the crack initiation is the major process at 32°F (0°C) and the crack propagation becomes more important as test temperature increases. With this concept as a background, it can be concluded that the effect of air void content on fatigue life at 68°F (20°C) is more pronounced than at 32°F (0°C) because, at the higher temperature, the failure is governed by crack propagation through void coalescence. In some materials with relatively strong bonding between matrix and reinforcing particles compared to particle strength, a large spherical cavity could decrease the stress concentration and, therefore, crack propagation rate. Conversely, when reinforcing particles are bonded to matrix relatively weakly compared to particle strength, which is the case of asphalt concrete, higher air void content provides better probability of void coalescence, faster crack propagation, and therefore, shorter fatigue life. Meanwhile, at 32°F

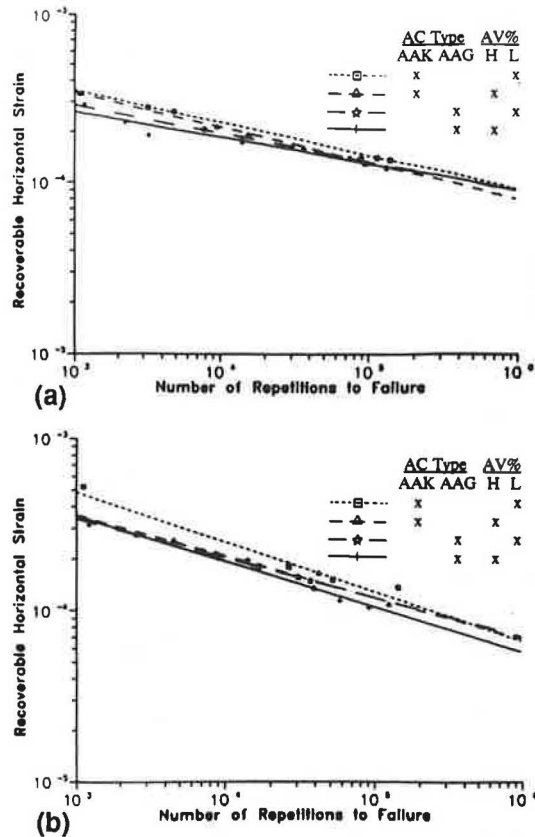


FIGURE 13 Diametral fatigue test results at 32°F (0°C) with (a) high asphalt content and (b) low asphalt content.

(0°C), the difference in the air void content may not influence the fatigue resistance because the failure is mainly governed by the crack initiation process.

Effect of Asphalt Type

Figures 13 and 14 show the effect of asphalt type on the fatigue performance of asphalt concrete. Mixtures with AAK-1 asphalt demonstrated higher resistance to fatigue than did mixtures with AAG-1 asphalt. Also, the difference in fatigue resistance of two asphalt mixtures is more pronounced at 68°F (20°C) than at 32°F (0°C). Table 4 shows the level of significance of asphalt type on slope and intercept at two different temperatures. Statistical results showed insignificant effect of asphalt type on slope at both temperatures, whereas the effect of asphalt type on intercept becomes more significant at 68°F (20°C) than at 32°F (0°C).

This behavior can be explained by comparing the penetration values of two asphalts at both temperatures. The following penetration values were extrapolated from the Bitumen Test Data Chart (penetration-viscosity-temperature chart) ($t^{\circ}F = (t^{\circ}C \div 0.55) + 32$):

Asphalt Type	At 32°F	At 68°F
AAK-1	8	49
AAG-1	3	33

TABLE 4 EFFECTS OF ASPHALT TYPE, ASPHALT CONTENT, AND AIR VOIDS ON FATIGUE PERFORMANCE AT DIFFERENT TEMPERATURES

Temperature	Level of Significance (p-value)					
	Asphalt Type		Asphalt Content		Air Voids	
	Slope	Intercept	Slope	Intercept	Slope	Intercept
32°F (0°C)	0.1610	0.0325	0.0118	0.0289	0.8676	0.2028
68°F (20°C)	0.6358	0.0012	0.2311	0.2651	0.4495	0.0995

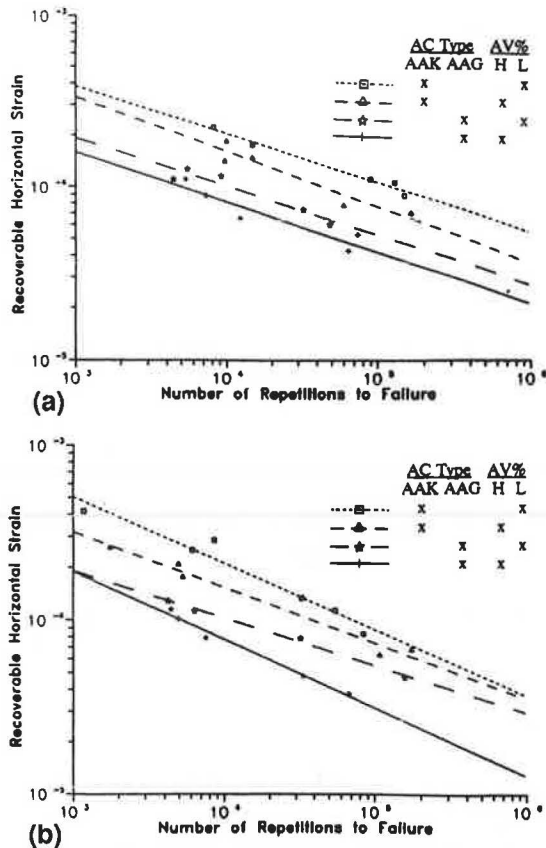


FIGURE 14 Diametral fatigue test results at 68°F (20°C) with (a) high asphalt content and (b) low asphalt content.

Within the temperature range studied in this research, AAK-1 asphalt is a softer binder which provides greater resistance to fatigue cracking. The difference in penetration values of both asphalts becomes greater at 68°F (20°C) than at 32°F (0°C). Therefore, the effect of asphalt type on the resistance to fatigue is more significant at 68°F (20°C) than at 32°F (0°C), as observed from Figures 13 and 14 and from the statistical test results.

CONCLUSIONS

It was proved that the diametral fatigue testing can be a promising tool in evaluating the fatigue performance of asphalt concrete under varying temperature and mixture variables. Within the limits of this study, the following principal conclusions can be drawn:

1. The value of 0.1 in. (0.25 cm) of maximum total horizontal deformation was found to be an appropriate failure criterion of the mixtures irrespective of the variables studied in this research.

2. The effect of mixture variables on fatigue life was better shown in the plots using recoverable horizontal strain.

3. Statistically, the effect of asphalt content was proved to be significant, although the data in some test conditions could not demonstrate the effect clearly. An increase of 0.6 percent above the optimum asphalt content statistically showed more significant improvement in fatigue resistance at 32°F (0°C) than at 68°F (20°C).

4. Assuming constant Poisson's ratio at different temperatures, it was found that temperature has significant effect on the fatigue life. For the given recoverable strain, lower temperature results in the longer fatigue life. For the given maximum total strain, the reverse is true.

5. An increase in air void content shortens the fatigue life with a more pronounced effect at 68°F (20°C) than at 32°F (0°C).

6. Temperature susceptibility of the asphalt binder has a pronounced effect on the fatigue life of asphalt concrete.

7. Asphalt concrete mixtures with a softer binder demonstrate the higher resistance to fatigue.

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