Effect of the Use of Modifiers on Performance of Asphaltic Pavements

N. Paul Khosla

The effects of two commercially available asphalt modifiers in improving the mechanical properties of asphalt paving mixtures were explored, and the abilities of these modifiers to mitigate pavement distress and improve overall pavement performance were evaluated. Nine binders were formulated with three asphalt grades and two modifiers. The modifiers investigated were carbon black and polymerized asphalt (styrelf). The dense-graded asphalt paving mixture specimens were subjected to dynamic and creep tests at temperatures of 0°, 40°, 70°, 100°, and 140°F, and the mechanical properties determined were resilient modulus, creep, permanent deformation parameters, and fatigue life. On the basis of these properties, the VESYS IIIA pavement performance prediction model was used to assess any improvement of the modifier on pavement distress and overall performance. The results indicate that the effect of the modifier on the paving mixture properties is not significant at low temperatures but pronounced at high temperatures. The predicted performance shows that carbon black is the most significant in reducing pavement rutting and polymer is the most significant in reducing fatigue cracking. Both modifiers show some degree of improvement in the overall pavement performance.

Choosing a suitable asphalt is of primary importance in the construction of a satisfactory pavement. The various grades and qualities of asphalt produced from different crudes and manufacturing processes show a marked difference in weathering resistance and durability. Because the highway engineer usually has a limited selection of asphalt, there is a need for a practical way to improve asphalt quality by adding ingredients. It is believed that asphalt cements modified with polymer (styrelf) and with carbon black may provide mixtures with improved durability, wear resistance, and general performance (*1–5*). Accordingly, the effectiveness of these two modified binders in improving the structural properties of asphaltic mixtures was verified, namely, their resilient modulus, fatigue life, resistance to rutting, and mitigation of low-temperature cracking.

MATERIALS AND SPECIMEN PREPARATION

The base asphalts used include AC-5, AC-10, and AC-20. These asphalts were sent to the Elf Aquitaine Laboratory in Terre Haute, Indiana, for blending with polymer. After blending, the modified asphalt is called "styrelf." Styrelf is a polymer system in which the polymer (styrene-butadiene block copolymer) undergoes an irreversible chemical reaction with asphalt through a patented vulcanizing process. MICROFIL8—pelletized carbon black using 8 percent maltenes—was also blended with each of the base asphalts. The amount of MICROFIL8 that should be used depends on the desired consistency of the binder and the viscosity of the original asphalt. Thus, to have the desired stiffening effect, a slightly higher amount of modifier is mixed with softer asphalts. Consequently, AC-5 was blended with 12 percent MICROFIL8, and AC-10 and AC-20 were blended with 10 percent MICROFIL8. The properties of the binders are summarized in Table 1.

Dense-graded granite aggregate was used. The gradation of the aggregates conforms to the North Carolina specifications for I-1 mix. The gradation of the aggregates is as follows:

Sieve Size	*Percent Passing*
3/4 in.	100
1/2 in.	96
3/8 in.	92
No. 4	66
No. 8	50
No. 16	39
No. 40	24
No. 80	10
No. 200	3

The mixtures were designed in accordance with the Marshall method of mix design (*6*); a summary of the mixture properties is given in Table 2. The specific gravity of the base asphalts ranged from 1.020 to 1.035 and the specific gravity of polymer-modified binders ranged from 1.029 to 1.044; the specific gravity of the pelletized carbon black was assumed to be 1.740. To keep the volume of the binder constant in all the mixtures, the binder contents used in the mixtures were varied, as shown in Tables 2 and 3.

Specimens for the resilient modulus and fatigue tests were 4 in. in diameter and 2.5 in. high and fabricated using the mechanical Marshall compactor. Specimens for the creep tests were 4 in. in diameter and 8 in. high and prepared with a modified Rainhart mechanical compactor. The mixture was compacted in four layers. The sequence of weight of the mixture, starting at the bottom was 1200, 1000, 800, and 600 g, and the corresponding numbers of blows on each layer were 60, 70, 80, and 85. This procedure was developed as part of a previous study (*7*) to provide uniform density from top to bottom of the specimen and produce a specimen with the same density as that prepared by the mechanical Marshall compactor.

Department of Civil Engineering, North Carolina State University, Box 7908, Raleigh, N.C. 27695.

TABLE 1 BINDER PROPERTIES

Property \ Binder	AC-5	AC-5+P	AC-5+ 12%C	AC-10	AC-10+P	AC-10+ 10%C	AC-20	AC-20+P	AC-20+ 10%C
penetration, 100g, 5 sec, 77°F	140	102	104	130	89	122	82	61	72
viscosity, abs, 140°F, P	741	2940	2250	1050	4285	1900	2220	8255	3840
viscosity, kin, 275°F, cst	315	544	616	412	797	614	502	1178	1170
Pen - vis number (PVN)pp	-0.2	0.3	0.5	0.2	0.7	0.7	-0.1	0.8	1.0

TABLE 2 SUMMARY OF MIXTURE PROPERTIES

Property \ Binder	AC-5	AC-10	AC-20
Percent asphalt by wt. aggregate	5.87	5.88	5.94
Percent asphalt by wt. mixture	5.55	5.56	5.61
Mix bulk Sp. Gr.	2.30	2.30	2.30
Absorbed asphalt (percent)	0.70	0.70	0.70
Aggregate bulk Sp. Gr.	2.58	2.58	2.58
Asphalt Sp. Gr.	1.020	1.023	1.035
VMA (percent)	15.78	15.71	15.85
Air Void (percent)	4.75	4.80	4.86
Marshall Stability (lb)	2190	2380	2550
Flow (0.01 IN)	11	10	10

TEST PROCEDURES

The procedures for creep, fatigue, and resilient modulus tests are briefly described. A detailed description of the test procedures and equipment is given elsewhere (*7*).

Creep Test

The incremental static creep tests were conducted on specimens 4 in. in diameter and 8 in. high to determine the permanent deformation coefficients to be used in the VESYS computer program. Test specimens at temperatures of −20°, 0°, 20°, 40°, 70°, 90°, and 120°F were tested under a creep stress of 20 psi. However, specimens at 140°F were tested under a creep stress of 10 psi. Furthermore, specimens made with AC-5 and AC-10 were too soft to be tested even under a creep stress of 10 psi at 120° and 140°F, so no creep test data were generated for these mixture specimens at these temperatures.

The total permanent strain at the end of each rest period following load increment, as shown in Figure 1, was calculated. The total permanent strain versus the incremental time of loading was plotted on log-log paper, as shown in Figure 2. A best-fit straight line is constructed through the plotted points intersecting the vertical axis at a 0.1-sec loading time. The permanent deformation coefficients of layer material were calculated as follows:

$$\mu = \frac{IS}{e} \qquad \alpha = 1 - S$$

where e is the resilient strain. The resilient strain is determined from a repetitive loading test conducted after the creep test.

TABLE 3 BINDER SPECIFIC GRAVITY AND BINDER CONTENT OF MIXTURES

Property \ Binder	AC-5+P	AC-5+C	AC-10+P	AC-10+C	AC-20+P	AC-20+C
Specific Gravity	1.029	1.107	1.036	1.095	1.044	1.098
% Asphalt by wt. agg.	5.929	6.389	5.955	6.306	5.992	6.361
% Asphalt by wt. mix.	5.599	6.005	5.647	5.932	6.653	5.981

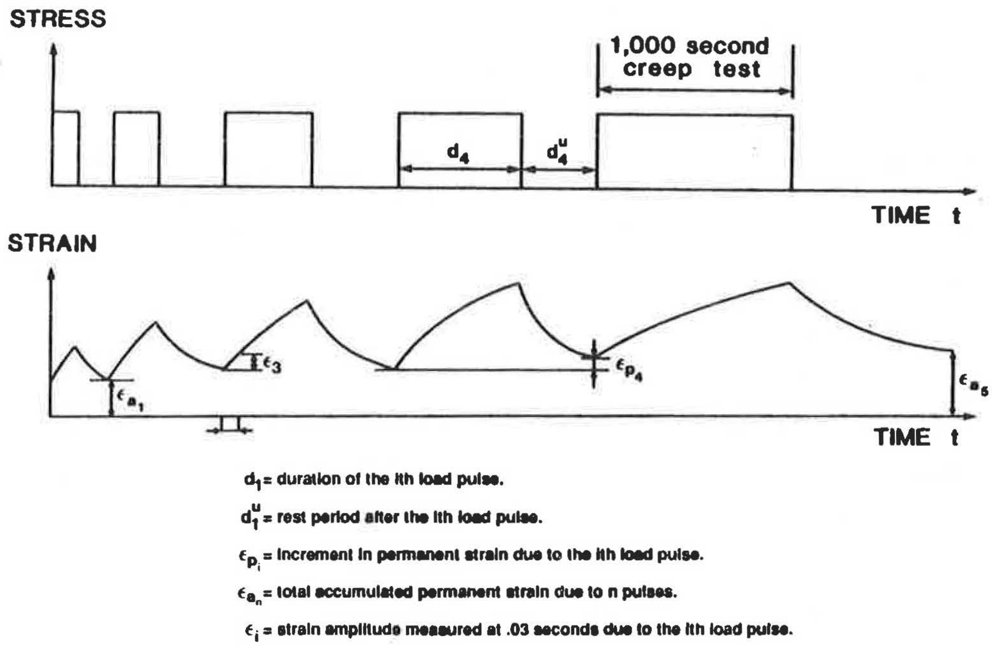

FIGURE 1 Stress and strain of incremental static test series.

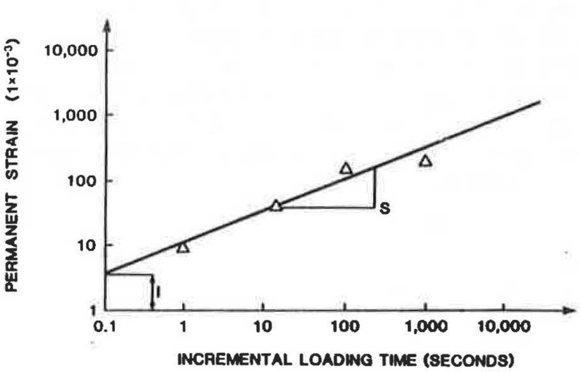

FIGURE 2 Permanent strain curve using incremental static creep test.

Resilient Modulus Test

The resilient modulus tests were conducted on diametral specimens (4 in. × 2.5 in.) in the indirect tension mode at 0°, 40°, 70°, 100°, and 140°F. A test load was applied along the vertical diameter of the test specimen, and the corresponding deformation was measured across the horizontal diameter. The resilient modulus was calculated as follows:

$$M_R = \frac{P(\nu + 0.2734)}{Dt}$$

where

P = load in (lb),

ν = Poisson's ratio (assumed to be 0.35),
t = thickness of specimen (in.), and
D = total horizontal resilient deformation.

Fatigue Test

Fatigue response of the mixtures was measured on diametral specimens in the indirect tension mode. The controlled stress mode of loading with square wave form was used, which included a 0.1-sec loading period and 2.9-sec unloading period. Stresses in the range of 15 to 50 psi were used, and the tests were conducted at 70°F.

In the fatigue test, a repetitive load was applied, and the initial strain was measured between 100 and 200 repetitions. The number of cycles to failure (N_f) was represented by the load's reaching a limiting value of 70 percent of the original load. The number of cycles to failure (N_f) versus the initial tensile strain (e) was plotted on log-log paper. Then a best-fit straight line was drawn through the points. The slope of the line was denoted as S, and the strain value corresponding to 100 load applications was denoted as I. The straight line was shifted horizontally to the right by a factor of 13.03, as suggested by Finn (*8*), to adjust for field fatigue properties. This is because laboratory tests do not take into account several important factors, such as healing of the pavement between stress applications, rest time between stresses, and variability in the position of the load within the wheel path resulting in a reduction of stress due to the passage of a certain number of vehicles. The mean value of fatigue coefficients was calculated as follows:

$$K_2 = \frac{1}{S} \qquad K_1 = 100(I)^{1/S}$$

ANALYSIS OF TEST RESULTS

The analysis of the data and plots of the test results pertaining to resilient modulus, creep, and fatigue characteristics are described. Two replicate specimens for each test condition were used.

Creep Test Results

As discussed, the total permanent strain at the end of each rest period was plotted on a log-log paper at a function of incremental loading times: 0.1, 1, 10, 100, and 1,000 sec. The permanent deformation plots of incremental static loading tests at 40°F and 100°F are shown in Figures 3 and 4.

An analysis of the plot in Figure 3 reveals that the mixtures containing the conventional asphalts AC-5, AS-10, and AC-20 exhibited significantly higher deformation than the mixtures modified with polymer and carbon black. Thus, it can be inferred that these modified mixtures will result in less rutting. An analysis of the rutting potential of these mixtures supports this premise.

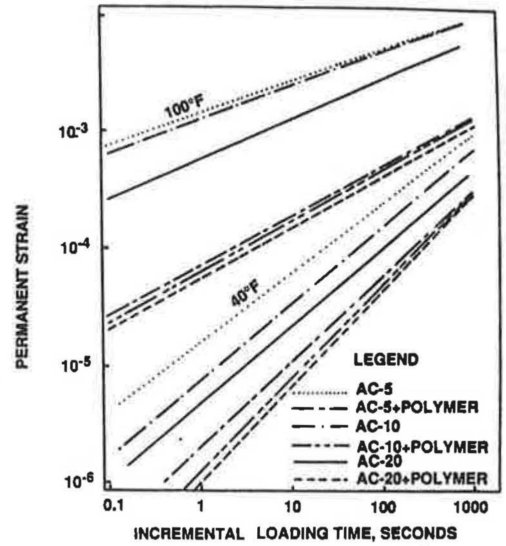

FIGURE 3 Permanent strain from incremental static loading test: AC-5, AC-5 + P, AC-10, AC-10 + P, AC-20, and AC-20 + P.

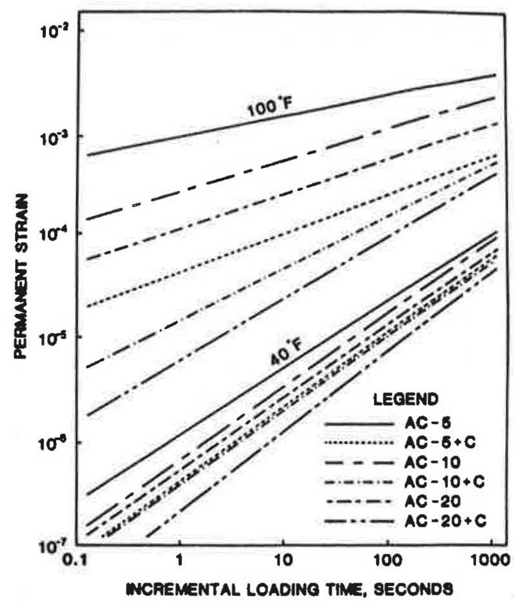

FIGURE 4 Permanent strain from incremental static loading test: AC-5, AC-5 + C, AC-10, AC-10 + C, AC-20, and AC-20 + C.

Resilient Modulus Test Results

The plot of the resilient modulus values versus temperature for the mixture types is shown in Figures 5 and 6. Because the mixture specimens made with AC-5 and AC-10 were too soft to be tested above 100°F, no resilient modulus data were generated for these mixtures at 120°F and 140°F. The data in Figures 5 and 6 indicate that at low temperatures, mixtures made with AC-5, AC-10, AC-5 + P, AC-10 + P, AC-5 + C, and AC-10 + C have almost the same modulus values, whereas

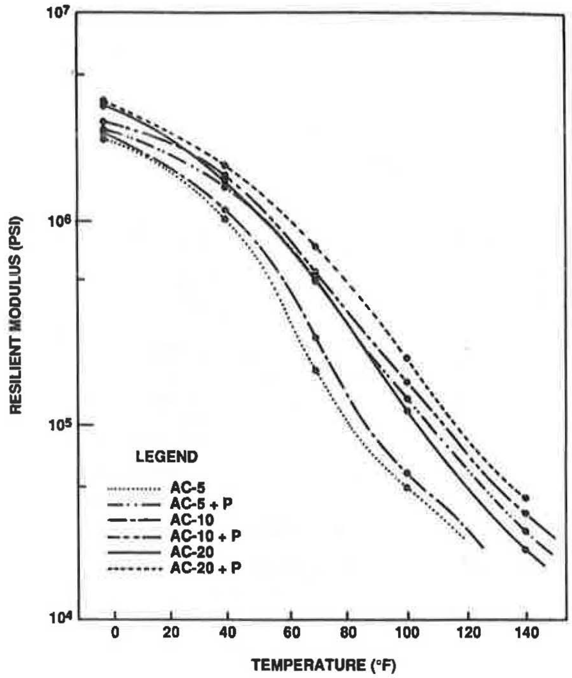

FIGURE 5 Resilient modulus versus temperature: AC-5, AC-5+C, AC-10, AC-10+C, AC-20, and AC-20+C.

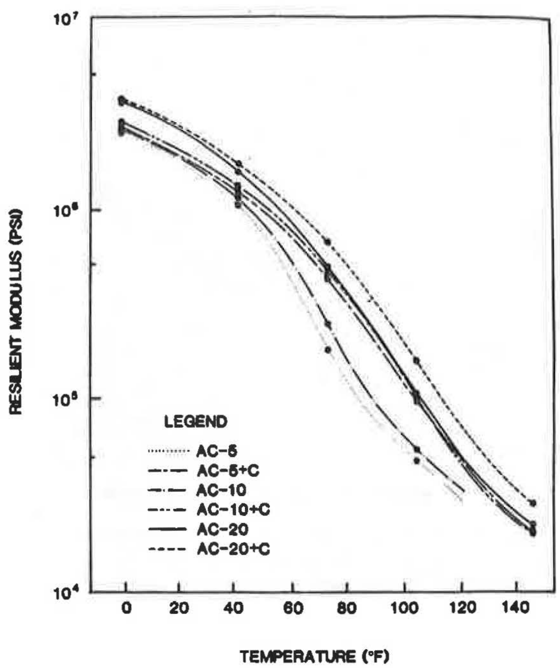

FIGURE 6 Resilient modulus versus temperature: AC-5, AC-5+P, AC-10, AC-10+P, AC-20, and AC-20+P.

AC-20, AC-20+P, and AC-20+C provide mixtures with slightly higher modulus values. At high service temperatures, mixtures made with AC-5 and AC-10 have the lowest resilient modulus values. In addition, at temperatures above 100°F, mixtures made with AC-5+P, AC-10+P, AC-5+C, AC-10+C, and AC-20 have no significant difference in their resilient modulus values, whereas AC-20+P and AC-20+C provide mixtures with the highest resilient modulus values. It can be concluded that adding polymer and carbon black to the asphalt reduces the temperature susceptibility and gives mixtures higher resilient modulus at high temperatures without affecting the modulus values at low temperatures.

Fatigue Test Results

The fatigue lives are plotted in Figures 7 and 8. The data in Figures 7 and 8 suggest that mixtures containing AC-5 and AC-10 have relatively shorter fatigue lives than other mixtures. Furthermore, the mixtures made with AC-5+P, AC-

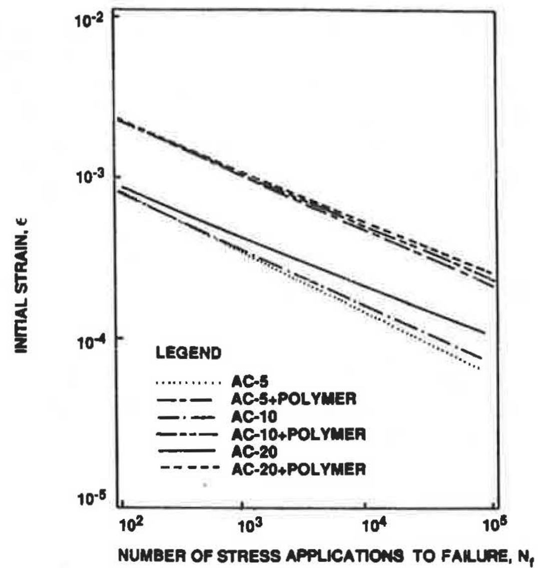

FIGURE 7 Strain versus cycles to failure at 70°F: AC-5, AC-5+P, AC-10, AC-10+P, AC-20, and AC-20+P.

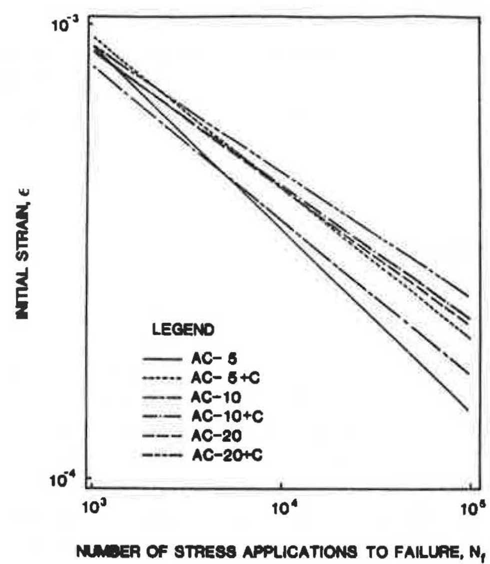

FIGURE 8 Strain versus cycles to failure at 70°F: AC-5, AC-5+C, AC-10, AC-10+C, AC-20, and AC-20+C.

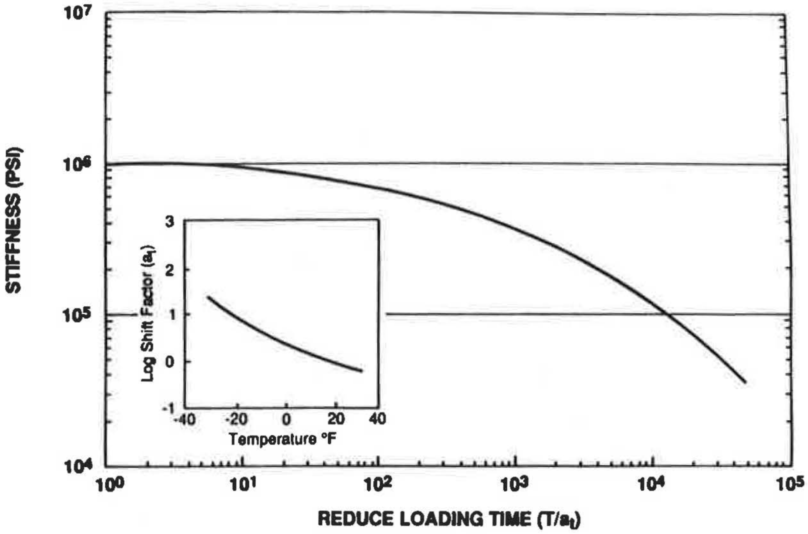

FIGURE 9 **Master curve for AC-5.**

10+P, AC-5+C, AC-10+C, and AC-20 exhibit similar fatigue life. AC-20+P and AC-20+C provide mixtures with the longest fatigue life. This trend is reflected in and supported by the analysis of performance of a representative pavement section.

Low-Temperature Characteristics

The low-temperature cracking phenomenon is a function of ambient air temperature and thermal loading time. Loading times in the range of a half-hour to several hours are considered reasonable. McLeod (*9*) has suggested a loading time of 20,000 sec as the nominal rate at which the pavement may be stressed because of chilling to low temperature. The experimental stiffness values from creep tests at −20°, 0°, 20°, and 40°F were measured up to a loading time of roughly 1,000 sec. To extend the data to the loading time of 20,000 sec at low temperatures, time-temperature superposition techniques were used. The experimental stiffness curves were moved along the loading time axis until all of them were superimposed and formed one master curve. An example of such a master curve for stiffness versus reduced loading time for mixture with AC-5 binder, along with the corresponding shift factor (a_t) versus temperature curve shown as an inset, is shown in Figure 9.

Similar master curves were prepared for other mixture types, and, for the loading time of 20,000 sec, the stiffness values at −30°, −20°, −10°, and 0°F were calculated and are reported in Table 4. To eliminate transverse pavement cracking, McLeod (*9*) has suggested limiting stiffness values given in Table 5. In this regard, the stiffness values in Table 4 suggest that at −30°F, only the mixtures containing AC-5, AC-5+P, AC-10+P, and AC-5+C would be able to mitigate low-temperature cracking. At −20°F and −10°F, in addition to the aforementioned binders, mixtures with AC-10 and AC-10+C would also be able to perform well. At 0°F, all the binders except AC-20 would be able to eliminate transverse pavement cracking under these criteria.

ANALYSIS OF PERFORMANCE OF REPRESENTATIVE PAVEMENT WITH AND WITHOUT POLYMER MODIFICATION

In order to assess the influence of the modified mixtures on the pavement performance, a representative pavement section was selected for analysis. The composition of this pavement section is shown in Figure 10. The VESYS IIIA structural subsystem was used to predict the pavement performance (*10*). On the basis of the mechanical properties of the materials in different layers of the flexible pavement, a typical traffic volume, and different environmental conditions, VESYS IIIA predicts the performance of a given pavement in terms of rutting, cracking, and present serviceability index.

The mechanical properties for the granular base course and the subgrade were determined as a part of a previous study (*7*) and are given in Tables 6 and 7. In addition, the mechanical properties of the mixtures for the surface course with and without modifiers are given in Tables 8 and 9. An analysis period of 20 years was chosen for this purpose; a hypothetical value of average daily traffic of 110 equivalent 18-kip axle loads (EAL_{18}) was selected for the analysis period. To predict the performance of the selected representative pavement under different environmental conditions, four regions were selected as shown in Table 10.

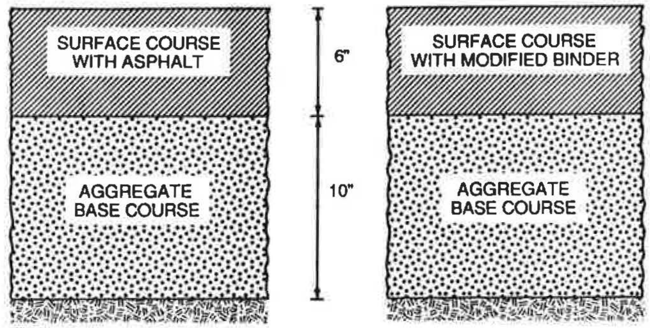

FIGURE 10 **Layer thickness of representative pavement section.**

TABLE 4 STIFFNESS VALUES IN POUNDS PER SQUARE INCH FOR LOADING TIME OF 20,000 SEC

	Temperature (°F)			
Binder	**-30**	**-20**	**-10**	**0**
AC5	$3.5x10^5$	$2.7x10^5$	$1.8x10^5$	$3.4x10^4$
AC5+P	$2.4x10^5$	$1.0x10^5$	$7.2x10^4$	$1.6x10^4$
AC5+C	$2.9x10^5$	$2.1x10^5$	$7.8x10^4$	$3.1x10^4$
AC10	$7.32x10^5$	$2.9x10^5$	$1.9x10^5$	$6.1x10^4$
AC10+P	$4.1x10^5$	$1.4x10^5$	$9.3x10^4$	$3.4x10^4$
AC10+C	$6.7x10^5$	$2.4x10^5$	1.10^5	$5.7x10^4$
AC20	$1.6x10^6$	$8.7x10^5$	$6.5x10^5$	$1.7x10^5$
AC20+P	$9.1x10^5$	$5.1x10^5$	$3.3x10^5$	$8x10^4$
AC20+C	$1.5x10^6$	$8.1x10^5$	$6.3x10^5$	$1.6x10^5$

TABLE 5 INFLUENCE OF MINIMUM PAVEMENT TEMPERATURE ON PAVEMENT MODULUS OF STIFFNESS TO BE USED FOR SELECTING ASPHALT GRADE (*9*)

Minimum Temperature of the Pavement (°F)	**Stiffness at Which Transverse Cracking Can be Expected**	**Pavement at Which Transverse Cracking Can be Eliminated**
-30	800,000	400,000
-20	600,000	300,000
-10	400,000	200,000
0	250,000	125,000

TABLE 6 RESILIENT MODULUS VALUES IN POUNDS PER SQUARE INCH FOR NONASPHALTIC LAYERS (*7*)

	Temperature (°F)					
Layer	**0**	**40**	**70**	**90**	**120**	**140**
Granular Base Course	54,000	27,000	26,000	27,000	29,000	29,000
Subgrade Soil	8,000	4,000	3,000	9,000	9,000	7,000

TABLE 7 PERMANENT DEFORMATION PARAMETERS FOR NONASPHALTIC LAYERS (7)

Temperature (°F) / Layer	0		40		70		90		120		140	
Parameter / Layer	μ	α	μ	α	μ	α	μ	α	μ	α	μ	α
Granular Base Course	.01	.81	.01	.81	.01	.84	.005	.87	.005	.87	.005	.87
Subgrade Soil	.16	.85	.16	.85	.16	.85	.04	.72	.06	.72	.06	.72

TABLE 8 RESILIENT MODULUS VALUES IN THOUSANDS OF POUNDS PER SQUARE INCH FOR I-1 MIX

Temperature (°F) / Binder	0	40	70	90	120	140
AC5	2,273	1,170	208	98	28	*
AC5+P	2,466	1,457	485	268	81	34
AC5+C	2,523	1,278	480	234	90	23
AC10	2,328	1,228	283	129	31	*
AC10+P	2,664	1,687	599	315	85	38
AC10+C	2,643	1,300	488	238	91	22
AC20	2,647	1,620	517	243	75	23
AC20+P	3,272	2,024	853	483	123	45
AC20+C	2,657	1,706	712	327	120	33

* Not Available

TABLE 9 PERMANENT DEFORMATION PARAMETERS FOR I-1 MIX

Temperature (°F)	0		40		70		90		120		140	
Parameter / Binder	μ	α	μ	α	μ	α	μ	α	μ	α	μ	α
AC5	.031	.756	.297	.404	.512	.673	.709	.710	.717	.728	*	*
AC5+P	.035	.743	.054	.270	.126	.542	.506	.562	.249	.525	.269	.697
AC5+C	.024	.762	.031	.175	.080	.513	.134	.697	.130	.620	.203	.724
AC10	.028	.757	.148	.34	.495	.636	.560	.686	.705	.715	*	*
AC10+P	.028	.753	.038	.218	.128	.517	.520	.588	.250	.546	.263	.689
AC10+C	.023	.773	.031	.174	.068	.603	.118	.634	.132	.640	.204	.748
AC20	.027	.762	.128	.34	.411	.617	.522	.665	.597	.674	.604	.690
AC20+P	.024	.760	.036	.201	.102	.514	.190	.511	.237	.544	.250	.656
AC20+C	.021	.793	.022	.140	.060	.640	.110	.510	.141	.650	.169	.741

* Not Available

TABLE 10 SEASONAL TEMPERATURE IN DEGREES FAHRENHEIT

Region	Winter	Spring	Summer	Fall
1	0	40	90	70
2	40	70	90	70
3	40	70	120	70
4	40	70	140	90

Rut Depth

Rut depth is a measure of the permanent deformation in the wheel path created by traffic loads. In the rut depth model, rutting is primarily a function of the laboratory-determined permanent deformation characteristics and stiffness of the materials in pavement layers and of the truck-traffic volume. The predicted rut depths for the different cases of the binders are shown in Table 11.

The pavement sections constructed with AC-5, AC-10, and AC-20 will reach the limiting value of rut depth of 0.6 in. in 4 to 6 years of service life. The same pavement sections constructed with the polymer-modified binders will take 8 to 10 years to reach a rut depth of 0.6 in., and those constructed with carbon black–modified binders will take 10 to 12 years to reach a depth of 0.6 in.

Fatigue Cracking

Table 12 indicates that the mixtures made with polymer-modified binders exhibit a much lower fatigue damage index than the conventional asphalts and carbon black–modified asphalts.

The predicted cracking index using the VESYS IIIA program indicates the expected fatigue cracking. The predicted

TABLE 11 RUT DEPTH IN INCHES

Region	Binder	1	5	10	15	20
1	AC5	.26	.56	.78	.95	1.01
	AC5+C	.17	.39	.56	.69	.80
	AC5+P	.23	.47	.64	.77	.87
	AC10	.22	.48	.67	.82	.94
	AC10+C	.16	.38	.55	.68	.79
	AC10+P	.21	.45	.61	.74	.84
	AC20	.20	.43	.60	.73	.84
	AC20+C	.15	.32	.52	.62	.72
	AC20+P	.18	.39	.55	.68	.78
2	AC5	.31	.66	.92	1.1	1.3
	AC5+C	.18	.41	.58	.72	.83
	AC5+P	.24	.50	.68	.81	.92
	AC10	.28	.58	.80	.97	1.11
	AC10+C	.18	.41	.58	.71	.77
	AC10+P	.22	.46	.63	.75	.86
	AC20	.23	.48	.66	.80	.92
	AC20+C	.17	.37	.54	.66	.77
	AC20+P	.18	.41	.58	.70	.81
3	AC5	.37	.73	.98	1.18	1.34
	AC5+C	.20	.44	.63	.77	.89
	AC5+P	.25	.52	.71	.84	.95
	AC10	.35	.67	.89	1.06	1.21
	AC10+C	.18	.41	.59	.73	.84
	AC10+P	.23	.48	.64	.80	.91
	AC20	.26	.52	.71	.84	.96
	AC20+C	.16	.37	.52	.65	.75
	AC20+P	.20	.44	.62	.73	.83
4	AC5	*	*	*	*	*
	AC5+C	.21	.46	.65	.80	.92
	AC5+P	.27	.53	.72	.89	.98
	AC10	*	*	*	*	*
	AC10+C	.20	.46	.65	.79	.92
	AC10+P	.24	.48	.66	.77	.87
	AC20	.36	.65	.80	.98	1.10
	AC20+C	.19	.42	.59	.73	.84
	AC20+P	.20	.46	.63	.76	.89

Time (Year) columns: 1, 5, 10, 15, 20.

* Not Available

values are mainly a function of fatigue curve parameters K_1 and K_2, primary response properties, traffic loading, pavement temperature variations, and layer thickness. The cracking index is a dimensionless parameter that estimates the occurrence of fatigue cracking; a value of 1 corresponds to the time when the cracks are initiated at the bottom of the asphaltic layer. The predicted cracking indexes for different regions are shown in Table 12.

The sections constructed with AC-5 and AC-10 will show severe cracking in the first 5 years of service life in all the regions; in the same period, AC-20 will show severe cracking in Regions 3 and 4 only. After 20 years of service life, pave-

TABLE 12 FATIGUE CRACKING DAMAGE INDEX

Region	Time (Year) / Binder	1	5	10	15	20
	AC5	.93	4.68	9.36	14.04	18.73
	AC5+C	.12	.59	1.17	1.76	2.34
	AC5+P	.01	.05	.10	.15	.20
	AC10	.55	2.74	5.48	8.22	10.96
1	AC10+C	.09	.45	.91	1.36	1.82
	AC10+P	.004	.023	.046	.068	.091
	AC20	.073	.366	.731	1.09	1.46
	AC20+C	.03	.15	.29	.44	.59
	AC20+P	.002	.009	.018	.026	.035
	AC5	1.15	5.77	11.54	17.32	23.05
	AC5+C	.09	.44	.88	1.32	1.75
	AC5+P	0.11	.055	.110	.165	.220
	AC10	.675	3.37	6.74	10.12	13.49
2	AC10+C	.07	.35	.70	1.05	1.39
	AC10+P	.005	.025	.050	.075	.099
	AC20	.092	.45	.92	1.37	1.83
	AC20+C	.03	.13	.26	.40	.53
	AC20+P	.002	.010	.019	.028	.037
	AC5	3.54	17.74	35.48	53.22	70.96
	AC5+C	.39	1.94	3.87	5.81	7.75
	AC5+P	.039	.197	.394	.590	.78
3	AC10	3.02	15.14	30.28	45.43	60.57
	AC10+C	.31	1.57	3.13	4.70	6.26
	AC10+P	.027	.136	.272	.407	.543
	AC20	.561	2.80	5.60	8.40	11.21
	AC20+C	.18	.92	1.83	2.75	3.67
	AC20+P	.012	.058	.117	.175	.233
	AC5	*	*	*	*	*
	AC5+C	3.21	15.68	31.36	47.12	54.84
	AC5+P	1.75	8.79	17.5	26.38	35.17
4	AC10	*	*	*	*	*
	AC10+C	3.6	17.84	35.68	53.68	72.28
	AC10+P	1.24	6.20	12.41	18.61	24.82
	AC20	4.51	22.59	45.19	66.77	90.39
	AC20+C	2.80	14.00	28.08	42.08	56.8
	AC20+P	.85	4.27	8.54	12.82	17.09

* Not Available

ment sections constructed with AC-5 + P, AC-10 + P, and AC-20 + P will be expected to have almost no cracking in Regions 1, 2, and 3. However, for Region 4, severe cracking will be anticipated in the first 5 years of service life.

For carbon black–modified mixtures, pavement sections constructed with AC-5 + C and AC-10 + C will experience slight to moderate cracking in 10 to 15 years in Regions 1 and 2, and Regions 3 and 4 will have moderate cracking in approximately 5 years. Sections using AC-20 + C will not be expected to have any cracking in 20 years in Regions 1 and 2, but they will experience moderate cracking in approximately 12 years in Region 3 and 5 years in Region 4.

Present Serviceability Index

The present serviceability index gives an indication of ridability of pavement in relation to present pavement condition. The mean serviceability index at time zero was assigned a value of 4.5, and the terminal serviceability index was assigned a value of 2.5.

A summary of present serviceability indexes is shown in Table 3. Data in Tables 13 and 14 indicate that the service life of the pavement sections will be influenced greatly by the choice of the binder. For the aforementioned service and environmental conditions, AC-5, AC-10, and AC-20 will provide pavements with 5 to 15 years of service life. The pavement sections constructed with polymer-modified binders will

TABLE 13 PRESENT SERVICEABILITY INDEX

Region	Binder	Time (Year) 1	5	10	15	20
	AC5	4.00	3.10	2.30	1.68	1.08
	AC5+C	4.37	3.88	3.09	2.62	2.21
	AC5+P	4.26	3.65	3.05	2.57	2.16
	AC10	4.24	3.33	2.71	2.18	1.70
1	AC10+C	4.38	3.88	3.24	2.64	2.23
	AC10+P	4.29	3.71	3.13	2.67	2.27
	AC20	4.32	3.73	3.17	2.50	2.07
	AC20+C	4.36	3.81	3.27	2.81	2.40
	AC20+P	4.34	3.79	3.26	2.82	2.42
	AC5	3.84	2.74	1.80	1.00	.268
	AC5+C	4.21	3.69	3.11	2.49	2.07
	AC5+P	4.22	3.53	2.43	2.44	2.01
	AC10	4.06	2.99	2.21	1.55	.96
2	AC10+C	4.22	3.70	3.13	2.54	2.07
	AC10+P	4.25	3.63	3.08	2.62	2.22
	AC20	4.24	3.57	2.83	2.17	1.73
	AC20+C	4.21	3.66	3.16	2.69	2.29
	AC20+P	4.32	3.76	3.23	2.78	2.38
	AC5	3.64	2.50	1.53	.71	.03
	AC5+C	4.30	3.41	2.82	2.32	1.86
	AC5+P	4.21	3.47	2.81	2.27	1.73
	AC10	3.69	2.68	1.86	1.17	.55
3	AC10+C	4.34	3.50	2.94	2.47	2.05
	AC10+P	4.24	3.58	2.98	2.49	2.05
	AC20	4.08	3.16	2.54	2.04	1.59
	AC20+C	4.37	3.72	3.13	2.73	2.36
	AC20+P	4.29	3.67	3.10	2.62	2.19
	AC5	*	*	*	*	*
	AC5+C	4.00	3.36	2.75	2.23	1.75
	AC5+P	3.90	3.13	2.53	2.04	1.61
	AC10	*	*	*	*	*
4	AC10+C	4.00	3.38	2.76	2.23	1.74
	AC10+P	3.98	3.29	2.75	2.31	1.92
	AC20	3.65	2.74	2.05	1.49	1.00
	AC20+C	4.03	3.48	2.94	2.48	2.06
	AC20+P	4.09	3.40	2.87	2.43	2.04

* Not Available

TABLE 14 PREDICTED SERVICE LIFE IN YEARS

Region	AC5	AC5+P	AC5+C	AC10	AC10+P	AC10+C	AC20	AC20+P	AC20+C
1	8.83	15.90	16.44	11.96	17.13	16.70	15.10	19.01	18.79
2	6.24	14.39	14.91	8.04	16.55	15.38	11.94	18.53	17.34
3	5.02	12.81	13.12	6.04	14.92	14.70	10.40	16.39	18.09
4	*	10.32	12.34	*	12.76	12.41	6.63	14.21	14.72

* Not Available
(TERMINAL PSI = 2.5)

have a service life of 10 to 19 years, and those constructed with carbon black–modified binders will have a service life of 12 to 19 years.

CONCLUSIONS

The following principal conclusions can be drawn:

1. Modifying asphalt with polymer or carbon black reduces the temperature susceptibility of the binders. In this regard, the mixtures made with styrelf and carbon black–modified binders provide higher resilient modulus values at higher temperatures without adversely affecting the modulus values at low temperatures.
2. Mixtures made with styrelf and carbon black–modified binders are most resistant to low-temperature cracking than conventional asphalts.
3. Modifying an asphalt cement with polymer or carbon black reduces the permanent deformation of the paving mixtures at high temperatures and thus reduces the potential for rutting. However, carbon black is more effective in reducing the rutting.
4. Using styrelf significantly improves the fatigue life of the pavements.

REFERENCES

1. D. N. Little, J. W. Button, R. M. White, E. K. Ensley, Y. Kim, and S. J. Ahmed. *Investigation of Asphalt Additives*. Report FHWA-RD-87-001. FHWA, U.S. Department of Transportation, 1986.
2. V. P. Puzinauskas, and E. T. Harrigan. *Modification of Asphalt Cement and Paving Mixes with Styrene and Butadiene Elastomer—Styrelf System*. Asphalt Institute, College Park, Md., 1987.
3. F. S. Rostler, R. M. White, and E. M. Dannenberg. Carbon Black as a Reinforcing Agent for Asphalt. *Proc. Association of Asphalt Paving Technologists*, Vol. 46, 1977, pp. 376–401.
4. A. G. Alliotti. Carbon Black—Its Nature and Possible Effects on the Characteristics of Bituminous Road Binders. *Proc., Australian Road Research Board*, Vol. 1, Part 1, 1962.
5. Z. Yao and C. L. Monismith. Behavior of Asphalt Mixtures with Carbon Black Reinforcement. *Proc. Association of Asphalt Paving Technologists*, Vol. 55, 1986, pp. 564–585.
6. *Mix Design Methods for Asphalt Concrete*. Manual Series No. 2 (MS-2). Asphalt Institute, College Park, Md., 1984.
7. N. P. Khosla. *Investigation of Premature Distresses in Flexible Pavements*. Report FHWA-NC-85-001. FHWA, U.S. Department of Transportation, 1985.
8. F. Finn et al. The Use of Distress Prediction Subsystems for the Design of Pavement Structures. *Proc., 4th International Conference on the Structural Design of Asphalt Pavements*, Vol. 1, University of Michigan, Ann Arbor, 1977.
9. N. W. McLeod. A 4-Year Survey of Low Temperature Transverse Pavement Cracking on Three Ontario Test Roads. *Proc. Association of Asphalt Paving Technologists*, Vol. 41, 1972.
10. W. J. Kenis. *Predictive Design Procedures: VESYS Users Manual*. Report FHWA-RD-77-154. FHWA, U.S. Department of Transportation, 1978.