Field Performance of Modified Asphalt Binders Evaluated with Superpave Test Methods
I-80 Test Project

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In the early 1980s heavy-duty pavements in Pennsylvania showed evidence of excessive rutting. As a consequence, the Pennsylvania Department of Transportation adopted several changes in its materials specifications and mixture design procedures. In addition, a number of modified binders were evaluated in an experimental test road that was constructed in 1989 in Clearfield County on Interstate 80. The construction was a 175-mm thick asphalt concrete overlay over an existing portland cement concrete pavement. Although the construction predated Superpave, original samples of the asphalt binder and loose asphalt mix were retained and were characterized using Superpave test methods. Field performance evaluations were performed immediately after construction and in subsequent years, giving a record of rutting, cracking, raveling, and overall visual performance. Overall, the mixtures have performed well during their 9 years of service. However, differences in the performance of the mixtures with the different binders are evident. These differences are related to the properties of the binder and the properties of the mixture as measured with the Superpave mixture and binder tests.

In the early 1980s heavy-duty pavements in Pennsylvania showed evidence of excessive rutting. The Pennsylvania Department of Transportation (PennDOT) responded by empanelling a department and industry rutting task force to address the problem and make recommendations to alleviate the problem. The task force made a series of recommendations that included changes in both material specifications and design procedures. Their recommendations included the following:

- A coarser gradation for the ID-2 wearing course mixture,
- A heavy-duty mix design based on 75-blow Marshall design with minimum limits on compacted air voids,
- Steps to ensure that mixtures are not overcompacted during construction,
- An increase in the coarse aggregate crush count, and
- A requirement that the sand fraction contain at least 75 percent crushed material.

The task force further recommended that the department evaluate several types of binder modification to assess their effectiveness in reducing rutting. The modifiers that were recommended by the task force are presented in Table 1.

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SCOPE

This paper describes the design, materials evaluation, and performance of a test project that was built in 1989 on Interstate 80 in Clearfield County, Pennsylvania, to evaluate the relative performance of different modifiers. The project contained a control section with unmodified binder and sections modified with two plastomers, two elastomers, Gilsonite, and polyester fibers (Table 1). A number of other experimental sections were included in the test project, but they are outside the scope of this paper. Although the design and construction of the project predated the development of the Superpave system, original samples of asphalt binder and asphalt mixture were retained by PennDOT and were tested by the authors using the Superpave test methods. Performance surveys were taken at yearly intervals until 1994; the pavement was last surveyed in November 1998.

CONSTRUCTION OF PROJECT

The test project is located on Interstate 80 in Clearfield County, Pennsylvania, between mile markers 120 and 128. The entire project, an overlay of an existing conventional jointed portland cement concrete pavement, was nearly 29 km (18 mi) long. A layout of the test site is shown in Figure 1. The pavement was placed in three lifts: a 75-mm-thick base course followed by a 63.5-mm-thick binder course and a 38-mm-thick wearing course. The modified binders were used in the binder and wearing course only; the control AC-20 was used in the base course. The mixtures were designed with 75-blow Marshall compaction using the task force recommendations described previously. Properties of the mix are presented in Table 1, and the aggregate gradation of the wearing course and of the binder course that contained the modified binders is shown in Figure 2. Recall that the construction predated Superpave, and therefore the design properties in Table 1 do not include Superpave design criteria.

The polymer-modified binders, except the low-density polyethylene (LDPE), and the fiber-modified binders were preblended at a plant in Pennsauken, New Jersey, and shipped to the job site. All of the modified mixes were made with the same base binder, the AC-20 control binder. The LDPE was blended at the job site with a high shear mixer. The LDPE-modified mix was produced at 163°C; the remaining mixes were produced at 150°C. The Gilsonite and the polyester fibers were added to the aggregate during the premixing

TABLE 1 Properties of Materials and Mixtures Obtained at Construction

Test Section Modifier	A Poly-ethylene	B Ethylene vinyl acetate	C Polyester fibers	D SB-reacted	E Gilsonite	G SBS 4141 Blended	Control Sections AC-20
Modifier, %	5.0	4.0	6.3[a]	4.3[c]	10[b]	4.0	--
Binder Properties							
Softening Point, °C	56	64	--	64	57	67	49
Ductility, 25°C, cm	3.5	4.5	2	55	0	47	4
Abs. Vis., 60°C, kP	6.40	4.0	--	18.0	7.4	59.0	2.17
Marshall Properties							
Stability, kN	12.0	10.5	10.6	13.9	17.3	18.6	17.1
Flow, 0.1mm	11	11	12.5	11	11	14	11
Laboratory Cores							
M_r,[d] psi, 25°C	947	905	---	602	1700	500	614
S_T,[e] psi, 25°C	187	195	---	126	195	188	170
Air Voids, %							
Marshall Samples	4.9	4.1	--	5.3	5.2	4.9	4.7
Field Cores, 10/89	7.4	6.1	--	7.1	7.0	7.1	6.1

[a] Based on binder content, 0.280 percent of total weight of mix.
[b] Based on total asphalt content.
[c] Six percent was added to binder during blending but it was not totally dispersed in binder. Analysis of blend showed 4.3% after blending.
[d] Resilient modulus
[e] Indirect tensile strength

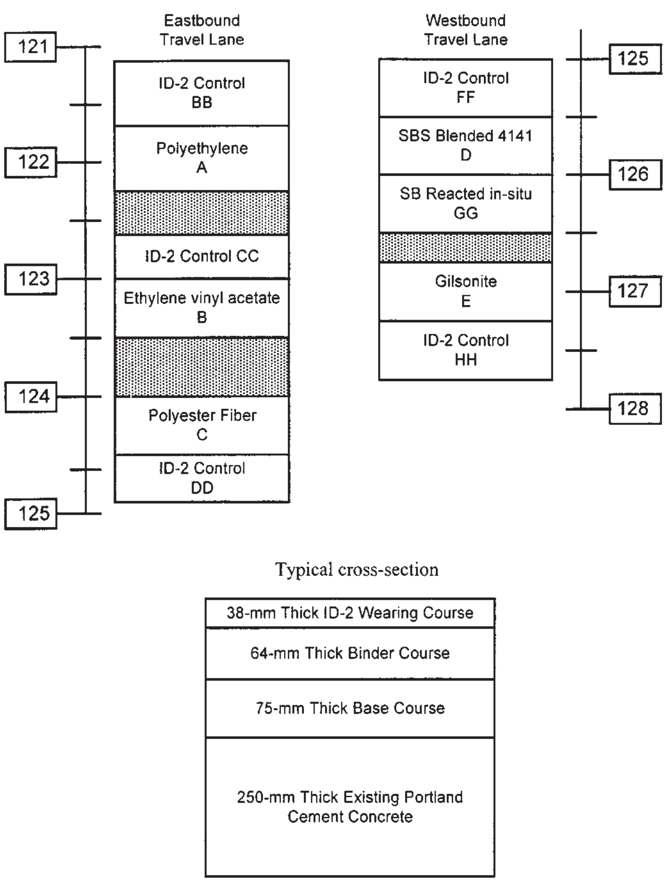

FIGURE 1 Plan view of test sections.

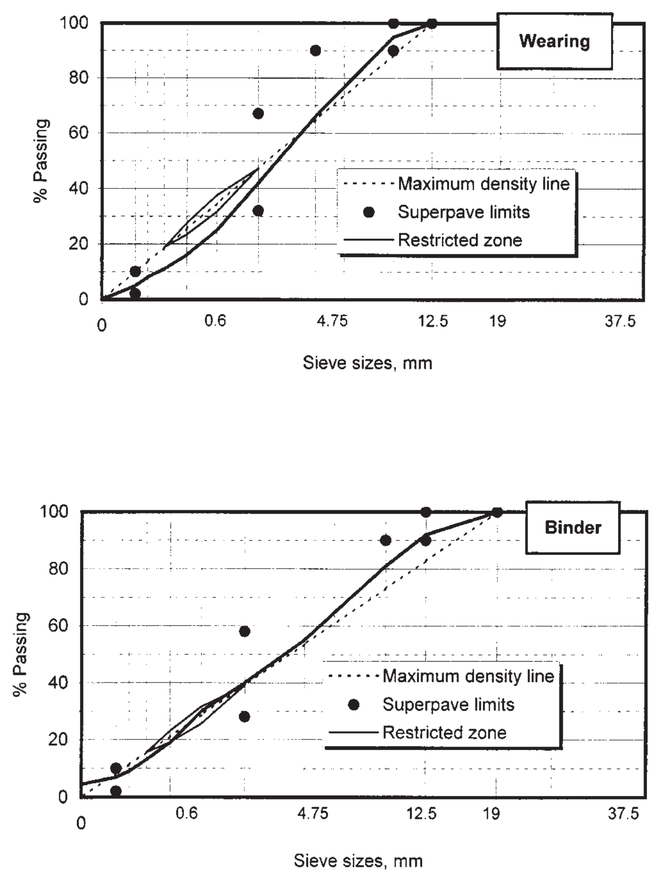

FIGURE 2 Gradation curves for ID-2HD wearing and binder course.

cycle at the batch plant. The dry-mix time for the fiber mix was increased by 10 to 15 s to allow for complete dispersion of the fibers. Dosage rates for the modifications are presented in Table 1. No problems were encountered during construction except that some clumping was observed in the mix modified with ethylene vinyl acetate (EVA). Mix temperatures were 135°C to 143°C, except for the LDPE-modified and fiber-modified mixes, which were placed at 141°C to 146°C. The in-place air voids are presented in Table 1, along with the air voids for cores removed from the pavement just after construction in October 1989. The construction variables, such as binder content, air voids, and gradation, were so well controlled in the project that it is highly unlikely that they caused the differences in the performance of the test sections. It should be noted that the Gilsonite and fiber mixes required additional mixing time, and the LDPE-modified mix required increased mixing and placement temperature. Average cost for modified asphalt mixes was $32.57 per ton versus $22.50 for the AC-20 mix.

CHARACTERIZATION OF ASPHALT BINDER

Conventional (pre-Superpave) properties of the binders are presented in Table 1. The viscosity values reported in Table 1 were obtained with capillary measurements and must be considered suspect given the non-Newtonian nature (phase angle <90 degrees) of the modified binder. Softening point temperatures and 25°C ductility values are also given in Table 1.

The Strategic Highway Research Program (SHRP) test procedures (AASHTO TP5-93, TP1-93) were applied to the binders as described subsequently. The polyester fibers were added to the AC-20 by blending in the laboratory. Given the problems that were encountered in making dynamic shear rheometer (DSR) and bending beam rheometer (BBR) test specimens, the dispersion and orientation of the fibers in the test specimen, and other theoretical considerations, the data for the fiber-modified binder were not considered valid and therefore are not reported in this paper. The binders were aged following ASTM D 2872 (AASHTO T240-94) and AASHTO PP1-93. The entire pressurized aging vessel (PAV) residue for a given binder was aged during a single run. The residue from the multiple pans was then combined into a single container, heated, and stirred to blend. The blended material was then poured into individual containers for further testing.

Testing Protocol

DSR

Frequency sweeps were conducted from 0.1 to 100 rad/s for the 8-mm plate and from 1 to 100 rad/s for the 25-mm plate. The unaged binder and the rolling thin-film oven test (RTFOT) residue were tested using at least two temperatures, whereas the PAV residue was tested using three temperatures for each, at 12°C intervals. The strain levels were chosen to minimize the amount of nonlinearity and discontinuities in the master curves. Therefore, a different strain was selected for each frequency decade, and the same strain was maintained throughout the decade (e.g., from 0.1 to 1.0, 1.0 to 10, and 10 to 100 rad/s).

BBR

The binders were tested at two or more test temperatures, and the data were reported at 8, 16, 30, 60, 120, and 240 s. Testing was conducted after 60 ± 5 min of conditioning at the test temperature. Separate beams were held for 24 h ± 30 min at the test temperature and tested for physical hardening at the test temperature closest to the specification temperature as determined from the 1-h testing. Aluminum molds were used for all of the testing.

Direct Tension Test (DT)

Tests were conducted for at least two temperatures that yielded failure strains from 1 to 10 percent. All tests were performed with an Instron 4552 Control closed-loop testing machine using silicone rubber molds. This machine is a vertical machine fitted with a nitrogen-controlled temperature chamber. The elongation measurements were obtained with a strain-gauge extensometer. Measurements were typically obtained at two test temperatures, sufficient to bracket 1 percent failure strain. All measurements were made with elongation rates of 0.1 mm/min.

Superpave Grading

Table 2 shows the limiting temperatures, the range between high- and low-temperature grading temperature, and the resulting perfor-

TABLE 2 Superpave Binder Grading Temperatures

Asphalt binder	G/sin δ [(a)] Tank	G/sin δ [(b)] RTFOT	G sin δ [(c)] PAV	S(60s) [(d)] PAV	m(60s) [(e)] PAV	1% [(f)] PAV	PG grade	Grading range
AC-20	<u>67.2</u>	68.1	19.7	-18.6	<u>-16.0</u>	-19.1	64-22	83.1
PE	<u>76.7</u>	78.1	23.2	-17.1	<u>-10.7</u>	-22.2	76-16	87.4
EVA	<u>77.2</u>	85.7	20.0	-17.6	<u>-11.2</u>	-21.7	76-16	88.4
SBS	<u>77.3</u>	79.7	16.4	-23.2	<u>-15.1</u>	-28.9	76-22	92.3
SB-Reacted	82.4	<u>80.9</u>	15.1	-22.6	<u>-17.9</u>	-30.0	76-28	98.8
Gilsonite	<u>80.3</u>	81.5	27.8	-14.1	<u>-9.3</u>	-14.9	76-16	89.6

Note 1: Underlined temperatures represent the controlling specification temperatures.

Note 2: (a) Temperature at which $|G^*|/\sin\delta = 1.00$ kPa, °C
(b) Temperature at which $|G^*|/\sin\delta = 2.20$ kPa, °C
(c) Temperature at which $|G^*| \sin\delta = 5.00$ MPa, °C
(d) Temperature at which S(60s) = 300 MPa, °C
(e) Temperature at which m(60s) = 0.300, °C
(f) Temperature at which failure strain = 1%, °C

mance grade (PG) obtained for the set of asphalt binders in the study. The results from the direct tension test were not used in grading the asphalt binders because grading criteria for using the DT were not established when the paper was written. The tabulated results indicate that the controlling high temperature is given by the unaged condition, except for the SB-reacted-modified binder, for which the control temperature is given by the RTFOT condition. The controlling low temperature was given exclusively by the *m*-value. The PGs varied from 64-22 for the base asphalt binder to 76-28 for the SB-reacted-modified binder. The highest low-temperature limit, −9.3°C, was found for Gilsonite-modified asphalt binder. This value was confirmed by the DT results. For grading range, the highest number was obtained by the SB-reacted-modified binder, 98.8°C, whereas the lowest corresponded to the base asphalt binder, 83.1°C.

Master Curve Characterization

The additional DSR data obtained for the set of asphalt binders in the PAV condition was used for master curve fitting. $|G^*|$ master curves for the PAV condition of the asphalt binders under investigation were obtained using the Christensen–Anderson model (*1*). The model provides simple expressions for $|G^*|$ and δ:

$$|G^*(\omega)| = G_g\left[1+(\omega_c/\omega)^{(\log 2)/R}\right]^{-R/(\log 2)} \tag{1}$$

$$\delta(\omega) = 90\Big/\left[1+(\omega_c/\omega)^{(\log 2)/R}\right] \tag{2}$$

where

$|G^*(\omega)|$ = absolute value of complex modulus at frequency ω,
G_g = glassy modulus,
ω_c = location parameter called the crossover frequency,
R = shape parameter called rheological index, and
$\delta(\omega)$ = phase angle at frequency ω.

All master curves were obtained at a reference temperature of 22°C. The horizontal shift factors for each individual temperature were considered unknown parameters in the nonlinear regression equation. Their estimated values, as well as the estimated values for the model parameters, were obtained at the end of each regression run. The conversion of BBR stiffness data to the absolute value of the dynamic complex modulus $|G^*|$ was done on the basis of the simple approximation $|G^*(\omega)| = S(t)/3$, where the test frequency ω in radians per second is equal to the reciprocal of loading time, *t*, in seconds. The complete list of the parameters obtained from fitting the model (Equation 1) to the test data is presented in Table 3. The glassy modulus was kept constant at $10^{9.1}$ Pa. The crossover frequency, ω_c, the frequency at which G' equals G'' and the phase angle is equal to 45 degrees, was reduced for all of the modified binders, reflecting a general increase in stiffness. The modification also increased the rheological index, *R*, reflecting a flattening of the master curve.

Physical Hardening

The BBR tests were conducted after two conditioning times, 1 h and 24 h. Physical hardening indices for both the stiffness at 60 s and the *m*-value at 60 s were calculated as the ratio of the value obtained after 24 h of conditioning divided by the value obtained after 1 h of conditioning. A summary of the physical hardening data and hardening indices is presented in Table 3. The base asphalt showed the smallest change in stiffness, an increase of 17 percent, and the largest change in the *m*-value, a decrease of 27 percent. Thus, on the basis of the data in Table 3, it can be concluded that in each case the modification increased physical hardening when stiffness was the criterion, but reduced physical hardening when the *m*-value was the criterion.

CHARACTERIZATION OF ASPHALT CONCRETE

Testing Protocol

Loose asphalt concrete mixtures containing Gilsonite, EVA, polyester fiber, and SB-reacted polymer modifiers were retained by PennDOT at the time of construction (mixes containing the other modifiers were not available at the time of this study). The retained loose mixtures were compacted in a Superpave gyratory compactor at 7 ± 1 percent air voids according to AASHTO TP 4-93. The compacted samples were sawed to produce test specimens 150 mm in diameter and 50 mm thick. The following tests were conducted on the test specimens:

TABLE 3 Master Curve Properties and Physical Hardening Aging Indices for Asphalt Binders

Asphalt Binder	Mastercurve Parameters		Test Temperature (°C)	BBR Test Results				Physical Hardening Indices	
				S(60s) (MPa)		m(60s)			
	ω_c [(a)]	R [(b)]		1h	24h	1h	24h	S(60s)	m(60s)
AC-20	0.9	2.4	-18	299	350	0.285	0.207	1.17	0.73
PE	-0.4	2.7	-18	333	426	0.245	0.208	1.28	0.85
EVA	-1.1	3.1	-18	325	411	0.261	0.225	1.27	0.86
SBS	-0.5	3.1	-24	322	466	0.231	0.201	1.45	0.87
SB-Reacted	0.2	3.0	-24	330	490	0.250	0.201	1.48	0.80
Gilsonite	-0.7	2.7	-18	451	602	0.230	0.192	1.34	0.83

[(a)] Logarithm of cross-over frequency, rad/s
[(b)] Rheologic index

- Indirect tensile creep test for 100 s at –20°C, –10°C, and 0°C, and indirect tensile strength test at –10°C according to AASHTO TP 9-94. Coefficients of thermal contraction were determined from 0°C to –25°C.
- Shear frequency sweep test at constant height at 4°C, 20°C, and 40°C according to AASHTO TP 7-94.

Indirect Tensile Creep and Strength Test

The indirect tensile creep test is conducted to evaluate the thermal-cracking behavior of asphalt concrete at low temperatures. A constant diametrical (creep) load is applied to cause a horizontal strain between 50 μm/m and 750 μm/m (*2*). Vertical and horizontal linear variable differential transformers (LVDTs) are attached to the center of the specimen and deformations are measured throughout the 100-s creep test. Creep compliance and Poisson's ratio were calculated from the test results. The coefficient of thermal contraction was also determined from 0°C to –25°C by observing changes in specimen dimension with the LVDTs. Strength tests were then conducted on the specimens at –10°C at 12.5 mm/min.

Shear Frequency Sweep Test at Constant Height Test

The shear frequency sweep test is a dynamic test in shear strain control (*2*). A constant shear strain with amplitude of 100 μ is applied during the test. The test was conducted at frequencies from 0.01 Hz to 10 Hz. During the test an axial load is applied to the test specimen to keep the specimen height constant. The shear complex modulus and phase angle are measured at various frequencies during the test.

Interpretation of Test Results

The indirect tensile test (IDT) data and a thermoviscoelastic analysis were used to calculate the critical pavement cracking temperature (*3,4*). The coefficients of thermal contraction for the asphalt concrete were from 3.24×10^{-5}/°C to 3.66×10^{-5}/°C. An average value of 3.52×10^{-5}/°C was assumed for thermal analysis. The tensile strength for Gilsonite- and fiber-modified asphalt concrete was 3.4 MPa (490 psi), and for EVA-modified and SB-reacted asphalt concrete it was 3.5 MPa (510 psi). The estimated critical pavement cracking temperature for Gilsonite- and EVA-modified asphalt concrete was –16.5°C, and for fiber-modified and SB-reacted asphalt concrete it was –19.5°C.

Comparisons of the critical temperatures for the asphalt binder and the critical cracking temperature calculated for the mixtures are presented in Table 4. The critical cracking temperature, calculated on the basis of asphalt concrete data, showed mixed results when compared with the binder properties. The mixture critical cracking temperature appears to agree more closely with the temperature at which the stiffness is 300 MPa or the *m*-value is 0.300 than with the temperature at which the DT failure strain is 1.00 percent.

Stiffness master curves for the available mixtures (IDT) and corresponding binders (BBR) were generated at a reference temperature of –18°C (Figure 3). Although the styrene-butadiene-styrene (SBS) gave the lowest moduli for both the binder and the mixture, the relative ranking of the EVA and Gilsonite reversed for the binder and the mixture—the Gilsonite was the stiffest in the binder. Dynamic shear master curves for the binder (DSR) and mixture (simple shear tester) are shown in Figure 4. The effect of the aggregate is clearly shown in the shape of the mixture master curve at frequencies above approximately 1 rad/s. The moduli rank in the same order for the binder and the mixture, with the Gilsonite being the stiffest, followed by the EVA and SB-reacted mixtures.

The complex shear modulus and phase angles for the mixtures at 12.6 Hz at temperatures of 4°C, 20°C, and 40°C are shown in Figure 5. At each temperature the ranking of the mixtures is the same. The Gilsonite modification produced the stiffest mixtures, followed by the EVA, polyester fiber, and SB-related modification. The increase in modulus was accompanied by a decrease in phase angle in the same order of ranking.

According to the rationale used in the binder specification, the fatigue life of asphalt concrete decreases as the dissipated energy per loading cycle increases. The dissipated energy decreases with an increase in complex shear modulus ($|G^*|$) and a decrease in the phase angle (δ) (*5*). Fatigue cracking occurs at intermediate temperatures from –10°C to 20°C. With this premise, the complex shear modulus and the phase angle results from shear frequency sweep test at 4°C and 20°C indicate that the fatigue life of both the EVA-modified and Gilsonite mixes would be less than that of the fiber and SB-reacted mixes. The same trend is indicated by the complex shear modulus master curves for both the asphalt concrete and the asphalt binder, as shown in Figure 4. These results are consistent with the field performance in which the observed fatigue cracking for EVA-modified and Gilsonite test sections was more severe than that of the fiber and SB-reacted test sections.

TABLE 4 Comparison of Critical Mixture Temperatures Using Binder Specification Criteria and Mixture Properties

Modifier	Binder Specification Criteria: Temperature where S = 300 MPa, °C	Binder Specification Criteria: Temperature where m = 0.300, °C	Binder Specification Criteria: Temperature where failure strain = 1%, °C	Critical Cracking Temperature from Thermoviscoelastic Analysis of Mixture, °C
Gilsonite	-14.1°C	-9.3°C	-14.9°C	-16.5°C
EVA-modified	-17.6°C	-11.2°C	-21.7°C	-16.5°C
SB-Reacted	-23.2°C	-15.1°C	-28.9°C	-19.5°C

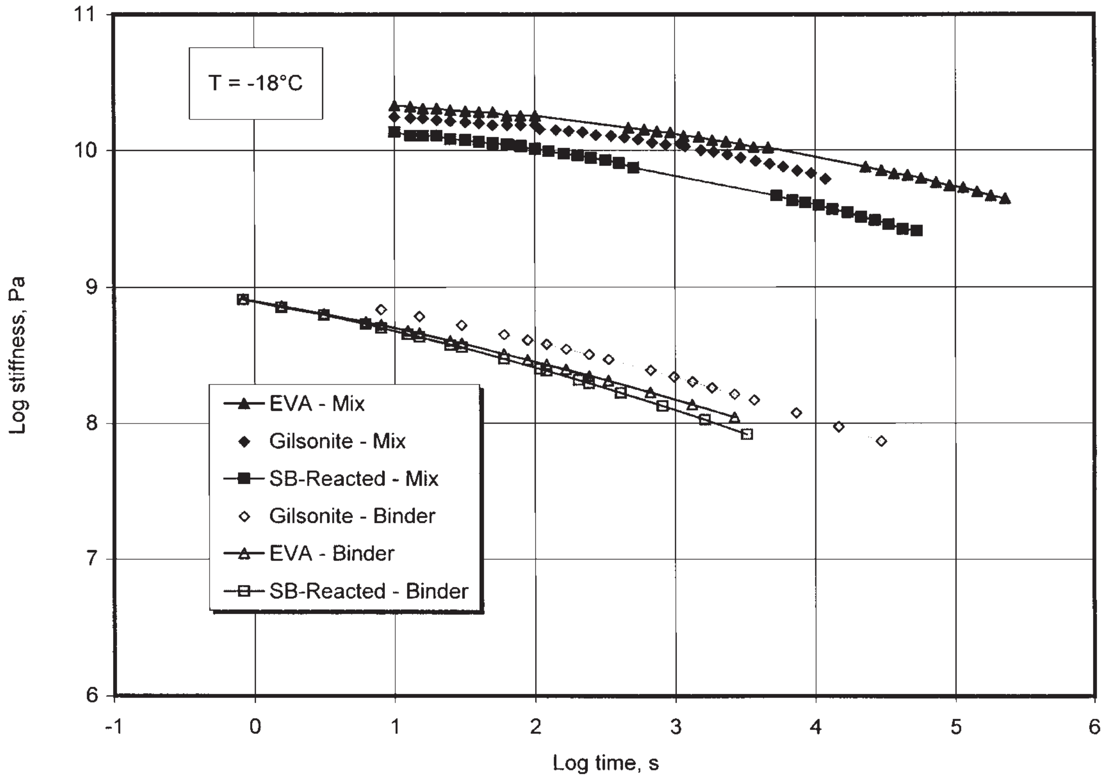

FIGURE 3 **Stiffness master curves for asphalt concrete and asphalt binder samples.**

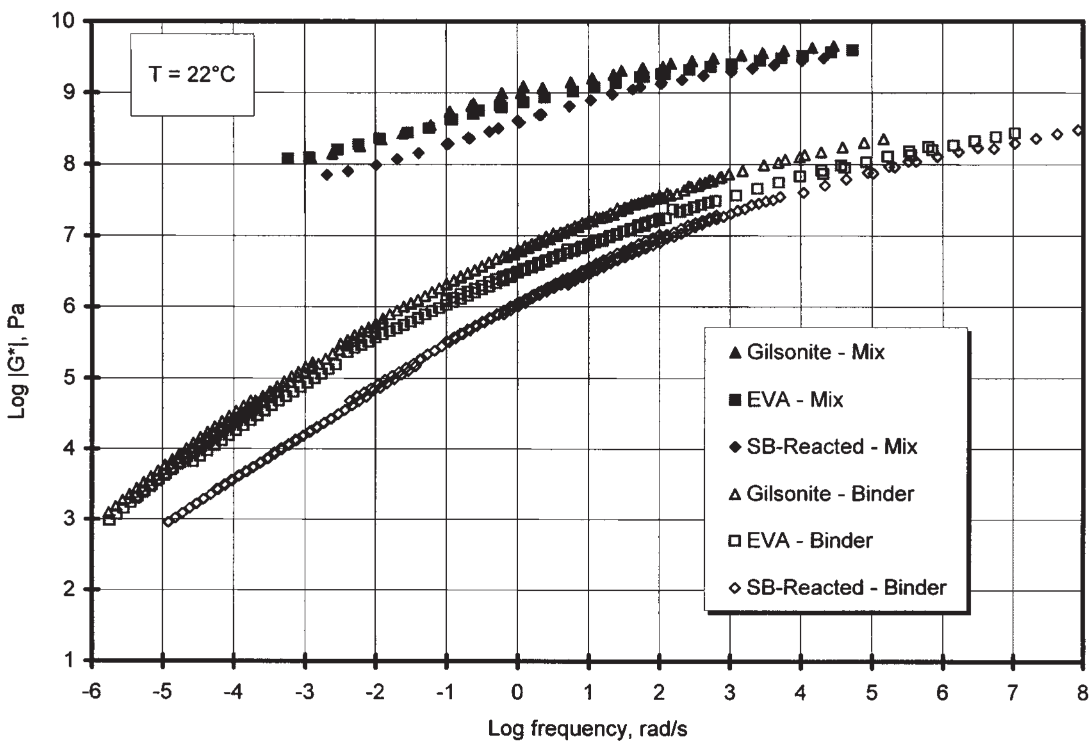

FIGURE 4 **Complex modulus master curves for asphalt concrete and asphalt binder samples.**

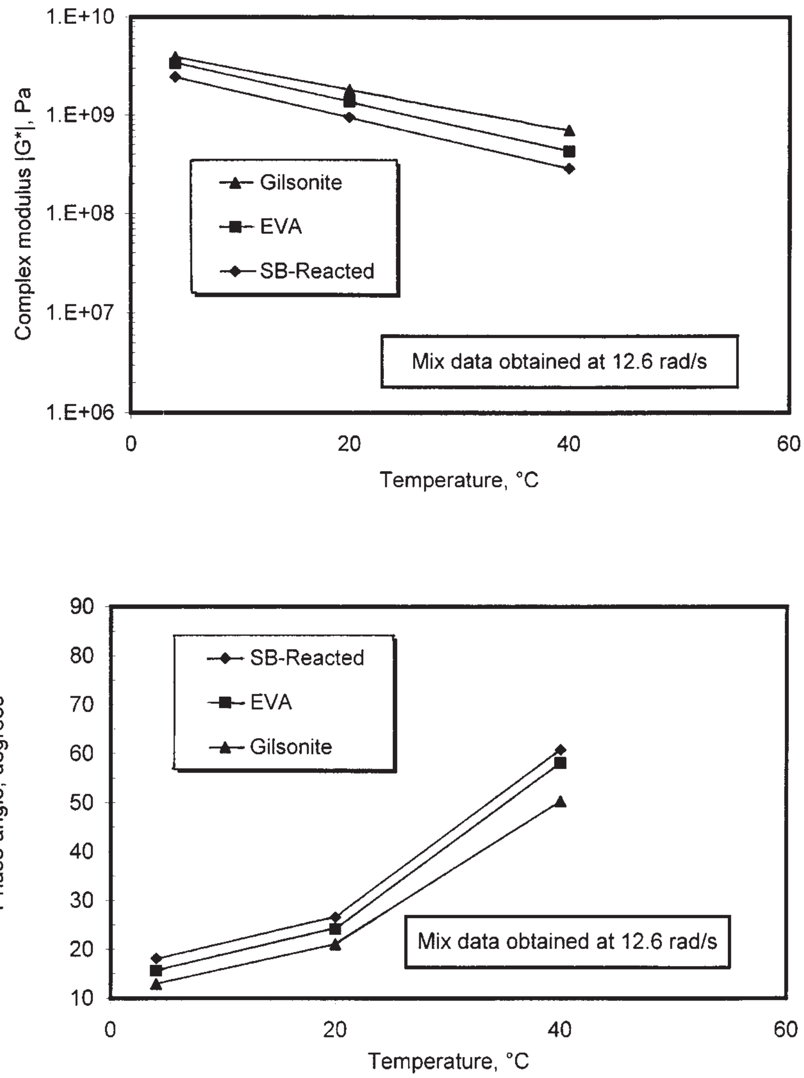

FIGURE 5 Complex modulus and phase angle for asphalt concrete samples.

PERFORMANCE

Overall, the performance of each test section (as of fall 1998), except the Gilsonite- and EVA-modified sections, has been good. Little rutting has occurred on any of the sections, including the control AC-20 section, although the rutting on the AC-20 section is somewhat larger than on the other sections (Table 5). The surface appearance of the AC-20 section has a more closed appearance in the wheel tracks, most likely reflecting the lower high-temperature stiffness of the AC-20 as compared with the modified materials.

To control reflection cracking, saw cuts were made in the overlay over the cracks (joints and structural cracks) in the underlying portland cement concrete pavement. The saw cuts were filled with sealant. The joints in the two elastomer sections are tight, with very little sign of secondary cracking. In contrast, limited secondary cracking was observed in the AC-20 control section, with considerable to severe secondary cracking in the two plastomer sections. A summary of the rutting and cracking performance is presented in Table 5.

Perhaps the most discriminating evidence is the visual performance of the mixes. During a 1997 walk-through inspection of the test sections, definite signs of raveling and what might be described as a brittle or friable appearance were observed on the Gilsonite section. This section showed the greatest aggregate loss of all the surfaces. This loss was especially evident at the reflection cracks, where the mix raveled badly. The department replaced the surface in the Gilsonite section in 1998 before the researchers had an opportunity to reinspect the section. Visual evaluation further discriminates between the fiber-modified mixes, the elastomer-modified mixes, and plastomer-modified mixes. The elastomer- and fiber-modified mixes are in excellent condition with no secondary cracking at the sawn joints and with little raveling. The plastomer-modified mixes give a visual performance rating between the elastomer-modified mixes and the Gilsonite.

A second and unexpected form of distress was observed in some of the test sections. What appeared to be longitudinal cracks became visible early in the Gilsonite section and later (1997) in the EVA and LDPE sections. An example of this cracking is shown in Figure 6, along with a core removed from a cracked section. From the core, it is obvious that the cracking is surface initiated and does not continue downward through the asphalt overlay. Further, the underlying pavement is structurally sound without any corresponding cracking. The longitudinal cracking was severe for the Gilsonite

TABLE 5 Performance Summary

Test Section Modifier	A Poly-ethylene	B Ethylene vinyl acetate	C Polyester fibers	D SB-reacted	E Gilsonite	G SBS 4141 Blended	Control Sections AC-20
Modifier, %	5.0%	4.0	6.3[(a)]	4.3[(c)]	10[(b)]	4.0	--
Rut Depth, mm							
Summer 1994	1.4	1.5	1.1	1.3	1.3	1.1	2.5
Fall 1998	2.3	3.5	2.4	2.5	overlaid	2.7	4.8

Comments from 1994 Survey:

All sections show some longitudinal edge and centerline cracking. Some coarse aggregate loss was observed in all sections with moderate loss (greatest) in EVA section. The EVA section appeared to have a more open texture. Fine aggregate loss was measured for all sections except non-existent on fiber and control sections. Rut depth was greatest for the control section but still within acceptable limits.

1997: Transverse and longitudinal cracking is evident in all sections but has not adversely affected serviceability. The Gilsonite section appears dry and brittle with excessive aggregate loss and potholing. This section will be replaced in next construction season. Cracking is now apparent at reflection joints in EVA and PE sections. SBS and SB-reactive sections show little cracking at the crack openings.

section (it was removed in 1998), moderate to severe for the EVA section, moderate for the LDPE section, and light to none for the elastomer and AC-20 sections. As of fall 1998, the EVA section was approaching the poor condition observed previously (fall 1996) in the Gilsonite section. Because of accelerated distress, the EVA section is likely to be replaced during the 1999 construction season.

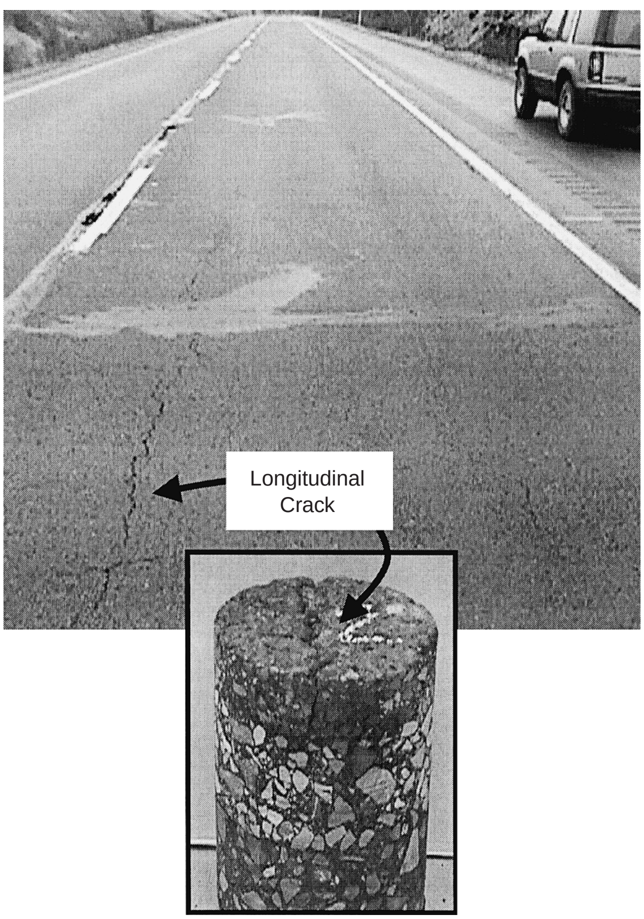

FIGURE 6 Longitudinal surface fatigue cracking.

This type of cracking was reported by Myers et al. in 1998 (*6*). Two cores were removed from the EVA test section in December 1998. One core was removed directly over a longitudinal crack in the inside wheel path, and the other was taken over a crack located between the wheel paths. Both cracks initiated at the surface and, as shown in Figure 6, have propagated only through the surface course (38-mm depth). Given the severity and unexpected occurrence of this cracking, continued investigation is planned to better identify the cause.

SUMMARY AND CONCLUSIONS

In summary, six asphalt modifiers were used in a test project constructed in 1989 on Interstate 80 in Pennsylvania. Improved aggregate specification and mix design criteria were used to produce the wearing coarse and binder mixtures for the project. The gradation and void criteria (Marshall 75-blow compaction) for the wearing course mixes met the current Superpave criteria for hot-mix asphalt construction. No significant construction problems were encountered during construction of the project. Because the project was an overlay over portland cement concrete, the primary forms of distress to date are raveling, secondary cracking at sawn joints, and longitudinal fatigue cracking (Gilsonite, EVA, and LDPE, in order of severity). Rutting to date is minimal, with less than 6 mm in all sections. The Gilsonite-modified test section has shown the earliest and most severe signs of distress and has been replaced (1998). The two sections constructed using mixtures containing plastomer-modified asphalt binders showed moderate signs of brittle distress, similar to that seen in the Gilsonite-modified section, but not as severe.

The SHRP binder specification test data and Superpave mixture performance tests show significantly greater modulus values for materials showing brittle distress compared with mixtures showing little or no raveling and surface cracking. Furthermore, the highest modulus values were seen for the Gilsonite-modified binder and mixture, followed by the plastomer-modified binders and mixtures. Thus, the intermediate modulus data also ranked the binder and mixtures in correct order of performance, with the worst-performing mixture showing the highest modulus values. These findings indicate that the observed raveling, surface cracking, and general deterioration of the poor-performing pavement are associated with stiffness and brittleness of these mixtures and the binders with which they have been made. The SHRP binder grade for the region in which the test road was constructed is 58-28, neglecting adjustments for traffic. The maximum temperature at which $G^*\sin\,\delta$ can equal 5.00 MPa for this climate is then 13°C. This temperature is significantly exceeded by all of the binders, except those modified with elastomeric polymers. The Gilsonite binder has a dynamic loss modulus of 5.00 MPa at 28°C, 13°C higher than the critical value for this climate. Therefore, the binder data in this case support the SHRP binder specification as well as the specification values. This preliminary conclusion merits additional study.

It should be pointed out that the fatigue-related distresses observed in this study were surface fatigue and secondary cracking at the joints. It is not the classic traffic-associated fatigue, which starts at the underside of bound pavement layers and works up toward the surface and for which the binder specification was developed. The fatigue-related distress observed in this project is possibly more complicated and may not be adequately addressed in the current SHRP binder specification, although the severity of the distress does appear to warrant an upper limit on the stiffness of the binder at intermediate temperatures.

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