Evaluation of Low-Temperature and Elastic Properties of Crumb Rubber– and SBS-Modified Bitumen and Mixtures

Baha Vural Kök[1]; Mehmet Yilmaz[2]; and Alaaddin Geçkil[3]

Abstract: In this study, the performances of bitumen and asphalt mixtures modified by crumb rubber (CR) were compared with those modified by styrene-butadiene-styrene (SBS). The resultant mixtures were evaluated for their rheological and mechanical performances by different experimental techniques such as rheological bitumen tests, i.e., dynamic shear rheometer (DSR), bending beam rheometer (BBR), and hot mixture performance tests, that is, indirect tensile stiffness modulus, fatigue, semicircular bending, and toughness index. Experimental studies show that it is necessary to use twice as much CR as SBS to reach the same performance attained by SBS. CR modification at high additive content exhibits higher elastic response, i.e., recoverable strain, than the SBS-modified mixture. While the resistance to crack initiation of CR-modified mixtures increases with increasing additive content, the resistance to crack propagation decreases dramatically according to fatigue and semicircular bending tests. **DOI: 10.1061/(ASCE)MT.1943-5533.0000590.** *© 2013 American Society of Civil Engineers.*

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Introduction

Highways produced with base asphalt cannot sustain low speeds and heavy loads due to the asphalt's drawbacks, such as low-temperature cracking, poor rutting, and low fatigue resistance. Previous studies showed that polymers improve rutting performance, adhesion, and cohesion of an asphalt binder (Kanitpong and Bahia 2005; Chen et al. 2002). Currently, the most commonly employed polymer used for bitumen modification is styrene-butadiene-styrene (SBS). However, it was detected that this modifier not only decreases the workability of hot-mix asphalt (HMA) but also fails to provide a cost-effective solution. The use of waste materials has thus become an important issue in this respect. Around the world, millions of tons of waste tires are generated and discarded each year. The tire stockpiles and landfills cause a number of problems to local communities, such as fire hazards and environmental concerns. The use of these scrap tires to modify asphalt is considered a significant issue because it contributes significantly to environmental conservation and facilitates economic sustainability by reducing the construction costs of modified roads (Yildirim 2007). For example, it was discovered that the addition of 9% by weight ground tire rubber to bitumen increased both the linear viscoelastic modulus and the viscosity at high in-service temperatures (Navarro et al. 2004). It was also reported that the addition of recycled tire rubber to asphalt mixtures using a dry process can improve the engineering properties of asphalt mixtures (Cao and Chen 2008).

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Crumb-rubber-modified (CR-modified) asphalt can be produced by either wet or dry processes, though the former is more popular. This is attributed to problems associated with the compatibility of mixtures. The addition of crumb rubber (CR) has proven helpful in increasing the voids in mineral aggregate in Superpave mix design and improving the rutting resistance of asphalt mixtures regardless of rubber size and type (Xiao et al. 2009). Chiu (2008) demonstrated, by a 4-year field evaluation, the satisfactory performance of asphalt-rubber-modified mixture and its potential to replace conventional dense-graded mixes. Studies show that the rubber content of asphalt-rubber mixtures has a significant effect on their performance with respect to the resistance to permanent deformation and cracking (Cao and Chen 2008). CR-modified asphalt was determined to have the best low-temperature anticracking performance at a CR content of between 5 and 20% according to bending beam rheometer (BBR) tests (Liu et al. 2009). Rubber-modified bitumen shows improved viscoelastic characteristics and, therefore, higher viscosity than unmodified binders, indicating an improved resistance to permanent deformation or rutting and low-temperature cracking. It was also reported that rubber-modified bitumen (9% by weight) shows very similar linear viscoelastic properties to SBS-modified bitumen having 3% by weight SBS at 10°C and 7% by weight SBS at 75°C (Navarro et al. 2002). Kok and Çolak (2011) showed that to achieve the same performance with SBS modification, the CR content must be higher than the SBS content. Rubber particles of multiple sizes were also believed to have a better sound-absorbing effect in spray applications (Zhu and Carlson 1999). Several researchers demonstrated the improved performance of bituminous mixtures with CR (Lalwani et al. 1982; McGennis 1995; Bahia and Davis 1994). Increased fatigue life, reduced reflective cracking and low-temperature cracking, and improved tensile strength were cited as the advantages of CR-modified mixtures (Oliver 2000).

In this study, the low-temperature performance and elastic properties of CR- and SBS-modified bitumen and bituminous mixtures produced by wet process were compared. In the first stage, the rheological properties of bitumen modified either with SBS or CR were examined by dynamic shear rheometer (DSR) and BBR tests. Next, the mechanical properties of SBS- or CR-modified hot bituminous mixtures were investigated by indirect tensile stiffness modulus, indirect tensile fatigue, toughness index (TI), and semicircular bending tests.

Materials and Methods

Materials and Sample Preparation

Asphalt cement, B 160–220, obtained from Turkish Petroleum Refineries was used as bitumen for mixture preparation. An SBS polymer, Kraton D-1101, used in the study was supplied by Shell Chemical Company. Five SBS-polymer-modified bitumen samples, (denoted as PMBs), were produced. The polymer contents of these PBMs ranged from 2 to 6% by weight, with 1% increments.

Similar to PBMs, five CR-modified bitumen samples were obtained whose rubber content ranged from 2 to 10% by weight, with 2% increments. Processing of scrap tires into CR can be accomplished through either ambient grinding or cryogenic grinding technologies. The ambient processes was found to be more effective at producing CR-modified binders that are more viscous and less susceptible to rutting and cracking. The use of rubber-particle sizes of less than 0.35 mm and high shear rates during manufacturing operations is highly recommended (Navarro et al. 2004). In this study, CR particles having sizes between 0.30 and 0.60 mm were obtained via ambient grinding processes.

Modified bitumens were produced with a laboratory-scale mixing device with a four-blade impeller (IKA) at a temperature of 180°C for 1 h at a rotation speed of 1,000 rpm.

Limestone aggregate was used in the asphalt concrete mixture. The properties of the aggregate are given in Table 1. A crushed coarse and fine aggregate, with a maximum size of 19 mm, was selected as the dense-graded asphalt mixture. The gradation of the aggregate mixtures is given in Table 2. The asphalt mixture was designed in accordance with the standard Marshall mix design procedure. The physical properties of the mixtures such as optimum bitumen content (OBC), bulk specific gravity (Gmb), maximum specific gravity (Gmm), air void (Va), voids filled with asphalt (VFA), and voids in mineral aggregates (VMA) are given in Table 3.

Table 1. Physical Properties of Aggregate

Property	Standard	Specific limit	Coarse	Fine	Filler
Abrasion loss (%) (Los Angeles)	ASTM C131 (ASTM 2006)	Max 30	29	—	—
Frost action (%) (with Na_2SO_4)	ASTM C88 (ASTM 2005)	Max 10	4.5	—	—
Flat and elongated particles (%)	ASTM D4791 (ASTM 2010a)	Max 10	4		
Water absorption (%)	ASTM C127 (ASTM 2012a)	Max 2	1.37		
Specific gravity (g/cm^3)	ASTM C127 (ASTM 2012a)		2.613	—	—
Specific gravity (g/cm^3)	ASTM C128 (ASTM 2012b)		—	2.622	—
Specific gravity (g/cm^3)	ASTM D854 (ASTM 2010b)		—	—	2.711

Table 2. Aggregate Gradation

Sieve size (mm)	Passing (%)
19	100
12.5	95
9.5	88
4.75	65
2.36	35
1.18	23
0.6	15
0.3	11
0.15	8
0.075	6

Dynamic Shear Rheometer and Bending Beam Rheometer Tests

The principal viscoelastic parameters obtained from DSR are the magnitude of the complex shear modulus (G^*) and that of phase angle (δ). G^* is defined as the ratio of maximum (shear) stress to maximum strain, and it provides a measure of the total resistance to deformation when the bitumen is subjected to shear loading (Airey et al. 2002). Permanent deformation is controlled by limiting $G^*/\sin\delta$ to values greater than 1.0 kPa (before aging) and 2.2 kPa [after rolling thin film oven (RTFO) aging]. Fatigue cracking is controlled by limiting the $G^* \sin\delta$ value of the pressure-aged (PAV) material to values less than 5000 kPa. The RTFO test is assumed to simulate short-term aging by heating a moving film of asphalt binder in an oven for 85 min at 163°C. The PAV is an oven-pressure vessel combination that takes RTFO-aged samples and exposes them to high air pressure (2070 kPa) and temperature. PAV is assumed to simulate the effects of long-term asphalt binder aging that occurs as a result of 5 to 10 years of HMA pavement service.

The DSR test was performed on base bitumen and SBS- and CR-modified binders using a Bohlin DSRII rheometer for unaged, RTFO-aged, and PAV-aged samples. The test was carried out under controlled-stress loading conditions using 1.59 Hz frequency at five different temperatures—52, 58, 64, 70, and 76°C—to determine the high-temperature performance grades and also to compare the elastic components of the SBS- and CR-modified binders by evaluating the phase angles.

Low-temperature cracking, commonly referred to as thermal cracking, is the most recognized non-load-associated distress (Mihai et al. 2004). Thermal cracking is caused by thermal-shrinkage-induced stresses resulting from environmental cooling

Table 3. Physical Properties of Mixtures

Mixture type	OBC (%)	Gb (g/cm^3)	Gmb	Gmm	Va (%)	VMA (%)	VFA (%)
Base	4.90	1.0282	2.348	2.447	4.07	14.78	71.90
2%CR	5.05	1.0310	2.346	2.444	4.01	14.89	73.06
4%CR	5.20	1.0339	2.344	2.439	3.93	15.10	73.99
6%CR	5.31	1.0359	2.337	2.437	4.07	15.42	73.58
8%CR	5.44	1.0377	2.337	2.433	3.96	15.55	74.51
10%CR	5.57	1.0390	2.332	2.429	4.01	15.82	74.64
2%SBS	5.02	1.0268	2.345	2.443	4.00	14.89	73.13
3%SBS	5.14	1.0259	2.339	2.439	4.11	15.24	72.93
4%SBS	5.26	1.0253	2.336	2.435	4.07	15.42	73.62
5%SBS	5.38	1.0232	2.331	2.431	4.10	15.71	73.82
6%SBS	5.50	1.0226	2.328	2.427	4.08	15.91	74.37

Note: OBC = optimum bitumen content; Gmb = bulk specific gravity; Gmm = maximum specific gravity; Va = air void; VMA = voids in mineral aggregates; VFA = voids filled with asphalt; Gb = the specific gravity of bitumen.

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(Zaniewski and Pumphrey 2004). The BBR is used to measure the stiffness of binders at very low temperatures. The test uses engineering beam theory to measure the stiffness of a small asphalt beam sample under a creep load used to simulate the stresses that gradually build up in a pavement when the temperature drops. *Creep stiffness* and *m-value* are the two parameters evaluated with BBR. The former measures how asphalt resists constant loading and the latter measures how the asphalt stiffness changes as loads are applied (Roberts et al. 1996). The creep stiffness of the asphalt beam sample at any time of loading (t) is determined by

$$S = PL^3/(4bh^3\delta_t) \quad (1)$$

where S = creep stiffness (MPa); P = applied constant load (N); L = span length of beam sample (102 mm); b = beam width (12.7 mm); h = beam thickness (6.35 mm); and δ_t = deflection (mm) at time t.

To prevent thermal cracking in Superpave, creep stiffness has a maximum limit of 300 MPa, and the m-value has a minimum limit of 0.300. For grading asphalt binders, the required performance characteristics are determined, and the temperatures at which these characteristics are satisfied establish the grade of the binder (Malpass 2003). It was reported that a decrease in creep stiffness leads to smaller tensile stresses in asphalt binders and reduced chances for low-temperature cracking (Asphalt Institute 2003). Liu et al. (2009) define the S/m-value ratio as coefficient λ; the smaller its value, the better the low-temperature performance. In this study, the test was performed at three different temperatures: −18, −24, and −30°C. Hence, the low-temperature performance grades of the binders were also determined. The binders are represented as performance grade (PG) X-Y in Superpave specification, where X is the highest pavement temperature rating, in degrees Celsius (°C), Y is the lowest pavement temperature rating, in minus degrees Celsius (°C), for which the PG binder was tested and expected to perform.

Indirect Tensile-Stiffness-Modulus Test

The stiffness-modulus test of asphalt mixtures measured in indirect-tensile mode is the most popular form of the stress—strain measurement methods used to evaluate the elastic properties of these mixtures. The indirect tensile-stiffness-modulus (ITSM) test defined by BS DD 213 is a nondestructive test. ITMS, which is considered a very important performance characteristic for pavement formulation, is defined as

$$S_m = P(R + 0.27)/tH \quad (2)$$

where S_m = stiffness modulus in MPa; P = peak value of applied vertical load (repeated load) (N); H = mean amplitude of horizontal deformation (mm); t = mean thickness (mm); and R = Poisson ratio (here assumed to be 0.35). The test was performed with deformation controlled using a universal testing machine (UTM). Target deformation was selected as 5 μm. During testing, the rise time, which is the time that passes for the applied load to increase from zero to a maximum value, was set at 124 ms. The load-pulse application was set to 3.0 s. The test was performed at 5°C.

Indirect Tensile-Fatigue Test

Fatigue is one of the most significant distress modes in pavements associated with repeated traffic loads (Ye et al. 2009). In this study, a constant-stress indirect tensile-fatigue test was conducted by applying a cyclic constant load of 500 kPa for 0.1 s followed by a rest period of 1.4 s. The measurements were carried out using a UTM at 5°C. The deformation of the specimen was monitored through linear variable-differential transducers clamped vertically onto the diametrical side of the specimen. A repeated dynamic compressive load was applied to specimens across the vertical cross section along the depth of the specimen using two loading strips 12.5 mm in width. The resulting total deformation parallel to the applied force was measured.

Toughness Index Test

The toughness index (TI) calculated from the indirect tensile test is a parameter describing the toughening characteristics in the post-peak region. The indirect tensile strength (ITS) test was used to determine tensile strength and strain of the sample. Cylindrical specimens were monotonically loaded to failure along the vertical diametric axis at a constant rate of 50.8 mm/min. Based upon the maximum load at failure, the ITS in kilopascals was calculated from the following equation:

$$\text{ITS} = 2P/\pi tD \quad (3)$$

where P = peak value of applied vertical load (kN); t = mean thickness of test specimen (m); and D = specimen diameter (m). A dimensionless indirect tensile TI, the TI is defined as follows:

$$TI = (A_e - A_p)/(\varepsilon - \varepsilon_p) \quad (4)$$

where A_e = area under normalized stress—strain curve up to strain ε; A_p = area under normalized stress—strain curve up to strain ε_p; ε = strain at point of interest; and ε_p = strain corresponding to peak stress. The TI compares the elastic performance of a specimen with that of a perfectly elastic reference material for which the TI remains constant at 1.0. On the other hand, for an ideal brittle material without postpeak load carrying capacity, the value of TI equals zero (Kabir 2008). In this study, the values of the indirect tensile TI were calculated up to tensile strains of 1, 2, and 3%. The test was performed at 25°C.

Semicircular Bending Test

This test method determines the tensile strength or fracture toughness of an asphalt mixture for the assessment of the potential for crack propagation. The crack propagation phase describes the second part of a failure mechanism during dynamic loading. The test was conducted according to EN 12697-44 at 0°C. The specimens were prepared with a gyratory compactor 150 mm in diameter and 50 mm thick. They were cut into two equal semicircular parts from the middle. A single notch 10 mm deep was cut in the middle of the specimens loaded by applying a constant cross-head deformation rate of 5.0 mm/min. The load and deformation were recorded continuously, and the fracture toughness (K_{Ic} N/mm$^{3/2}$) was determined using the following equations:

$$\sigma_{\max} = 4.263.F_{\max}/D.t \quad (5)$$

$$K_{Ic} = \sigma_{\max}.5.956 \quad (6)$$

where $\sigma_{\max}$ = maximum stress at failure; $F_{\max}$ is the maximum force in newtons; D = diameter of specimen in millimeters; t = thickness of specimen in millimeters. The fracture energy was also calculated according to the TC-50-FMC specification, which was used for asphalt mixtures by Li and Marasteanu (2010). The fracture energy (G_f) can be obtained by the following formula:

$$G_f = W_0/A_{\text{lig}} \quad (7)$$

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Table 4. DSR Test Results

Temperature (°C)	Base	2%SBS	3%SBS	4%SBS	5%SBS	6%SBS	2%CR	4%CR	6%CR	8%CR	10%CR
	$G^*/\sin\delta$ (Pa)										
52	2263	5168	7452	9551	15001	22133	3346	4486	6514	8788	11710
58	1182	2387	3690	4735	7551	11803	1597	2158	3254	4495	6182
64	523	1194	1788	2444	3896	6352	815	1084	1660	2359	3239
70	265	615	902	1271	2104	3276	423	559	876	1242	1717
76		333	484	687	1202	1801	228	304	471	659	923
	δ										
52	79.96	72.66	66.33	65.40	61.70	60.10	76.40	72.71	69.16	65.18	62.68
58	81.43	76.26	71.48	69.98	66.61	61.50	79.34	76.30	72.66	68.61	66.02
64	83.99	78.88	76.15	73.39	70.26	66.30	81.79	78.97	75.81	72.89	69.86
70	84.85	80.68	79.84	75.70	72.70	70.21	83.83	81.43	78.84	76.36	73.69
76		81.75	82.49	78.04	74.80	73.02	84.02	83.26	81.53	79.74	77.25
	$G^*.\sin\delta$ (Pa.10^6) PAV residue										
16	2.2050						1.7760				
19	1.6270	1.3463	1.2240				1.2874	1.2753	1.5260		
22		0.6998	1.1521	0.4947				0.9439	1.0964	0.8792	0.9635
25				0.3590	0.5840					0.6504	0.7265
28					0.3500	0.6310					
31						0.5010					
	$G^*.\sin\delta$ (Pa) RTFOT residue										
58	7277						9552				
64		10960	11938					6849	10560		
70				7602						7701	9294
76					6340	8311					

Note: SBS = styrene-butadiene-styrene; CR = crumb rubber; PAV = pressure-aged value.

where W_0 = fracture work, the area below the measured load-deformation curve; A_{lig} = area of ligament, which is the product of the notch depth and the thickness of the specimen.

Results and Discussion

DSR and BBR Test Results

The complex modulus G^* and phase angle δ of the base and modified binders were measured at different temperatures. The $G^*/\sin\delta$ of neat and RTFO-aged binders and $G^*.\sin\delta$ of PAV-aged binders were also determined at the desired temperature to specify the high-temperature performance grades. DSR test results are given in Table 4. The base and 2% CR-modified binders were graded as PG 58. A grade of PG 64 was assigned to 2 and 3% SBS-modified and 4 and 6% CR-modified binders. A grade of PG 70 was assigned to 4% SBS-modified and 8 and 10% CR-modified binders. A grade of PG 76 was assigned to 5 and 6% SBS-modified binders. The rutting parameter ($G^*/\sin\delta$) values of SBS modification were higher for all temperatures than those of CR modification (Fig. 1). This can result from either a higher complex modulus or lower phase angles or the combined effects of factors. Obviously, high CR content is required to reach the same rutting parameter with SBS modification. The variation of phase angles with $G^*/\sin\delta$ is plotted in Fig. 2 to evaluate the elastic behavior of binders at the same rutting parameter level. It is apparent that modified binders are more flexible than base bitumen. There was almost no difference between the modified binders' curve at low $G^*/\sin\delta$ values. However, the difference began to increase at high $G^*/\sin\delta$ values. When $G^*/\sin\delta$ values of the CR-modified binders and unmodified binders equal to each other CR modification gives low phase angles, indicating an elastic response. The $G^*/\sin\delta$ indexes were determined by dividing the value of $G^*/\sin\delta$ obtained at 52°C to that obtained at 76°C (Fig. 3). The SBS-modified binders had a higher slope than CR-modified binders, indicating less temperature susceptibility for high temperatures. Although CR modification can provide the same

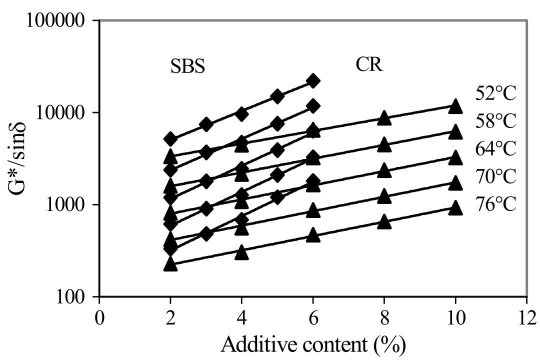

Fig. 1. Variation in $G^*/\sin\delta$ versus additive content

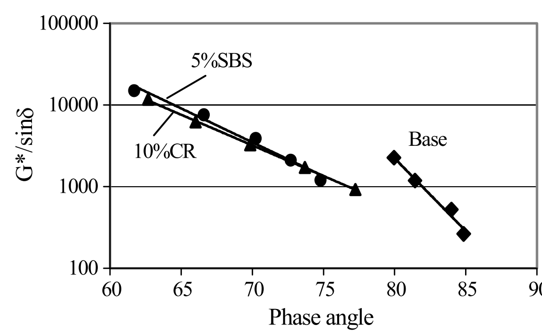

Fig. 2. Relation between $G^*/\sin\delta$ and phase angles

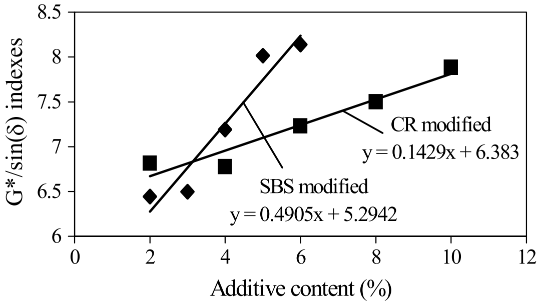

Fig. 3. Relation between $G^*/\sin\delta$ indexes and additive content

performance as SBS modification at low SBS contents, CR demand increases significantly to provide the same performance as SBS modification at high SBS contents.

From the BBR tests at different temperatures, shrinkage resistance parameters, stiffnesses, and *m*-values of the control, SBS- and CR-modified binders were calculated, and the results are shown in Table 5. The *m*-values of all binders decreased and creep stiffness values increased with decreases in temperature. There was no arrangement within the *m*-values of binders at different additive contents. Furthermore, the variation of the creep stiffness values of the SBS-modified binders was not regular with increases in additive content. However, the creep stiffness values of CR-modified binders, except for 2% CR, decreased regularly with increases in CR content and increased regularly with decreases in temperature. These results indicate that a more homogeneous blend occurs at different CR contents than at different SBS contents. The minimum creep stiffness value of the CR-modified binder belongs to 10% modification and is 30% lower than that of the base bitumen. The minimum creep stiffness value of the SBS-modified binder resulted from 6% modification and is 6% greater than that of the base bitumen. As seen from Table 5, the creep stiffness values of all binders were no greater than 300 MPa, even at the highest additive content and at the lowest temperature. Hence, the *m*-values determined the low-temperature performance grades. It was determined that all binders satisfied the requirements at −24°C (PG-34) except for 6% SBS-modified binder, which satisfied the requirements at −18°C (PG-28). Flexible materials exhibit high deflection at low temperatures. In this respect, it is obvious that the deflection values of CR-modified binders increased steadily with additive content at all temperatures. Moreover, 10% CR-modified binder had higher deflection values than the base binder. This also indicates a potential to dissipate the energy induced by loading. Irregular deflection values were attained for the SBS-modified binders. It was seen that 10% CR modified binder had the largest deflections among the modified binders, and only this binder had higher deflections than base bitumen at all temperatures. With the DSR test results, the performance grades of the base and modified binders were determined (Table 5). The performance grade as PG 64-34 is ensured by 2 and 3% SBS modifications and by 4 and 6% CR modifications. The same grade as PG 70-34 is ensured by 4% SBS modification and by 8 and 10% CR modifications. The variations on creep stiffness of the same graded binders are given in Figs. 4 and 5. Expectedly, there is almost no change in creep stiffness values at −24°C for the binders (Fig. 4) since they have the same low-temperature performance grade. However, CR-modified binders seem more flexible than SBS-modified binders by exhibiting lower stiffness values at −18°C and −30°C. As PG 64-34 binders are likely to be subjected to temperatures below −34°C frequently, CR-modified binders having low stiffness values guarantee a longer service life of pavement. SBS-modified binders give higher stiffness at all temperatures (Figs. 4 and 5). Therefore, it is economically advisable to prefer 6% CR to 3% SBS for PG 64-34 and 10% CR to 4% SBS for PG 70-34.

An increase in creep stiffness causes an increase in thermal stresses, leading to thermal cracking. While *m*-values decrease, the rate of stress relaxation also decreases and the ability to relieve thermal stresses by flow decreases in HMA pavement (Roberts et al. 1996). Since lower stiffness values and higher *m*-values indicate a good low-temperature anticracking property, the S/m-value ratio was also evaluated (Figs. 6 and 7). It is seen from the figures that the effects of temperature on S/m-values of the SBS- and CR-modified binders are different. While S/m-values of SBS-modified binders remain constant, those of CR-modified binders decrease with increases in additive content at −18°C. The S/m-values values started to increase smoothly with increasing additive content for SBS-modified binders and increased up to 4% CR content and began to decrease at −24°C. A similar trend was observed at −30°C for all binders. The lowest S/m values were obtained by 5% SBS-modified and 10% CR-modified binders at −30°C which is severe condition for roads. Any of the binders can reach a flexibility of base bitumen at −24 and −30°C. However, at 4, 6, 8, and 10% CR modifications more flexible behavior was evident than in B 160/220 base bitumen at −18°C.

Table 5. BBR Test Results

Temperature (°C)	Base	2%SBS	3%SBS	4%SBS	5%SBS	6%SBS	2%CR	4%CR	6%CR	8%CR	10%CR
	m-values										
−18	0.362	0.355	0.372	0.330	0.338	0.345	0.383	0.359	0.370	0.352	0.358
−24	0.338	0.328	0.313	0.313	0.303	0.274	0.325	0.324	0.316	0.310	0.307
−30	0.270	0.239	0.260	0.216	0.242	0.245	0.257	0.255	0.265	0.269	0.248
	Creep stiffness (Mpa)										
−18	89.31	97.22	99.41	91.92	100.72	94.74	97.54	77.39	68.56	63.27	62.06
−24	112.45	144.25	149.85	138.97	142.67	158.84	123.94	151.59	140.23	134.90	112.91
−30	129.31	224.55	267.15	207.53	202.55	207.25	134.22	159.71	154.87	148.80	122.45
	Deflection (mm)										
−18	0.893	0.825	0.801	0.880	0.796	0.843	0.820	1.034	1.161	1.266	1.292
−24	0.709	0.541	0.533	0.583	0.570	0.514	0.643	0.527	0.594	0.591	0.710
−30	0.620	0.362	0.299	0.387	0.398	0.390	0.594	0.493	0.515	0.536	0.651
	PG										
	58-34	64-34	64-34	70-34	76-34	76-28	58-34	64-34	64-34	70-34	70-34

Note: SBS = styrene-butadiene-styrene; CR = crumb rubber.

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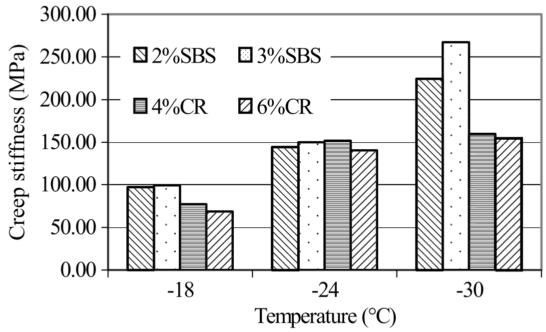

Fig. 4. Creep stiffness values of PG 64-34 binders versus temperature

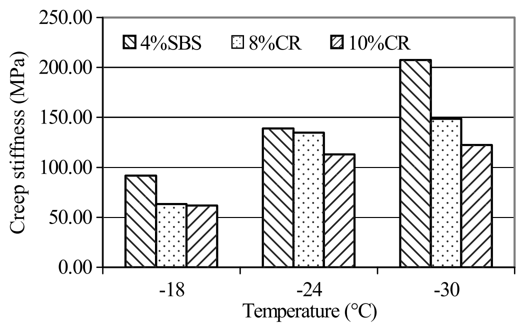

Fig. 5. Creep stiffness values of PG 70-34 binders versus temperature

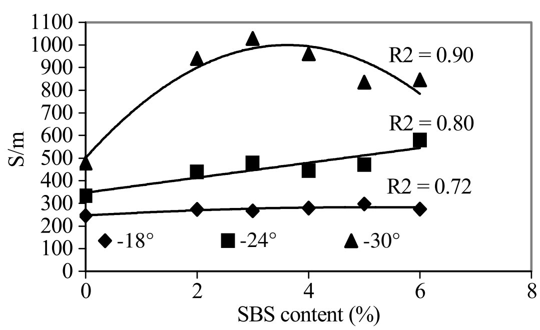

Fig. 6. Variation in S/m-values versus SBS content

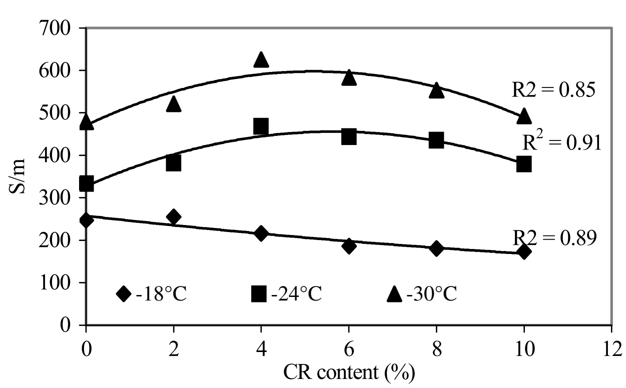

Fig. 7. Variation in S/m-values versus CR content

Indirect Tensile-Stiffness-Modulus Test Results

Three specimens were tested for each of the control and modified mixtures. To obtain a stiffness modulus value for a mixture, each specimen was tested at three different positions and the mean of nine values was used.

The stiffness modulus values of all mixtures are shown in Fig. 8. As seen from the figure, CR-modified mixtures have stiffness moduli between 6736 and 7586 MPa, depending on the amount of the CR in the mixture. On the other hand, for SBS-modified mixtures, stiffness values are greater than 7700 MPa. The figure shows that up to a certain additive content, i.e., 5% for SBS and 6% for CR, the modulus value increases with increasing additive content. Higher additive content leads to a significant decrease in the stiffness modulus. The test was performed in deformation-controlled conditions and at low temperature; thus, it is obvious that mixtures with lower stiffness values have greater flexibility.

The stiffness values of the SBS- and CR-modified mixtures show differences compared to those of the control mixture. The stiffness modulus of all SBS-modified mixtures studied are higher than that of the control mixture, causing pavement to exhibit lower strain at low temperatures. In contrast, the highest CR content, i.e., 10%, exhibits lower stiffness than that of the control mixture.

The elasticity of the mixtures was assessed by comparing the recoverable strains at a time just after load release, such as at 300 ms. The time-strain relations for the control, 5 and 6% SBS-modified, and 10% CR-modified mixtures are given in Fig. 9. The figure shows that the mixtures modified by 10% CR have the highest recoverable strain, i.e., elastic response. A lower elasticity was obtained for 5 and 6% SBS-modified mixtures as compared with the control mixtures.

Another comparison can be made by assessing the force-strain relation of the mixtures (Fig. 10). The strain values, which are fixed

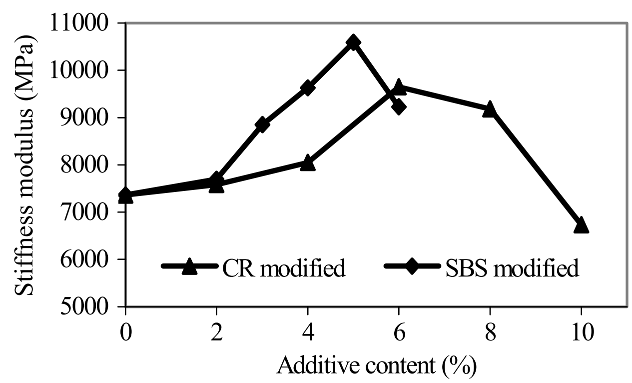

Fig. 8. Variation in stiffness modulus of mixtures with additive content

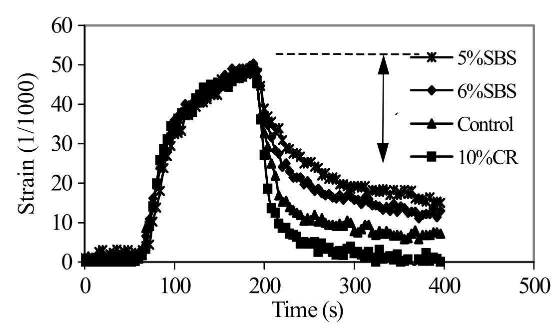

Fig. 9. Time-strain relation of mixtures

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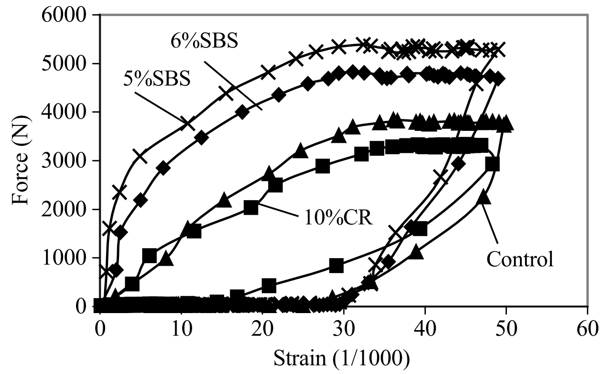

Fig. 10. Force–strain relation of mixtures

at 5 μm, return to zero after the peak force. However, the total area under each curve showing the ability of the sample to absorb the elastic energy, is different. Of all systems studied, 10% CR-modified mixtures had a minimum total area, indicating an ability to dissipate energy. Meanwhile, Fig. 10 shows that, to reach the target strain, a 10% CR-modified mixture requires approximately 50% less force than does a 5% modified mixture.

Indirect Tensile-Fatigue Test Results

Based on the results of the fatigue tests performed, the number of load repetitions yielding failure was determined for base, SBS-modified, and CR-modified mixtures. Figs. 11 and 12 show

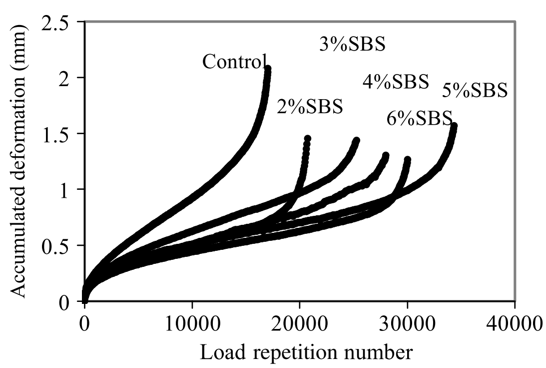

Fig. 11. Accumulated deformation-load repetition relation of SBS mixtures

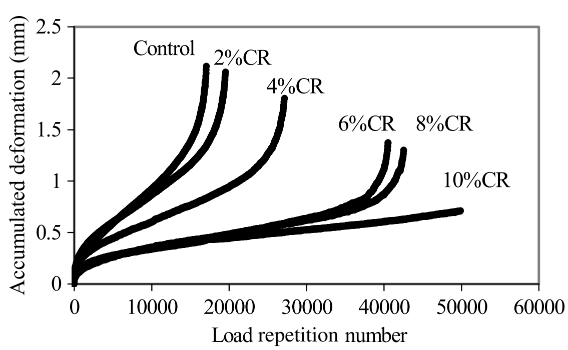

Fig. 12. Accumulated deformation-load repetition relation of CR mixtures

examples of the changes in accumulated deformation versus the number of load repetitions. As seen from the figures, there are three stages in these curves where initially the accumulated deformations increase rapidly since the air voids are pinched in with the load repetition. In the second stage, there is a linear increase in the amount of the accumulated deformations of the mixtures. Since the test was performed in stress-controlled conditions, after the first crack appears, the final stage, that is, the tertiary stage, initiates and the deformations increase rapidly.

As seen from the figures, a 5% SBS-modified mixture gave the highest load cycle number among the SBS-modified mixtures. Fig. 12 shows that the load repetition number increased steadily with increasing CR content. Similar performances were exhibited by 6 and 8% CR-modified mixtures. A superior performance was demonstrated by a 10% CR-modified mixture such that it did not pass to the tertiary stage even at a 50,000-load cycle. Compared with the control mixture, 8 and 10% CR-modified mixtures had, respectively, 2.7 and more than 3 times higher load cycle numbers. It is obvious that CR-modified mixtures at higher additive content could resist repeated traffic loads with no low-temperature cracking for longer periods than SBS-modified mixtures.

The fatigue performance of SBS added mixtures are worth examining. The collected data point out that the low- and high-temperature performances of these systems might be different from each other. For example, 6% SBS modification did not perform as well at low temperatures as it did at high temperatures. BBR tests also confirm this situation, indicating that 6% SBS-modified mixtures have a low-temperature-performance grade that is one level (6°C) lower than that of the other SBS-modified mixtures. Interestingly, this stiffness increase given by BBR tests was not observed in stiffness modulus tests for 6% SBS modification. Therefore, the decrease in the load repetition number for 6% SBS modification compared to 5% SBS modification is considered to be induced by a nonuniform distribution of higher SBS content or insoluble particles. Hence it can be said that SBS modification greater than 5% is not suitable for low-temperature conditions. It should also be noted that 5% SBS-modified mixtures have twice the cycle number of control mixtures.

Toughness Index Test Results

The variations in the TI value with additive content for different strain levels are presented in Fig. 13. It can be seen that CR- and SBS-modified mixtures demonstrate different performances. The figure shows that the TI increased with 2% SBS addition and then began to decrease with increasing SBS content. On the other hand, in the case of CR addition, the TI increased up to 6% CR content and then began to decrease under further CR modification. It was

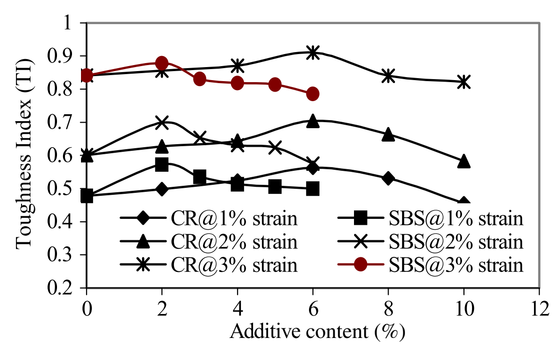

Fig. 13. Variation in toughness index versus additive content

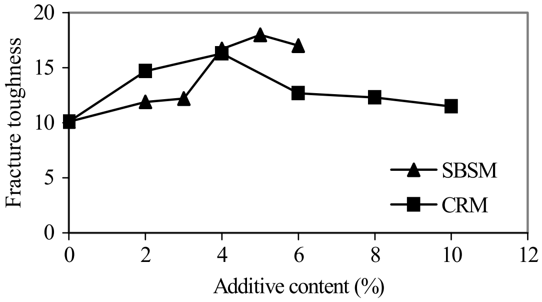

Fig. 14. Variation in fracture toughness versus additive content

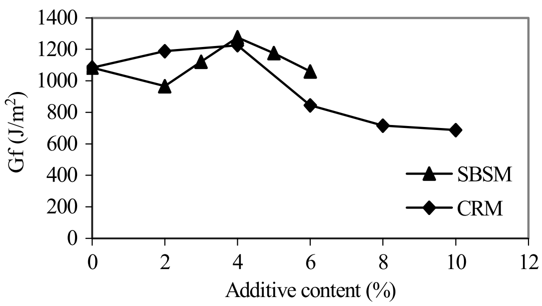

Fig. 15. Variation in calculated G_f versus additive content

determined that 2% SBS- and 6% CR-modified mixtures exhibited higher elasticity than the other mixtures of different modification contents. According to these results, the mixtures at high content of modification are expected to have inferior fatigue behavior. However, the fatigue life was determined to increase with increasing additive content. This can be attributed to either the differences in loading type between the test methods (one is deformation controlled, the other stress controlled) or the differences between the testing temperatures of samples. Another implication can be deduced from the experimental study that the TI and fatigue test cannot be associated to each other with the applied procedure in this study. However, the TI test provides a comparison for the elasticity or brittleness properties of the mixtures. It is obvious that CR modification has higher TI values than SBS modification, especially at high additive contents, indicating an improved elastic response. This also shows that CR-modified mixtures have a better ability to bridge cracks that may develop within a mix during loading. It was also determined that TI values increase with decreasing strain level, indicating different performances for the same contents of modifications. While 4% SBS and CR modifications have similar TI values at 2 and 3% strain levels, a difference is observed at a 1% strain level. Therefore, it can be concluded that TI values must be investigated at different strain levels to make a proper comparison among the mixtures.

Semicircular Bending Test Results

Fig. 14 shows the fracture toughness of the mixtures. It is seen that K_{1c} values increase up to 5% SBS and 4% CR additions. SBS modification gives higher K_{1c} values than CR modification when the amount of additive is higher than 4%. The variation in calculated G_f values is given in Fig. 15, which is compared to TI results. The brittle effects of high additive contents are more pronounced for CR modification. All these results indicate that a CR modification greater than 4% exhibits lower resistance to crack propagation than SBS modification of the same amount.

Conclusion

Based on the experimental work conducted in this study, the following conclusions can be drawn:

According to DSR test results, significantly higher CR content is required to reach the same rutting parameter with SBS modification. For the same and higher $G^*/\sin\delta$ values, CR modification gives low phase angles, indicating an elastic response.

The rheological tests demonstrated that 2 and 3% SBS modifications and 4 and 6% CR modifications ensure the same grade as PG 64-34. Similarly, 4% SBS modification and 8 and 10% CR modifications ensure the same grade as PG 70-34.

The stiffness modulus of the SBS-modified mixtures have higher values than the control mixtures, causing pavement to exhibit lower strain at low temperatures. It was determined that 10% CR-modified mixtures had the highest recoverable strain after the removal of the load and had a minimum area under the force-strain curve, indicating that the mixture had an ability to dissipate energy.

According to indirect tensile fatigue tests, 5% SBS-modified and 10% CR-modified mixtures exhibited the best fatigue performance. 10% CR modification exhibited twice performance than 5% SBS modification.

CR modification gave higher TI values than SBS modification, especially at high additive content, indicating an ability to bridge cracks that might have developed within the mixture during loading. Higher elasticity was exhibited by 2% SBS and 6% CR modifications compared with the other levels of modification. It was also determined that the TI and fatigue test could not be associated to each other due to the different loading types applied.

While CR modification has higher resistance to crack initiation, which is the first part of the failure mechanism, it cannot resist propagation of cracks after 4% additive content according to calculated fracture energy values.

Overall, the test results made it clear that CR modification is more suitable for combating low-temperature cracking in colder climates. On the other hand, as the TI and semicircular bending tests demonstrate, it is obvious that more than 4% CR-modified mixtures must be reinforced immediately after crack initiation for economical reconstruction. In this respect, the use of CR modification is preferred over SBS modification because it can provide a significant cost savings due to the high price of SBS and will also prevent the accumulation of this waste material in the environment.

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