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Fracture Properties of Aged Asphalt Mixtures Incorporating Recycled Tire Rubber and Styrene-Butadiene Rubber at Low Temperatures

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Abstract: A laboratory investigation was conducted to evaluate the fracture properties of aged asphalt mixtures containing recycled ground tire rubber (GTR) and styrene-butadiene rubber(SBR) modifiers at low temperatures. Asphalt mixtures were compacted by the gyratory testing machine and aged according to the SHRP short term oven aging (STOA) and long term oven aging (LTOA) procedure. An experimental program was to evaluate the effects of different modifiers and aging conditions on the indirect tensile strength, creep compliance, resilient modulus, and fracture energy of compacted asphalt mixtures.

The results indicate that the fracture energy of asphalt mixtures at -10℃ decreases slightly after the STOA or LTOA process in general, but the SBR-modified asphalt mixtures tend to increase at ultimate compaction condition.The GTR-modified asphalt mixtures have lower elastic stiffness (higher creep compliance intercepts) as compared with the unmodified mixtures before and after the LTOA process.The SBR-modified asphalt mixtures have higher fracture energy at -10℃ than that of mixtures containing the unmodified AC30 and the GTR-modified asphalt. Based on indirect tensile creep compliance, tensile strength, failure strain and fracture energy, the GTR and SBR-modified mixtures show a greater potential to resist low temperature cracking as compared with the unmodified mixtures.

Keywords: Fracture resistance; recycled tire rubber; ground tire rubber (GTR); styrene-butadiene rubber (SBR); fracture energy; asphalt mixture.

1. Introduction

The effect of temperature on the rheological response of asphalt binders is perhaps better considered in terms of temperature dependency, which, as illustrated by the isochronal curves shown in Figure 1, varies with both loading time and temperature [1]. Since these two parameters, temperature and time dependency are not simple linear functions, the temperature susceptibility of asphalt will depend on the testing temperature of the material. Moreover, temperature susceptibility is not a parameter which is based on a single variable, but it depends on temperature range, time of measurement, and the physical property that is measured. Therefore, using parameters which are based on measurements at specific conditions, such as PI, PVN, or VTS as indicators of temperature susceptibility can not characterize fully the rheological behavior of asphalt cements [1-2]. However, the estimates provided by Figure 1 are in considerable error at lower temperatures and longer loading times [2].

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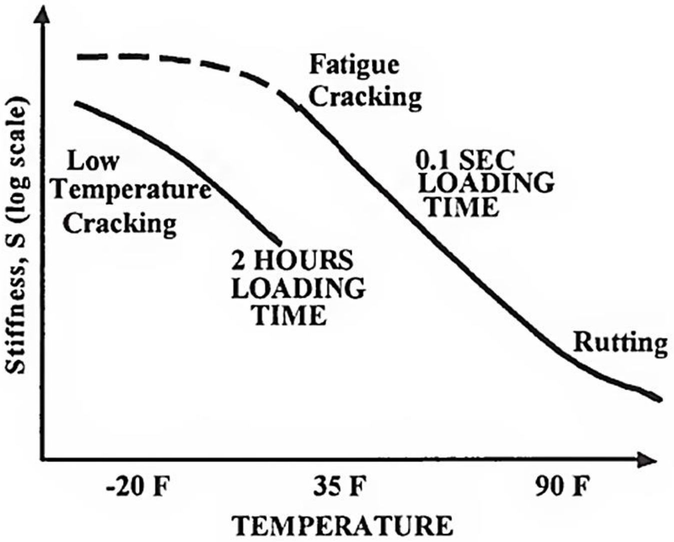


Figure 1. Temperature Dependency of Typical Asphalt Cement at Two Loading Times.

A decrease in penetration and increase in viscosity is the direct sign of aging of an asphalt binder. For most of the cracked pavements, the reclaimed asphalts have a penetration value (measured in units of 0.1 mm) of less than 20 [3]. McLeod and other researchers [4] suggested that when the viscosity of asphalt exceeds 1 x 10^{10} poises at the lowest service temperature, the pavement would undergo thermal cracking. As a rough prediction of thermal cracking, a limit viscosity of 2 x 10^{10} poises has been suggested by Davis [5]. Additionally, many researchers have used aging index to quantify long-term aging that occurs in service. The aging index is defined as the ratio viscosity of the aged bitumen to that of the unaged one. Kandhal P. S. [6] stated that raveling was observed on the test pavements when this aging index (based on viscosity at 25 °C) exceeded 12. Although a single-point aging index can effectively describe the increase in stiffness when the response is essentially viscous, such single-point aging indices will not always accurately reflect changes in stiffness at low temperatures, where delay elasticity is a significant portion of the response. That means that a single-point measurement is insufficient to characterize the rheological change that occurs with aging. Fracture or failure in an asphalt pavement occurs at the critical condition when the combination of thermal condition and the vehicular loading effects produce stress, strain, or energy sufficient to break the asphalt concrete. Thermal and load stressing conditions are major factors to produce pavement cracking [7].

In recent years, fracture mechanics [8] and continuum damage mechanics [9-11] have found increasing use in the analysis and evaluation of asphalt mixtures and pavements at low temperature. Each approach offers advantages and disadvantages with respect to the type of failure mechanism that can be considered. Continuum damage mechanics offers an optional fundamental explanation of damage than conventional fatigue approaches. However, given the fact that only a continuum can be modeled, continuum damage mechanics is incapable of properly addressing the mechanism of crack propagation (i.e., once a crack is introduced, the system is no longer a continuum). The mechanism of the crack propagation has widely been studied by many researches in the past decade [12–20]. The criteria to investigate mixed mode fracture mechanism can mainly be categorized as stress-based [12–15], strain-based [16–19], and energy-based [20]. In addition, damage mechanics does not provide a true physical interpretation of damage; damage may be any change in the material's micro- or macro-structure that results in a reduction in the material's stiffness [21].

Conversely, the fracture mechanics approach assumes there are inherent flaws in the material. When loading occurs, there are higher stress concentrations around the flaws because load is distributed over a smaller area, which means the material no longer has a uniform stress distribution. Fracture mechanics describes the propagation of cracks through materials. Paris and Erdogan [22] developed a crack rate law for use in linear elastic homogeneous materials. Since asphalt concrete is a heterogeneous viscoelastic material, so Paris' Law may not completely explain how asphalt mixtures behave. For example, cracks are known to grow discontinuously in asphalt concrete mixtures [23]. Paris' Law does not explain crack initiation because it assumes cracks are already present. Despite the fracture mechanics approach's flaws, crack length is a measurable interpretation of damage, which provides a sound base from which to proceed [21].

The primary reason for using recycled tire rubber is that it provides significantly improved engineering properties over conventional paving grade bitumen [24]. To date, field performance has been very good. As an extra benefit, the ground tire rubber from over eight and one half million tires in Arizona has been recycled since 1988, in the making of HMA with AR [25]. As demonstrated by various studies [25–31], recycled tire rubber obtained through the wet process have reduced fatigue and reflection cracking, greater resistance to rutting, improved aging and oxidation resistance and better chip retention due to thicker binder films. By using the recycled tire rubber for asphalt pavements field trial and evaluation in Taiwan, it can be successfully constructed and performed better than the conventional dense-graded mixes by using local materials and paving techniques [32]. A summary of the benefits of using recycled tire rubber for road pavement applications is given: (a) Improved durability, (b) Savings in energy and natural resources by using waste products, (c) Improved resistance to surface initiated and fatigue/reflection cracking, (d) Reduced temperature susceptibility, (e) Improved aging and oxidation resistance, (f) Improved resistance to rutting, (g) Reduced traffic noise, (h) Lower pavement maintenance costs, (i) Improved safety due to better long-term colour contrast for pavement markings, (j) Reduced construction times due to thinner lifts [24].

The objectives of this study are: (1) performing diametral resilient modulus test, indirect tensile creep test and indirect tensile strength test on modifier asphalt mixtures aged according to the SHRP short term oven aging (STOA) and long term oven aging (LTOA) procedures; and (2) evaluate the fracture properties of aged asphalt mixtures containing recycled tire rubber or styrene-butadiene rubber modifiers at low temperatures.

2. Materials and Methods

This research investigates the fracture properties and effects of aged asphalt mixtures containing ground tire rubber (GTR) and styrene-butadiene rubber (SBR) modifiers at low temperatures. Three different asphalts (conventional and modified) were blended with an aggregate with a fixed gradation and at 6.5% asphalt content in the laboratory to produce mixtures complying with the Florida DOT S-I mix specification. The three binders used were (1) AC30, (2) AC30 + 10%GTR, and (3) AC10 + 3%SBR. Physical properties of the modified asphalt binders are listed in Table 1 and compared with the AC30 binder. The aggregate blend was obtained by blending a Florida Mining Materials #67 stone, S1-B, screening, a Canal Auth sand, and a mineral filler. Table 2 lists the gradations of the individual aggregates and the gradation of the aggregate blend used also meets the Florida DOT S-I (structural) mix specifications for a 19.0 mm mixture. The control specimens were those made of unaged mixtures compacted by the gyratory testing machine (GTM). Marshall-size specimens were compacted in the GTM to two compaction levels which are to simulate (1) the initial field compaction and (2) the condition after five to ten years of traffic on the pavement. The initial (field) compaction condition was achieved by 18 GTM revolutions to the samples at 275-300°F.

The ultimate compaction condition was achieved by 200 GTM revolutions to the samples at 140°F. The other mixtures were subjected to the SHRP proposed short term oven aging (STOA) and long term oven aging (LTOA) processes. After the STOA or LTOA process, all mixtures were compacted by the gyratory testing machine at initial and ultimate compactions, 18 and 200 revolutions respectively, to reach air contents of 6 to 7 and 3 to 4 percents. Nine replicate samples were fabricated and tested for each condition. In addition, for each type of mixture, two extra samples were made in order to run the Rice specific gravity tests. This amounts to a total of (9x3 x3 x2+2x3 x3=) 180 samples were tested in this study. The compacted asphalt mixture samples were evaluated by the following tests:

(1) Diametral resilient modulus and indirect tensile creep tests at temperatures of 0, -10 and -20°C.
(2) Indirect tensile strength tests at a temperature of - 10°C.

Table 1. Physical Properties of the Base and Modified Asphalt Binder.

Characteristics	AC30	AC30+GTR	AC10+SBR
Penetration (25°C)	61.5	44.1	86.3
Ductility (25°C) (cm)	＞100	87	＞100
Viscosity (60°C,poise)	3409	15847	2705
Fraass Breaking Point (°C)	-12.3	-12.5	-16.8
Flash Point (°C)	295	310	278
Performance grade	PG64-28	PG76-28	PG64-34

Table 2. Gradation of Aggregates and Job Mix Formula for Asphalt Mixture.

Aggregate Type	#67 stone	S-IB	Screening	Sand	Mineral Filler	S-I Mixture Specification	Job Mix Formula
Sieve (mm)	Passing (%)						
3/4in. (19.0)	100	100	100	100	100	100	100
1/2in. (12.5)	72.2	100	100	100	100	88-100	94
3/8in. (9.51)	25.0	89.5	100	100	100	75-93	84
#4 (4.76)	4.7	14.7	97.3	100	100	47-75	57
#10 (2.00)	1.0	1.0	61.2	100	100	31-53	42
#40 (0.420)	0.5	0.4	16.0	72.8	100	19-35	22
#80 (0.177)	0.4	0.2	2.3	15.0	99.1	7-21	8
#200 (0.074)	0.0	0.0	0.0	0.0	85.8	2-6	4

2.1 Diametral Resilient Modulus Test

Diametral resilient modulus test was run using the test set-up for the SHRP Indirect Tensile Test at Low Temperatures (ITLT), which is showed in Figure 2 [7]. The resilient modulus is defined as the ratio of the applied stress to the recoverable strain when a repeated dynamic load is applied. The resilient modulus test is run in a load-control mode and involves applying a repeated have sine waveform load to the specimen for a period (0.1 second). The load was selected to keep horizontal strains in the linear viscoelastic range (typically between 150 and 350 microstrains during a 5 second) during the resilient modulus test and is applied vertically in the vertical diametral plane of a cylindrical specimen of asphalt mixture. Poisson's ratio, which is essential for accurate determination of resilient modulus in this method, is determined using the measured horizontal and vertical deformations. Resilient modulus can be determined using this Poisson's ratio along with repeated load, specimen dimensions, and the time-dependent horizontal deflection, using equations, which were developed, based upon three dimensional finite element analyses by Roque and Buttlar [11].

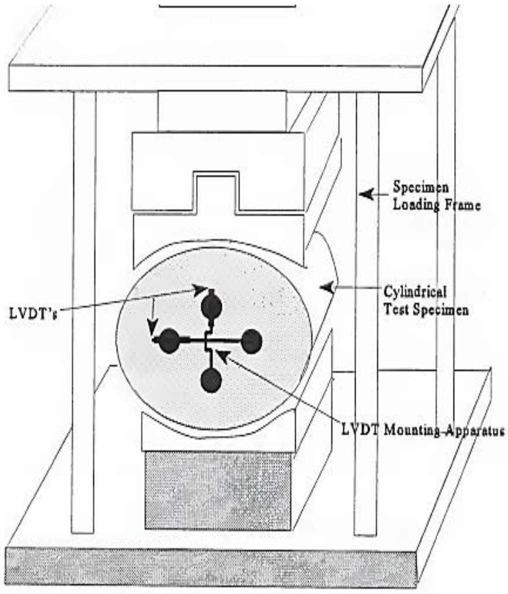


Figure 2. Sketch of Diametral Resilient Modulus Test and Indirect Tensile Test [7].

2.2 Indirect Tensile Creep Test

The SHRP Indirect Tensile Test at Low Temperatures (ITLT) was used to measure the creep properties of the mixtures. The creep test is run in a load-control mode and involves applying a static load very quickly and then holding it constant throughout the duration of the test. The horizontal and vertical deformations measured near the center of the specimen are used to calculate tensile creep compliance as a function of time. The load was selected to keep horizontal strains in the linear viscoelastic range (typically between 40 and 120 microstrains at t=30 seconds) during the creep test. Poisson's ratio, which is essential for accurate determination of creep compliance in this method, is determined using the measured horizontal and vertical deformations. The development and evaluation of this test method and specific test procedures were presented in detail by Buttlar and Roque [11].

2.3 Indirect Tensile Strength Test

The indirect tensile strength is the maximum tensile stress that a specimen can tolerate before fracture. This mix property is useful for predicting thermal cracking potential and characterizing the resistance to failure of asphalt concrete caused by tensile stresses. The tensile strength test was set up immediately after accomplishing the tensile creep test by changing load-control mode to displacement-control mode. The ramp movement to the specimen at a rate of 0.5-inch per minute for 4-inch-diameter specimens was used in this study. Data were collected at a rate of 20 Hz for the duration of the test. Both the horizontal and vertical deformations as well as the applied load can be recorded during the test. The average peak stress was calculated from peak-applied forces obtained by three replicate specimens. The applied force could be extracted from the recorded data of the testing machine. This test method is similar to ASTM D6931-17 [33] and covers procedures for preparing and testing laboratory-fabricated or field-recovered cores of asphalt mixtures to determine the indirect tensile (IDT) strength. The indirect tension strength of concrete tested by the Super pave IDT strength test was calculated as:

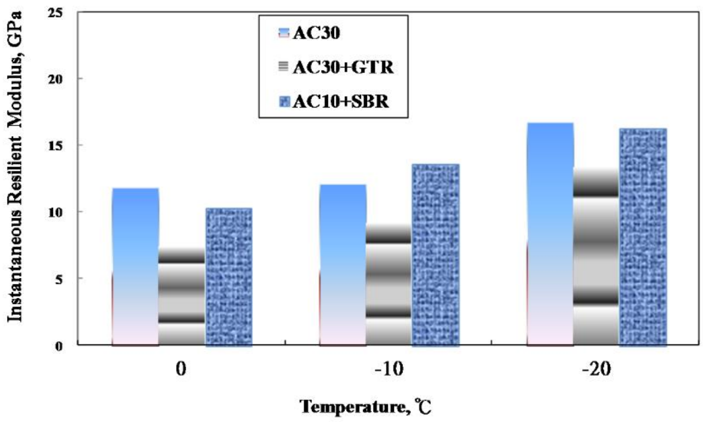
$$\mathrm{T_{IDT}}=\frac{2\mathrm{P}}{\pi \mathrm{d}\times \mathrm{l}} \tag{1}$$

Where T_{IDT} = the indirect tensile strength; P = the peak applied load;
d = the diameter of concrete specimen; l = the thickness of concrete specimen.

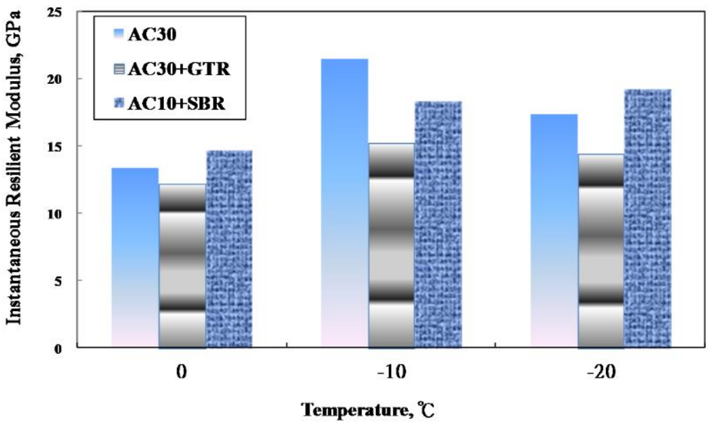
3. Results and Discussion

3.1 Results of Diametral Resilient Modulus Test

Comparisons of the instantaneous and total resilient moduli plots for unmodified and modified asphalt mixtures with similar air void content at 0, -10 and -20℃ are shown in Figures 3 and 4. It can be seen that the GTR-modified asphalt mixtures before and after the LTOA process have lower instantaneous and total resilient moduli as compared with the unmodified mixtures. The mixtures with SBR-modified asphalt have lower air void content and exhibit approximately the same instantaneous and total resilient moduli as compared with the unmodified mixtures. Mixtures with high resilient modulus at a low temperature are considered too brittle to resist the thermal and load associated cracking. At low temperatures, a low elastic modulus is beneficial to prevent a rapid accumulation of tensile stresses and strains during temperature drops. Because of the very short loading time (0.1 second) and very low temperatures involved in the resilient modulus test, some test results were inconsistent at temperatures of -10 and -20℃. The cause of the inconsistency might be due to micro damage sustained in the asphalt mixture upon cooling to low temperatures, or that the LVDT was frozen and could not respond normally. The inconsistency could also be due to different air void contents of the mixtures at different aging conditions. In general, both the STOA and LTOA aging processes tend to increase the resilient modulus at temperatures of 0, -10 and -20℃. The instantaneous resilient moduli are slightly higher than the total resilient modulus for all mixtures before and after the STOA and LTOA aging processes.

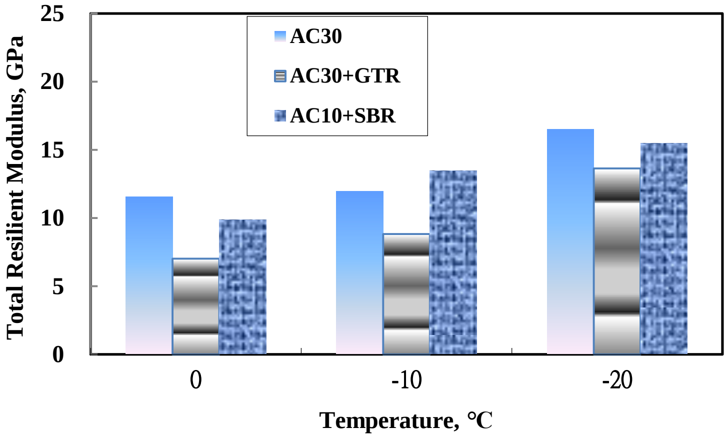


(a) Unaged Samples at Initial Compaction

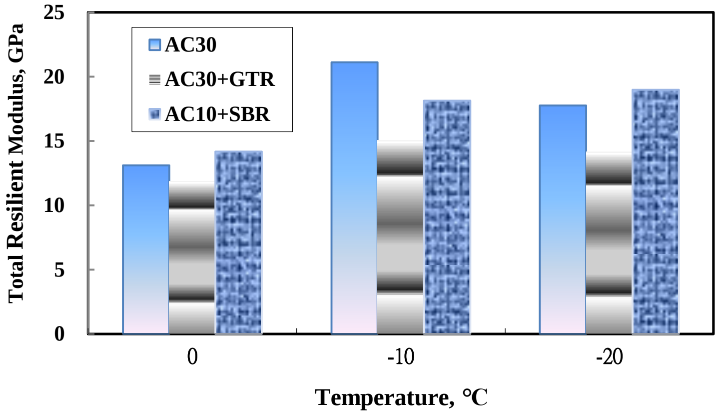


(b) LTOA Samples at Ultimate Compaction

Figure 3. Comparison of Instantaneous Resilient Moduli of Mixtures.



(a) Unaged Samples at Initial Compaction



(b) LTOA Samples at Ultimate Compaction

Figure 4. Comparison of Total Resilient Moduli of Mixtures.

3.2 Results of Indirect Tensile Creep Tests

Asphalt mixtures exhibit viscoelastic behavior. Viscoelastic deformation is the result of the combined elastic and viscous response of the material when subjected to an applied load. The rheological behavior of asphalt mixtures can be modeled by using different combinations of Hookean springs and Newtonian dashpots. Maxwell, Kelvin, Van der Poel, Burgers, and Kuhn and Rigden models are well-known rheological models which make use of a combination of Hookean springs and Newtonian dashpots. Among these models, the Burgers model is a simple model that can model this behavior well. The slope and intercept from the Burgers model were used to evaluate and analyze indirect tensile creep data in this study. This model is illustrated in Figure 5. In this model, the top spring models the purely elastic deformation, the combined middle spring and dashpot model the delayed elastic deformation, and the lower dashpot models the purely viscous flow. Creep testing of asphalt mixtures provides information on elastic, delayed elastic and viscous components. The creep compliance from a creep test represents the strain caused by a unit stress. It can be represented by the response of a Burgers model subject to a unit applied stress. The creep compliance can be expressed as:

$\mathbf{1/E_0 + l/E_1 x(l-e^{-tE_1/\eta_1})+t/\eta_0}$, where the time t and can be expressed in the terms of the elastic moduli E_0, E_1 and the viscosities η_0, η_1. It can be noted that at large t the slope of the creep compliance plot approaches $1/\eta_0$. The intercept of the creep compliance plot with the vertical axis is equal to $(1/E_0 + l/E_1)$, which can be considered to be equal to 1 / (total elastic stiffness) of the mixture.

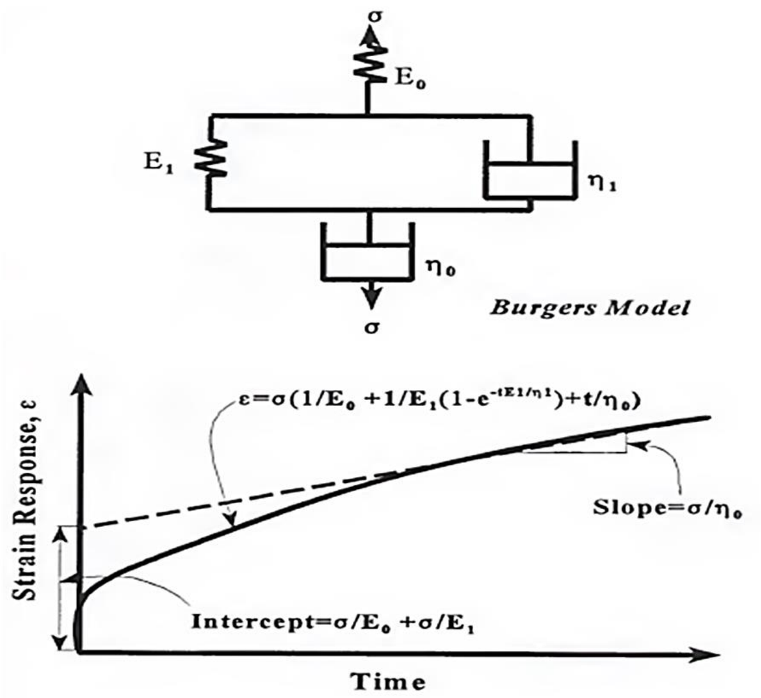


Figure 5. Burgers model.

The values of creep compliance at 19 different times and one average Poisson's ratio are reported in Tables 3 and 4. Comparisons of creep compliance at 1000 seconds for all mixtures at different temperatures and after different aging effects are shown in Figure 6. In general, the creep compliance reduces as temperature decreases for all mixtures. The creep compliance also slightly decreases after the STOA or LTOA process for all mixtures.

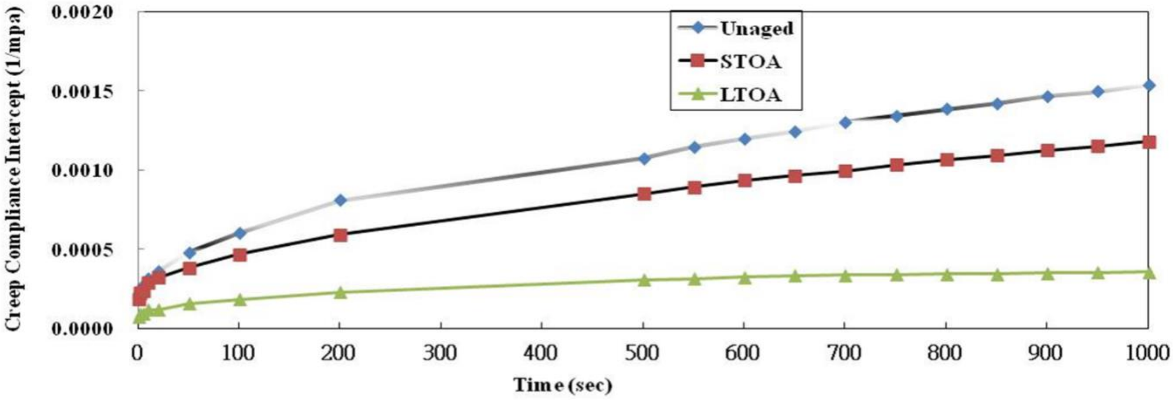
Table 3. Results of Analysis of Creep Compliance Data for Samples of Initial Compaction.

Asphalt	Aging Condition	Temp. ℃	Air Void (%)	Intercept (1/MPa)	Slope (1/MPa)	R^2	Poisson's Ratio
AC30	Unaged	0	8.5	2.95E-04	5.27E-07	0.997	0.16
		-10	8.4	2.34E-04	1.51E-07	0.995	0.19
		-20	8.3	1.03E-04	2.27E-08	0.958	0.25
	STOA	0	11	3.89E-04	4.59E-07	1.000	0.21
		-10	11.3	1.83E-04	1.50E-07	0.994	0.24
		-20	11.6	1.24E-04	6.63E-08	0.998	0.29
	LTOA	0	11.4	2.20E-04	3.19E-07	0.989	0.24
		-10	11.3	1.75E-04	1.39E-07	0.997	0.29
		-20	11.2	1.02E-04	7.94E-08	0.997	0.31
AC30+GTR	Unaged	0	7.0	6.52E-04	9.01E-07	0.995	0.21
		-10	6.8	3.35E-04	5.44E-07	1.000	0.23
		-20	6.6	1.50E-04	8.71E-08	0.995	0.24
	STOA	0	7.4	5.39E-04	6.48E-07	0.998	0.12
		-10	7.6	2.84E-04	3.04E-07	0.998	0.18
		-20	7.8	1.94E-04	1.61E-07	0.999	0.23
	LTOA	0	7.7	2.63E-04	9.67E-08	0.972	0.44
		-10	7.8	1.27E-04	1.65E-07	0.997	0.33
		-20	7.9	9.65E-05	4.57E-08	0.988	0.32

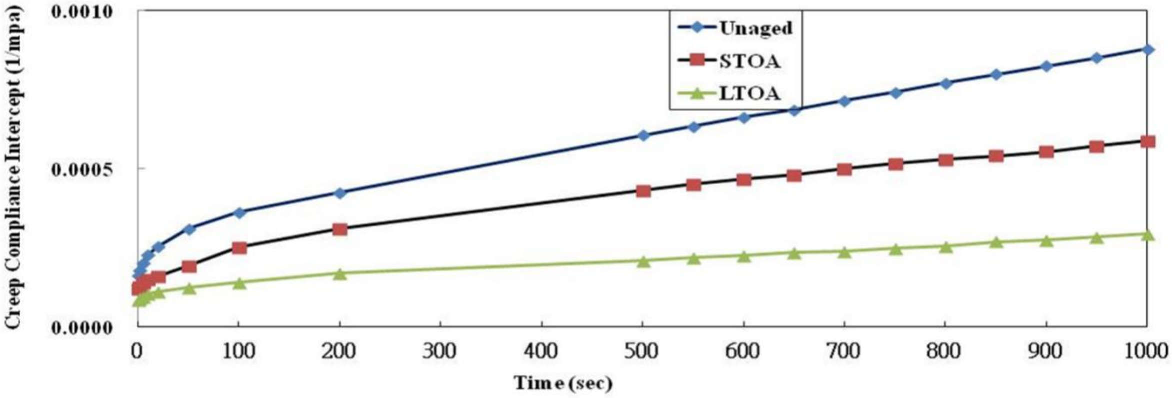
AC10+SBR	Unaged	0	6.7	4.72E-04	5.98E-07	0.998	0.21
		-10	6.8	2.16E-04	3.43E-07	0.999	0.25
		-20	6.9	1.06E-04	4.70E-08	0.998	0.40
	STOA	0	7.2	4.57E-04	6.50E-07	0.999	0.22
		-10	7.0	1.55E-04	1.00E-07	0.998	0.33
		-20	6.8	8.55E-05	4.24E-08	0.998	0.33
	LTOA	0	6.8	2.91E-04	2.85E-07	0.990	0.38
		-10	7.0	1.60E-04	1.37E-07	0.997	0.30
		-20	7.2	1.06E-04	5.60E-08	0.994	0.26

Table 4. Results of Analysis of Creep Compliance Data for Samples of Ultimate Compaction.

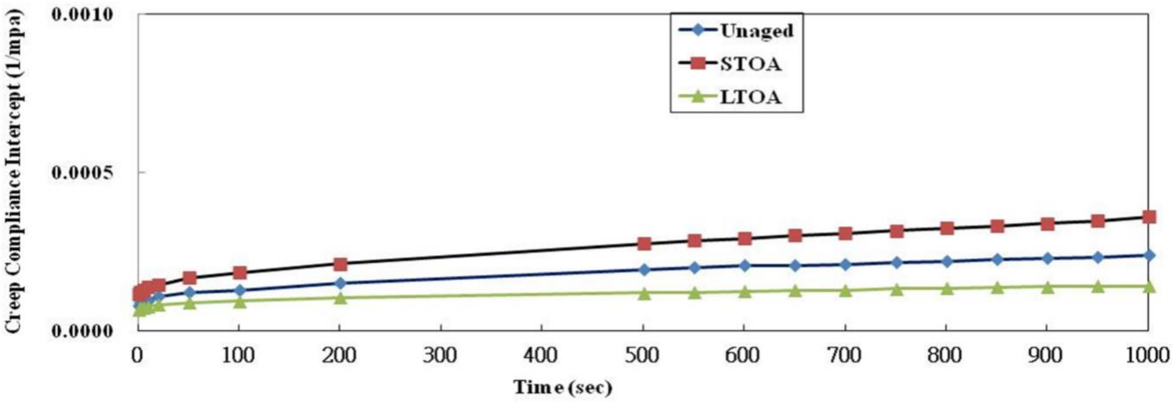
Asphalt	Aging Condition	Temp. ℃	AirVoid(%)	Intercept (1/MPa)	Slope (1/MPa)	R^2	Poisson's Ratio
AC30	Unaged	0	4.5	4.31E-04	4.08E-07	0.994	0.16
		-10	4.5	3.48E-04	2.89E-07	0.999	0.14
		-20	4.6	1.61E-04	8.83E-08	0.996	0.29
	STOA	0	5.1	3.03E-04	4.61E-07	0.998	0.33
		-10	4.8	1.92E-04	1.49E-07	0.992	0.30
		-20	4.5	6.41E-05	1.88E-08	0.971	0.50
	LTOA	0	4.5	2.57E-04	1.78E-07	0.994	0.12
		-10	4.7	1.05E-04	3.44E-08	0.993	0.32
		-20	4.9	8.59E-05	3.58E-08	0.984	0.26
AC30+GTR	Unaged	0	4.7	3.47E-04	3.59E-07	0.997	0.22
		-10	4.6	1.86E-04	1.10E-07	0.996	0.20
		-20	4.5	7.47E-05	2.44E-08	0.997	0.50
	STOA	0	4.8	3.16E-04	1.98E-07	0.983	0.18
		-10	4.9	1.64E-04	5.40E-08	0.952	0.17
		-20	5.0	9.37E-05	3.12E-08	0.977	0.31
	LTOA	0	4.7	2.81E-04	2.26E-07	0.993	0.25
		-10	4.9	1.35E-04	1.07E-07	0.998	0.32
		-20	5.1	1.00E-04	3.67E-08	0.994	0.26
AC10+SBR	Unaged	0	3.5	3.53E-04	3.89E-07	0.998	0.33
		-10	3.6	2.36E-04	1.99E-07	0.989	0.35
		-20	3.7	9.7E-05	6.27E-08	0.999	0.32
	STOA	0	3.8	3.79E-04	4.67E-07	0.998	0.22
		-10	3.7	1.91E-04	1.39E-07	0.996	0.25
		-20	3.6	7.49E-05	2.24E-08	0.980	0.50
	LTOA	0	3.6	1.94E-04	1.40E-07	0.993	0.41
		-10	3.7	1.17E-04	7.82E-08	0.994	0.29
		-20	3.9	8.09E-05	4.46E-08	0.995	0.25



(a) 0℃ (AC30+GTR at Initial Compaction)



(b) -10℃ (AC30+GTR at Initial Compaction)

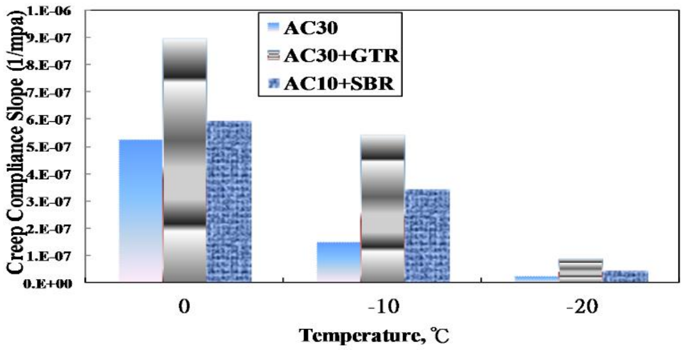


(c) -20℃ (AC30+GTR at Initial Compaction)

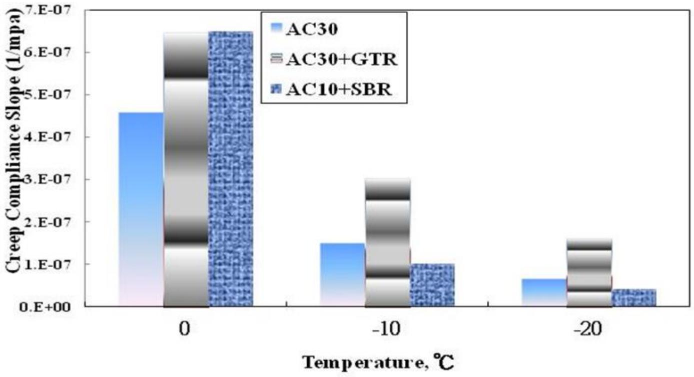
Figure 6. Comparison of Creep Compliance of Mixtures at Different Aged Conditions.

An investigation was made to determine when the creep compliance versus time plot reaches a steady state. The instantaneous slopes at different times were compared to the slope at 1000 seconds. It was found that in general at times beyond 500 seconds, the observed slope changes by less than 1%, which is of the same magnitude as the expected experimental error from this test. Thus, it was concluded that, in most cases, steady state condition was reached at 500 seconds.

Tables 3 and 4 summarize the air void contents of the mixtures, and the intercepts and slopes of the creep compliance plots, of the regression line for determination of slopes and intercepts, and Poisson's ratio at 0, -10 and -20℃ for all mixtures tested at initial and ultimate compaction conditions. Comparisons of the slopes of the creep compliance versus time plots for unmodified and modified asphalt mixtures with similar air void content at 0, -10 and -20℃ are shown in Figure 7. At 0℃, the SBR modified asphalt mixtures after the LTOA process have lower creep compliance slopes as compared with the unmodified mixtures. At the test temperatures of -10 and -20℃, the mixtures with SBR- or GTR-modified asphalt exhibit substantially higher creep compliance slopes as compared with the unmodified mixtures. A mixture with higher creep compliance slope (i.e. low viscous stiffness) would strain more through viscous deformation and not build up tensile stresses at low temperatures. Also, comparisons of the intercepts of the creep compliance versus time plots for unmodified and modified asphalt mixtures with similar air void content at 0, -10 and -20℃ are shown in Figure 8. It can be seen that the GTR modified asphalt mixtures before and after the LTOA process have higher creep compliance intercepts as compared with the unmodified mixtures. The mixtures with SBR modified asphalt have lower air void content and exhibit approximately the same creep compliance intercepts as compared with the unmodified mixtures at -10 and -20℃.

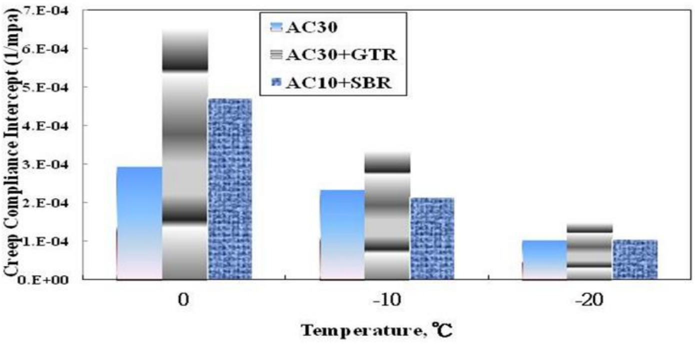


(a) Unaged Samples at Initial Compaction

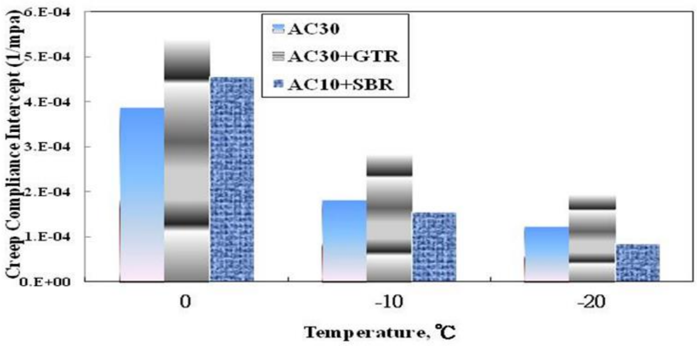


(b) LTOA Samples at Ultimate Compaction

Figure 7. Comparison of Creep Compliance Slopes for Different Asphalt Mixes.



(a) Unaged Samples at Initial Compaction



(b) LTOA Samples at Ultimate Compaction

Figure 8. Comparison of Creep Compliance Intercepts for Different Asphalt Mixes.

3.3 Results of Indirect Tensile Strength Tests

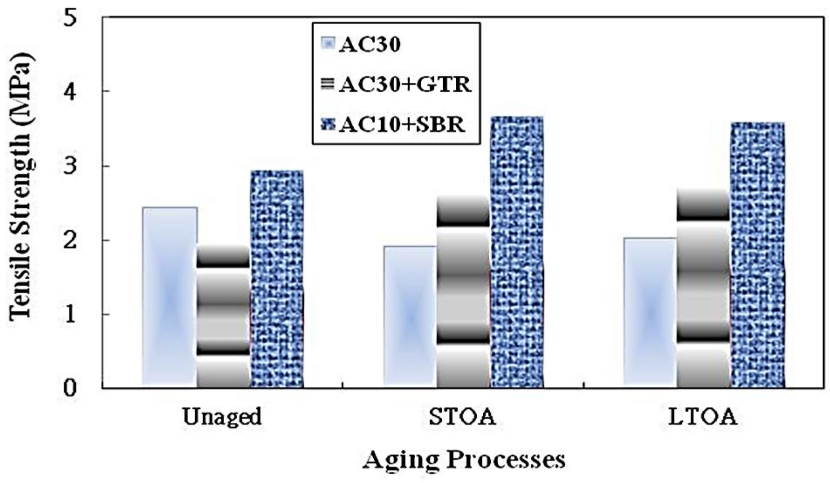
The indirect tensile strength is the maximum tensile stress that a specimen can tolerate before fracture. It indicates the resistance to failure of asphalt concrete caused by tensile stresses and can be used for predicting the thermal cracking potential of a mixture. Asphalt mixtures that can tolerate high strains prior to failure are more likely to resist cracking than the mixes that cannot tolerate high strains. Results of the indirect tensile strength tests at -10℃ for all mixtures at initial and ultimate compaction conditions are shown in Tables 5 and 6. Reported are air void content, tensile strength, horizontal strains at failure, and fracture energy. This test was only performed at -10℃ on samples which had previously been tested by resilient modulus and indirect tensile creep tests. It can be seen that the air void content of the samples strongly affects their tensile strength. Mixes with higher air voids tend to have lower tensile strengths. However, horizontal strain at failure is apparently not affected by air void content. The test results indicated that the SBR modified mixtures have higher tensile strength at both initial and ultimate compaction. The high tensile strengths of SBR modified samples are partly attributed to the low air void content of the compacted samples. However, the aging processes did not have significant effects on the tensile strengths of the samples. Comparisons of tensile strengths for all of the test mixtures are shown in Figure 9. These plots again show that the SBR-modified mixtures have higher tensile strengths than the other mixtures at similar compaction and aging conditions.

Table 5. Results of Indirect Tensile Strength Test at -10℃ for Samples of Initial Compaction.

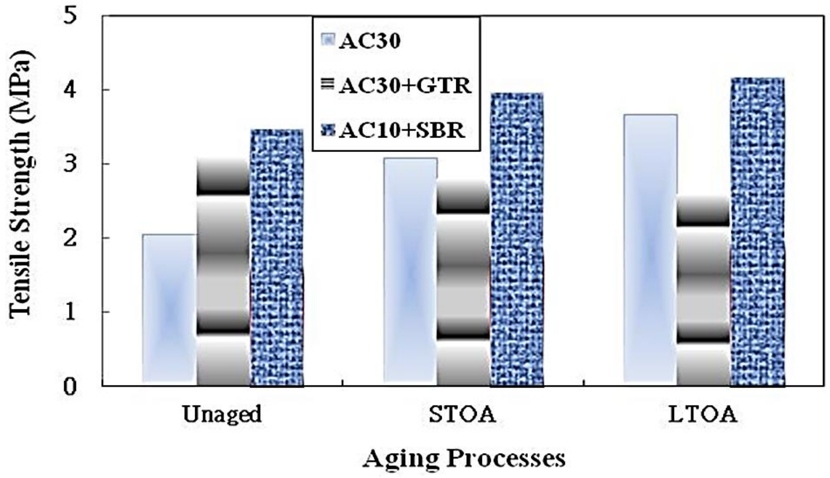
Asphalt	Aging Condition	Air Void (%)	Tensile Strength (MPa)	Failure Strain (10^{-3} mm/mm)	Fracture Energy (J/m^2)
		Means of 3 replicates/Standard deviation			
AC30	Unaged	8.4/1.45	2.44/0.60	15.63/3.81	7.78/2.60
	STOA	11.30/0.56	1.92/0.34	11.56/1.51	3.84/1.02
	LTOA	11.30/0.61	2.03/0.38	10.46/3.66	3.76/1.62
AC30+GTR	Unaged	6.80/0.17	1.96/0.11	21.11/4.47	7.72/2.74
	STOA	7.80/0.92	2.62/0.28	18.47/3.62	8.71/2.55
	LTOA	7.80/0.69	2.71/0.37	16.96/3.30	8.09/2.09
AC10+SBR	Unaged	6.80/0.36	2.96/0.26	24.26/4.31	13.95/2.87
	STOA	7.00/0.10	3.68/0.17	18.80/9.42	12.61/8.88
	LTOA	7.00/0.35	3.61/0.35	18.61/9.82	12.13/9.34

Table 6. Results of Indirect Tensile Strength Test at -10℃ for Samples of Ultimate Compaction.

Asphalt	Aging Condition	Air Void (%)	Tensile Strength (MPa)	Failure Strain (10^{-3} mm/mm)	Fracture Energy (J/m^2)
		Means of 3 replicates/Standard deviation			
AC30	Unaged	4.5/0.87	2.05/0.61	15.75/2.77	5.68/2.11
	STOA	4.73/0.70	3.08/1.31	17.55/6.36	10.74/7.69
	LTOA	4.70/0.17	3.66/0.40	15.61/3.74	10.40/3.60
AC30+GTR	Unaged	4.60/0.17	3.10/0.09	20.51/0.19	12.00/0.37
	STOA	4.90/0.17	2.79/0.37	15.70/2.75	7.90/1.66
	LTOA	4.90/0.10	2.59/0.13	15.06/3.03	7.41/1.91
AC10+SBR	Unaged	3.60/0.10	3.48/0.57	23.88/4.16	14.51/4.50
	STOA	3.70/0.10	3.97/0.03	24.15/2.87	18.23/5.12
	LTOA	3.70/0.10	4.18/0.25	22.93/4.44	16.39/3.20



(a) Initial Compaction



(b) Ultimate Compaction

Figure 9. Comparison of Indirect Tensile Strengths of Mixtures.

3.4 Fracture Energy

Fracture energy is the energy required to cause fracture of an asphalt mixture. It is determined by calculating the total area under the stress-strain curve from an indirect tensile strength test. Comparisons of fracture energies at test temperature of -10°C for different material effects and different aging effects are shown in Figures 10 and 11. The test results indicated that the fracture energy slightly decreases after the STOA or LTOA process for the samples of initial compaction at -10°C. This shows that the fracture energy value decreases as the asphalt binder hardens. As seen from these plots, all the SBR-modified asphalt mixtures have higher fracture energy at -10°C than that of mixtures containing the unmodified AC30 and the GTR-modified asphalt. At initial and ultimate compaction conditions, the SBR-modified mixture has higher fracture energy than the GTR-modified mixture and the unmodified one. Both the STOA and LTOA aging processes tend to decrease the fracture energy at temperatures of -10°C in general, but the SBR-modified asphalt mixtures tend to increase at ultimate compaction condition.

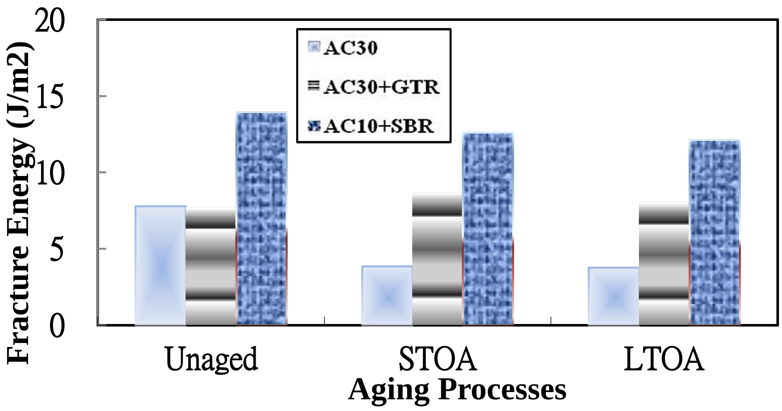


Figure 10. Comparison of Fracture Energies of Mixtures at Initial Compaction.

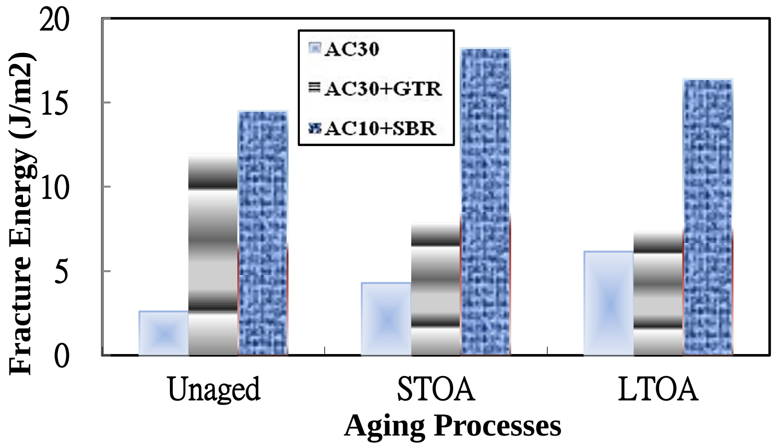


Figure 11. Comparison of Fracture Energies of Mixtures at Ultimate Compaction.

4. Summary

In this paper, the investigation was made on the fracture properties of aged asphalt mixtures containing recycled tire rubber or styrene-butadiene rubber modifiers by using the Superpave indirect tensile test at 0, -10 and -20°C. The following summary was drawn, according to the results obtained.

As compared with the SBR-modified asphalt mixtures were seen to have higher creep compliance, tensile strength, failure strain and fracture energy at low temperatures, which indicate a greater potential to resist low temperature cracking. The GTR-modified asphalt mixtures showed lower resilient modulus than that of the unmodified mixtures. It was found that the SBR-modified asphalt mixtures did not exhibit a lower viscosity (low creep compliance slope) than the unmodified mixtures after the LTOA process at 0°C. However, at -10 and -20°C, the SBR-modified asphalt mixtures exhibit substantially lower viscosity (higher creep compliance slope) than the unmodified mixtures. The GTR-modified asphalt mixtures have lower elastic stiffness (higher creep compliance intercepts) as compared with the unmodified mixtures before and after the LTOA process. The SBR-modified asphalt mixtures exhibit approximately same elastic stiffness (creep compliance intercepts) and resilient moduli as compared with the unmodified mixtures at -10 and -20°C before and after the LTOA process. At initial and ultimate compaction conditions, the SBR-modified mixture has higher fracture energy than the GTR-modified mixture and the unmodified one. Based on indirect tensile creep compliance, tensile strength, failure strain and fracture energy, the GTR and SBR-modified mixtures show a greater potential to resist low temperature cracking as compared with the unmodified mixtures.

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