FINAL REPORT

GUIDELINES FOR USE OF MODIFIED BINDERS

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16. Abstract

The Superpave mix design procedure improved mixture quality; however, it still remains a volumetric design devoid of performance tests. A mixture produced for high traffic volumes (above 10 million ESALs), results in low design asphalt content due to the increased gyrations. The lower binder content, although enhancing the rutting performance, might increase the cracking susceptibility of the mix. Modified asphalts have gotten a lot of attention during the past few years due to the promise of enhanced mixture performance.

This study consolidated work done in Florida with polymer-modified asphalts – mainly Ground Tire Rubber (GTR) and Styrene Butadiene Styrene (SBS). Issues with the use of modifiers as well as their relative performance have been examined and summarized. Information reviewed included data from the Heavy Vehicle Simulator (HVS), laboratory experiments — Asphalt Pavement Analyzer (APA), Servopac, and Indirect Tension Test (IDT) — as well as field results.

Test results clearly show that SBS-modified mixtures outperform control mixtures in rutting experiments, and laboratory information indicates that the same holds for cracking. The basic benefit of GTR is that it can increase binder content without drain-down; the increased binder content subsequently improves cracking resistance. Use of SBS-modified asphalts appears warranted and cost effective. SBS-modified asphalt is highly recommended for intersections (high volume, slow moving traffic) and OGFC; however, environmental benefit of use of rubber in pavements cannot be overlooked (investigate hybrid binder).

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yards	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	ml
gal	gallons	3.785	liters	l
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams	Mg
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5(F-32)/9 or (F-32)/1.8	Celcius temperature	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	l
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
psi	poundforce per square inch	6.89	kilopascals	kPa

NOTE: Volumes greater than 1000 l shall be shown in m^3.

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
ml	milliliters	0.034	fluid ounces	fl oz
l	liters	0.264	gallons	gal
m^3	cubic meters	35.71	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg	megagrams	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact)		
°C	Celcius temperature	1.8C + 32	Fahrenheit temperature	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	psi

* SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.

(Revised August 1992)

EXECUTIVE SUMMARY

The current asphalt mix design procedure – Superpave Level 1 – essentially remains a volumetric design procedure devoid of validated performance-based tests for asphalt mixtures. The current procedure assumes the number of gyrations applied by a gyratory compactor resemble the traffic conditions to which the mixture will be subjected. The design asphalt content is selected to produce 4% air voids at the design number of gyrations for a particular level of traffic and environment. Mixtures produced with conventional asphalt binders, particularly those on high traffic volume facilities (high number of gyrations), may not have adequate resistance to cracking as a result of lower design asphalt content.

Based on the above observations, adequate rutting and cracking performance may not always be attainable for high-traffic volume Superpave mixtures designed with conventional asphalt cement. Research results, however, point to the use of modifiers as a way to produce a mixture with desirable rutting resistance, as well as sufficient fracture resistance at lower in-service temperatures. This study was undertaken to consolidate and evaluate work that has been done in Florida in the area of modified asphalt mixtures.

The primary findings of this work may be summarized as follows:

- Ground tire rubber (GTR) has been used in open and dense graded friction courses. The main benefit is that GTR can increase binder content while preventing drain-down; the increased binder content subsequently improves cracking resistance. For dense-graded mixtures (tested with 12% rubber), however, GTR creates problem with the aggregate structure; it prevents the aggregate structure from achieving maximum shear strength orientation.

- Field data for polymer-modified mixture-performance in Florida is mostly anecdotal, not involving controlled scientific studies. In most cases observations indicate good rutting performance relative to prior history in existing location. Experiments with the Heavy Vehicle Simulator (HVS) as well as other laboratory tests – APA, GTM, Servopac – showed that SBS-modified binder (PG 76-22) out-performed the control binder (PG 67-22) in rutting performance.

- Indirect Tension Test (Superpave IDT) results analyzed with the HMA Fracture Mechanics model showed that the SBS modified mixtures could benefit cracking, mainly by reducing the rate of damage accumulation. SBS modifiers, however, do not have any effect on resilient modulus or the creep-energy of the material. A short loading time test, including complex modulus, is not able to capture the benefits of the modifier.

- The added cost of SBS-modified binder amounts to $100 per ton of liquid binder, or about $6 to $8 per ton of HMA (10 to 15% price increase in total cost). This may be reduced with continued use; case-by-case scenario results in higher costs because contractors need to use different storing tanks with agitators, purchase the binder in smaller quantities, etc.

- Based on the HMA Fracture Model, for pavements with sufficient structure (i.e. HMA thickness not required for SN, or assuming SN obtained from base) then SBS reduces required thickness that results in 5-30% reduction in initial cost depending on traffic level; not considering improved life-cycle cost.

The following recommendations are based on the findings and conclusions from this study:

- A pavement design procedure that accurately reflects benefits of polymer modified asphalt (PMA) should be identified/developed; complex modulus will not be able to capture the SBS modifier benefits.

- Production issues associated with PMA should be further investigated (i.e. effect of moisture in absorptive and non-absorptive aggregate, absence of tender zone).

- Use of SBS modified asphalts appears warranted and cost effective. If sufficient structure is present, for traffic level D or higher, use of SBS-modified asphalt is recommended.

- SBS-modified asphalt is highly recommended for intersections (high volume, slow moving traffic) and open-graded friction-courses (OGFC); however, environmental benefit of use of rubber in pavements cannot be overlooked. A project to investigate a hybrid binder – combination of SBS polymer with rubber – is already under way.

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CHAPTER 1
INTRODUCTION

1.1 Background

The Hveem and Marshall mix-design methods have been used since the 1940s and 1950s. Even though these methods performed well for decades, the increased traffic volume and higher truck-loads in the early 1980s necessitated an improved method to design mixtures for various traffic volumes, axle-loads and environments.

The Strategic Highway Research Program (SHRP) established in 1987 by Congress was instituted with the sole objective of improving the performance and durability of roads in the United States. One-third of the SHRP budget was channeled to the development of performance-based asphalt mix specifications with direct correlation between laboratory analysis and field performance [1]. The Superpave (*Sup*erior *Per*forming Asphalt *Pav*ements) mix design method is one of the outcomes of the SHRP research program.

Superpave mix design has gained considerable popularity among various states across the country, including Florida. Even though the procedure being implemented today remains purely volumetric, it has the following advantages over the traditional Marshall and Hveem mix-design procedures:

- Additional requirements that attempt to eliminate the use of substandard or unacceptable aggregates
- Selection of binders using fundamental properties that incorporates or takes into account a broader range of in-service temperatures

- Gyratory compaction that more closely simulates field compaction and traffic conditions

In spite of the improvements, the Superpave Level 1 mix design procedure – being implemented today – essentially remains a volumetric design procedure devoid of validated performance-based tests for asphalt mixtures. The volumetric design procedure assumes that the number of gyrations applied by the Superpave gyratory compactor represents the traffic conditions to which the mixture will be subjected. The asphalt content is designed to produce 4% air voids at the design number of gyrations for a particular level of traffic and environment. No other material properties are currently required to determine the mixture's susceptibility to rutting, cracking and other forms of distress.

Superpave mix design's switch to gyratory compactor has placed stricter requirements on the mixture's shear resistance at higher temperatures. At the same time, there are no appropriate checks to guarantee the mixture's adequate cracking resistance. Research observations from Superpave projects indicate that mixtures produced with conventional asphalt binders, particularly those on high traffic volume facilities, may not have adequate resistance to cracking as a result of lower design asphalt content. This is a direct result of the Superpave design procedure since the increase in number of gyrations (which simulates higher traffic volume) produces lower design asphalt content.

Based on the above observations, adequate rutting and cracking performance may not always be attainable for high-traffic volume Superpave mixtures designed with conventional asphalt cement. Research results, however, point to the use of modifiers as a way to produce a mixture with desirable rutting resistance, as well as sufficient fracture resistance at low in-service temperatures.

1.2 Objectives

The objectives of the project are as follows:

- To consolidate and document work that has been done in Florida in the area of modified asphalt mixtures. A database should be developed to include laboratory, HVS, and field test results as well as all findings from the work

- To summarize all issues that have been identified by this work regarding the advantages and disadvantages associated with the use of modifiers such as production, placement, and cost

- To analyze all of the data obtained to evaluate the relative performance of modified and non-modified mixtures subjected to similar conditions

- To develop recommendations based on the findings, including descriptions that would be incorporated into the FDOT pavement design manual and general guidelines for the use of modified binders

- To use the information obtained to evaluate the relative benefits of ground tire rubber modification in terms of both performance-related benefits and costs

CHAPTER 2
ASPHALT BINDER MODIFIERS

2.1 Overview

Modified bituminous materials can bring real benefits to highway construction and maintenance, by improving performance and/or extending the life of the pavement. The choice of materials (modifiers), however, and their relative performance improvements in Florida is not well documented at present. This chapter will provide some background on modified bituminous materials, particularly materials where the asphalt binder have been modified by the addition of a polymer.

Asphalt modifiers (or additives) have been used in the road construction industry as far back as the early 1950s [2]. However there has been a renewed interest in the use of modifiers in asphalt pavements due to reasons which are not entirely different for its use in the early 1950s, i.e., to improve the performance of asphalt pavements in terms of increased resistance to pavement distresses – cracking, rutting, and stripping.

Modifiers are blended directly with the binder or added to the asphalt concrete mix during production to improve the properties and/or performance of the pavement. Haas et al. [3] gave a comprehensive definition of a modifier:

> An asphalt cement modifier or additive is a material which would normally be added to and/or mixed with the asphalt before mix production, the resulting binder and/or the mix; or where an aged binder is involved, as in recycling, to improve or restore the properties of the aged binder.

Based on the definition above, an ideal asphalt modifier used in HMA aims to the following primary objectives:

- To obtain stiffer mixes at high service temperatures to reduce rutting susceptibility
- To obtain softer mixes at low service temperatures to minimize thermal cracking
- To improve the fatigue resistance of HMA mixes
- To improve the asphalt-aggregate bond to improve resistance to stripping or moisture damage
- To improve resistance to abrasion which also reduces other forms of surface disintegration
- To rejuvenate aged asphalt binders

2.2 Types of Modifiers

Many products are labeled as modifiers in the asphalt industry today, each focusing to mitigate on one or more distress modes. These products range from naturally occurring substances – rubber, Gilsonite, sulphur, and lime – to complex engineered substances – styrene-butadiene-styrene (SBS), styrene-ethylene-butylene-styrene (SEBS), and ethyl-vinyl-acetate (EVA) [2]. More recently, Crossely and Hesp [4] investigated Silane-functionalized polydiamines as a potentially effective asphalt modifier against moisture damage and low temperature cracking. Table 2.1 is a generic classification of modifiers borrowed from Terrel and Walker [5].

2.2.1 Natural Asphalts

Gilsonite and Trinidad Lake Asphalt are naturally occurring asphalts that are most commonly used as additives in HMA pavements. Gilsonite is naturally occurring asphalt with a penetration of 0 to 3 at 77˚ F, and ring and ball softening point between 250 and 350˚ F [2]. It is known to increase the viscosity of HMA mixes resulting in better rutting performance at high service temperatures. However, it also results in mixes with high viscosities at low service temperatures adversely affecting the low temperature cracking performance [6]. Trinidad Lake asphalt has a penetration range of 3-10 at 77˚ F and a softening point in the range of 200 to 207˚ F. It has been used in construction for high stress areas such as intersections and access to toll booths [2].

Table 2-1. Generic classification of asphalt modifiers currently being used or tested in HMA pavements

Modifier Type	Generic Example
1. Fillers	Carbon Black Mineral Fillers (Fly ash, Crusher fines, Lime, Portland cement)
2. Extenders	Sulphur, Lignin
3. Elastomers	
a. Natural Latex	Natural rubber
b. Synthetic Latex	Styrene-butadiene rubber (SBR), Styrene-butadiene-styrene (SBS), Styrene-isoprene-styrene (SIS)
c. Block, Di-block copolymers	Styrene-butadiene di-block copolymers
d. Reclaimed tire rubber	Crumb rubber
4. Plastormers (Thermoplastics)	Ethyl-vinyl-acetate (EVA) Polyvinyl chloride (PVC) Ethylene propylene (EPDM) Polyethylene / Polypropylene Ethylene Acrylate Copolymer
5. Anti-stripping agents	Amines, Lime
6. Hydrocarbons (Natural Asphalts)	Gilsonite Trinidad Lake Asphalt Recycling and rejuvenating oils
7. Antioxidants	Lead compounds, Carbon, and Calcium salt
8. Oxidants	Manganese salts
9. Miscellaneous	Deicing calcium chloride, Silicones

2.2.2 Mineral Fillers

Numerous researchers have investigated the use of mineral filler as an additive in HMA. Some of these materials used as additives in the paving industry include dust from crushing and screening of aggregates, lime, Portland cement, carbon black, and fly ash. Researchers found [7,8] that the use of these substances, especially carbon black, is beneficial to the durability, the wear resistance, and the temperature susceptibility of the mix. Mineral fillers may also be used to fill voids thereby preventing the reduction in the asphalt cement content of a mix, increase the stability and apparent viscosity of the mix, improve the bonding between asphalt cement and aggregate, and also used to meet aggregate gradation specifications [2,4].

Because of the very fine and sub micron-sized particles Carbon black is usually combined with boiling point maltene oil (approximately 8% by weight) to form palletized substances known as Microfil 8. Yao and Monismith [7] reported that the addition of 15-20% by weight of Microfil 8 improved the fatigue life, resilient modulus, and resistance to rutting of asphalt mixes. Button et al. [9] also reported that the addition of 15% Microfil 8 in AC-5 asphalt significantly increased the resistance to permanent deformation as compared to that of straight AC-20.

2.2.3 Polymers

The term "polymer" does not necessarily refer to a synthetic material. A polymer is a combination of a large number (*poly*) of similar small molecules (*meros*) or "monomers" into large molecules or "polymers". The properties of the resultant polymer depend on the sequence and chemical structure of the constituent monomers. Polymeric materials can be engineered to have peculiar physical and chemical properties depending on the initial properties of the constituent monomers.

There are a large number of naturally occurring polymers; these can be organic or mineral substances such as hair, rubber, diamonds and sulphur. Even bitumen could fall under the polymer category because of the long-chain nature of some of the organic molecules that are the constituent parts of bitumen.

Polymers can be classified in many ways – based on origin, structure, chain, thermal properties, and deformation properties – however, in asphalt research the focus falls on the thermal and deformation properties. A polymer, according to its thermal property, can be either a “thermoplastic” or a “thermoset”. Thermoplastics, when reacted with appropriate ingredients can usually withstand several heating and cooling cycles without suffering structural breakdown. When heated, a thermoset undergoes a chemical composition change to produce a cross-linked solid polymer [10].

Depending on their deformation property, polymers can be either “elastomers” or “plastomers”. Elastomers can exhibit high extensibility (up to 1000%) from which they recover rapidly upon removal of the stress. Plastomers, which exhibit plastic behavior at in-service temperatures, will deform but will not return to their original dimensions when the load is released [10].

A polymer usually influences the asphalt binder characteristics by dissolving into certain component fractions of the bitumen itself. The spread of the long chain polymer molecules creates an inter-connecting matrix of the polymer through the bitumen. It is this matrix of the long chain molecules of the added polymer that modifies the physical properties of the asphalt binder.

2.2.3-1 Thermosets – Ground Tire Rubber

Ground Tire Rubber (GTR) is a polymer that according to its thermal and deformation properties is categorized as a thermoset elastomer. Crumb rubber, produced from GTR, is a modifier that has gotten attention in recent years partly because of the need for a solution to the increasing number of discarded-tire piles (285 million tires are discarded every year in the US [2]). Several states, including Florida, have enacted legislation to address the issue of tire recycling. In 1988, the Florida Senate Bill 1192 directed the Florida Department of Transportation (FDOT) to evaluate the potential use of reclaimed tire rubber in the construction of asphalt pavements. In 2003, a record number (80%) of the old tires were recycled in the United States for other uses including HMA, fuel, and playground equipment [11].

GTR has been blended with asphalt in various types of pavement construction such as seal coats, inter-layer, and open-graded friction courses [12,13]. When GTR is mixed with asphalt binder (135 to 200°C) the rubber particles swell to at least twice their original volume – due to chemical and physical interactions between the rubber and asphalt particles – causing a significant increase in the viscosity of the asphalt-rubber mixture. The resulting modified binder was reported to have lower temperature susceptibility, increased resistance to plastic deformation at high service temperatures, and improved resistance to age hardening [14,15]. Even though there are potential benefits in the use of crumb rubber, one major issue of rubber-modified binders is its questionable suitability as RAP [2] and relatively higher initial cost.

2.2.3-2 Thermoplastics – SBS

Thermoplastic materials are solids with significant elasticity at room temperature and turns into viscous fluid materials at higher temperatures. When cooled these substances regain their original or rubber-like nature [16]. Thermoplastic elastomers are generally block copolymers of the $(SB)_nX$ type, where “S” represents the polystyrene block, “B” the polybutadiene block, and “X” the coupling agent.

Styrene-butadiene-styrene (SBS) is reported to substantially increase the strength of the mix at high service temperatures [17]. The modifier forms a lattice in the binder, which provides the desired properties of elasticity, plasticity, and elongation. Therefore SBS-modified asphalts tend to improve the adhesive property of a mix, fatigue resistance, rutting resistance, low temperature flexibility and resistance to bleeding. Collins and Mikols [18] found that addition of SBS polymers to asphalt binders can reduce penetration, increase ring and ball softening points, improve low temperature ductility, increase toughness and tenacity, and increase the viscosity at service temperatures.

2.3 Reported Benefits and Problems with Polymers

This section will outline some of the benefits and problems that can be encountered with the use of polymer modified asphalts as reported by various researchers. Almost all modified bituminous materials, however, are proprietary materials which hamper the researcher's ability to determining the benefits of different materials, and to directly compare one material with another. In addition, there is limited published information related to actual field-performance comparisons between modified and non-modified asphalt pavements.

2.3.1 Reported Benefits

The benefits of modified asphalt cement can be realized by a careful selection of the binder additive, since certain modifiers are appropriate for specific applications. In general, asphalt cement should be modified to achieve the following types of improvements [2]:

- Higher stiffness at high service temperatures to prevent rutting and shoving
- Lower stiffness and enhance relaxation properties at low service temperatures to improve cracking performance
- Increase adhesion between asphalt binder and aggregate in the presence of moisture to reduce the possibility of stripping

In 1994, the Maryland State Highway Administration (MDSHA) initiated a competition between the HMA industry and the Portland Cement Concrete (PCC) industry for the reconstruction of two adjacent intersections that repeatedly experienced rutting – U.S. Rt. 40 at MD Rt. 213 and U.S. 40 at Landing Lane. The HMA industry removed all eight-inches of the failed pavement, and repaved the intersection with a six-inch Superpave base layer and a two-inch Superpave surface layer, both mixed with SBS modified PG 76-22 asphalt. The PCC industry replaced the top 6-¼ inches of the original failed HMA layer with PCC whitetopping. By late 1999, the PCC showed cracking and open joints, and was ultimately removed and replaced with HMA. At the time of the removal of the PCC intersection in 2000, the modified HMA section has rutted less than 1/16 of an inch and was still in excellent condition [19].

In Kentucky, the intersection of U.S. 27 and KY 80 was reconstructed in 1998 due to poor rutting performance. The Plantmix Asphalt Industry of Kentucky used the Superpave process for material characterization and mix design, with one hundred percent crushed aggregate and PG 76-22 modified binder. The gradations of the base and wearing courses were both essentially Superpave gradations, but were slightly coarser at the bottom control points. The modified binder appears to have solved the rutting problem, and the intersection is performing well [20].

The South Carolina Department of Transportation (SCDOT) used the Asphalt Pavement Analyzer (APA) to evaluate the relative performance of asphalt mixtures with different binders – PG 76-22 polymer modified and PG 64-22 [21]. The polymer-modified binder mix exhibited 56 percent less rutting than the non-modified (measured on 12.5mm Superpave samples). This was an indication that the addition of polymer-modified binder would enhance the rutting resistance of asphalt pavements.

In a side by side experimental study on I-55, near Grenade in northern Mississippi, Uddin and Nanagiri [22] compared the performance of a neat AC 30 asphalt binder to eight modified binders. All nine mixes are designed with the Marshall method, since at the time of construction (1996) the Mississippi Department of Transportation (MDOT) was still using the Marshall Mix design method. The sections were constructed in two 1½-inch thick lifts of binder and surface course. Three years after construction (November 1999), a survey measured 0.3 inch rut depth at the control section (neat AC 30); whereas the SBS-modified and the GTR-modified outperformed the control section with 0.05 and 0.04 inch rut depth respectively.

Furthermore, MDOT now requires polymer modified asphalt (PMA) for the top two courses of any pavement designed for 3 million or more equivalent single-axle loads (ESAL) [23]. Suppliers are required to start with a PG 67-22 binder, then add polymer to improve its performance grade to PG 76-22. Cost analysis for Mississippi showed that the PMA adds three to five dollars per ton of mix, which increases the cost to \$38 to \$42 per ton of mix paved.

Stuart and Mogawer [24] studied the effect of eleven different asphalt binders – eight polymer-modified, one air blown, and two unmodified – to the rutting performance of asphalt mixtures. Results from the Superpave Shear Tester (SST), the French Pavement Rutting Tester (PRT), and the Hamburg Wheel-Tracking Device (WTD), ranked the unmodified asphalt mixtures last based on rutting performance.

2.3.2 Known Issues with Polymer Modified Asphalts

The possible problems with modified binders are mainly in the storage of the liquid asphalt, mixing temperatures, and the length of time the material is held at elevated temperatures before placing. The blending of asphalt and polymer, other than being proprietary information, is not an easy process, so modified binder is usually purchased in a ready blended form from the supplier. Therefore, the asphalt plant has to purchase PMA in large quantities, usually a 20 ton tanker, which would produce approximately 250 tons of asphalt concrete. This means small tonnages of most modified bituminous materials are not financially feasible [25].

It is usually necessary for the modified binder to be held in a tank that is capable of being agitated in some way, as the polymers being of a different density to the bitumen tend to separate if kept in storage for prolonged periods. The polymer additive can be destroyed by the temperature being too high during mixing, or by being held at elevated temperature for a long period of time after mixing. Even the binder storage times should be kept as short as possible to prevent deterioration of the polymer [23].

2.4 Interviews

In-person interviews were held with representatives from: (1) the paving industry – Mr. Ken Murphy of Anderson Columbia Co, Inc, (2) the FDOT – Mr. Steve Sedwick of District 2, and (3) the asphalt industry – Mr. Frank Fee of CITGO Asphalt. The first two interviews were held at the offices of Anderson Columbia Co, Inc. and the FDOT offices in Lake City the first week of May 2004. Mr. Frank Fee was able to come to our offices at UF for an interview on May 6th 2004.

The discussion during the interviews was based on the following questioner to ensure that people from the various industries will comment on similar aspects of the use of modifiers in Florida:

1. How and why did you decide to use modified asphalt?
2. How did you select the type of modifier?
3. How did you design the modification of the binder (modifier content)?
4. What kind of effect on HMA performance did you expect?
5. What is the actual verified effect on HMA performance?
6. Describe the impact of using modified asphalt on production.
7. What is your opinion about the level of usage of asphalt modifiers in Florida?
8. Comment about the economical benefit of asphalt modification.
9. Based on your personal experience, make recommendations about design, handling, placement and compaction of modified HMA.

Overall, the three people formally interviewed, along with other DOT personnel and University faculty that discussions were not recorded, supported the use of modified asphalts, particularly SBS-modified. Mr. Ken Murphy mentioned that their firm (Anderson Columbia Co, Inc.) has had three major projects dealing with SBS modified binder – I-10 Washington County, I-75 Columbia County, and I-95 St John County. The decision for polymer modified asphalt was the responsibility of the FDOT, to improve rutting and cracking performance. Anderson Columbia Co, Inc. reported that the SBS modified binder was easier to place and compact due to the absence of a ‘tender zone’. According to Mr. Murphy, the ease of placement of the mixture increased the asphalt plant production by 30%. However, in the case of absorbent aggregate (i.e. limerock), the mix production slows down approximately by 50% due to the extra time it takes to dry the aggregate. Based on his experience, Mr. Murphy suggests that SBS polymer modified binder does not allow for any moisture to escape once the aggregate has been coated with asphalt.

Mr. Murphy went on to say that is still early to talk about performance enhancements and actual field results; a view shared by most of the people we interviewed. For the price premium of the modified binders, Mr. Murphy said the cost increase amounts to approximately $100 per ton of liquid binder; an acceptable increase in price compared to the cost of the entire project. Mr. Murphy described the use of modifiers in Florida as “cautious” and “slow”, and reiterated that he would like to see an increase in the use of polymer-modified mixtures. Closing, he said that prices of the finished modified mix will decrease when the asphalt producers and plant owners are assured that there will be an increase in demand from the state.

Mr. Steve Sedwick, District 2 FDOT, mentioned that there are a few small and relatively young projects scattered around the district, a fact that makes assessing the performance of these sections difficult. However, Mr. Sedwick said that use of polymer modified asphalts, specifically SBS modified, is suggested wherever rutting-problems occurred. The price increase for the SBS-modified mixtures is approximately $6 per ton of mix, which compared to the performance benefits appears to be a welcome premium. Closing, Mr. Sedwick said that Florida need to embrace new technology (i.e. PMA) for pavement performance improvement.

Mr. Frank Fee, representing CITGO Asphalt, shared his experience and opinions about polymer modified asphalts. Mr. Fee has been involved with PMA since the early 80s, investigating the benefits of Ethylene Vinyl Acetate (EVA) modifiers to prevent draindown. In the late 80s, when the SBS-type modifiers were introduced from Europe, Mr. Fee was involved in the I-80 experiment where the Pennsylvania DOT built several sections of modified binders and monitored their performance. Based on personal experience, Mr. Fee is a strong supporter of SBS-modified binders and would also like a commitment for higher-volume usage of PMA from the state. The increase in usage, Mr. Fee continued, will allow the asphalt and mix producers to offer lower prices.

2.5 Summary

The discussion presented in this chapter outlined the main types of asphalt modifiers and described the ones that are most commonly used in Florida – GTR and SBS. The field evaluation work in the literature focused on rutting performance enhancements, which clearly showed the benefits of asphalt concrete with the addition of modifiers. No field experiments reporting on top-down cracking were found in the literature; however, in later chapters we will examine the laboratory-verified benefits of PMA.

CHAPTER 3
PERFORMANCE OF MODIFIED ASPHALT MIXTURES IN FLORIDA

3.1 Overview

This chapter will provide a summary of the Florida experiences with polymer modified asphalts – GTR modified and SBS modified – including laboratory evaluation of the modified mixture properties, HVS experiments and interviews and reports for field performance.

3.2 Heavy Vehicle Simulator Experiment

The FDOT Materials Office recently acquired a Heavy Vehicle Simulator (HVS), Mark IV Model that can simulate 20 years of interstate traffic on a test pavement within a short period of time. Shortly after, the University of Florida in cooperation with FDOT initiated a study to evaluate the long-term performance of Superpave mixtures and SBS-modified Superpave mixtures with particular emphasis on rutting resistance.

The main objectives of this study, as it relates to PMA performance, were to evaluate the rutting performance of a typical Superpave mixture used in Florida with SBS modified and non-modified binders, and to evaluate the difference in rutting performance of a pavement using two lifts of modified mixture versus a pavement using only the top lift of SBS-modified mixture.

3.2.1 Experiment Setup

Figure 3.1 shows the layout of the HVS test track at the FDOT Research Park facility. The test track consisted of seven test lanes 12 to 13.5-feet wide by 30 feel long and were divided into three test sections – identified as Sections A, B, and C. Adjacent to the test lanes was a 94-feet long area that was used for maneuvering the HVS.

The test track had a 10.5-inch limerock base placed on top of a 12-inch limerock stabilized subgrade. Lanes 1 and 2 were paved with two 2-inch lifts of the SBS-modified Superpave mixture. Lane 3 had a 2-inch lift of the modified Superpave mix over a 2-inch lift of unmodified Superpave mix. Lanes 4 through 7 were paved with two 2-inch lifts of the unmodified Superpave mix. Sections C in Lane 1 through 5 were assigned to Phase I, whereas Sections A and B in Lane 1 through 5 were assigned to Phase II of the experiment.

Phase I was conducted at ambient condition on five test sections – 1C through 5C – and Phase II was conducted with temperature control on the remaining ten test sections. In Phase II, Lanes 1 and 2, which have two 2-inch lifts of SBS-modified Superpave mixture were tested at controlled pavement temperatures of 50° C and 65° C; the rest of the test sections in Phase II were tested at 50° C. The testing sequence was arranged such that the effects of time on each lane could be averaged out. In order to minimize damage, the test sequence was arranged in a manner that the HVS would not have to drive over a section that had not yet been tested.

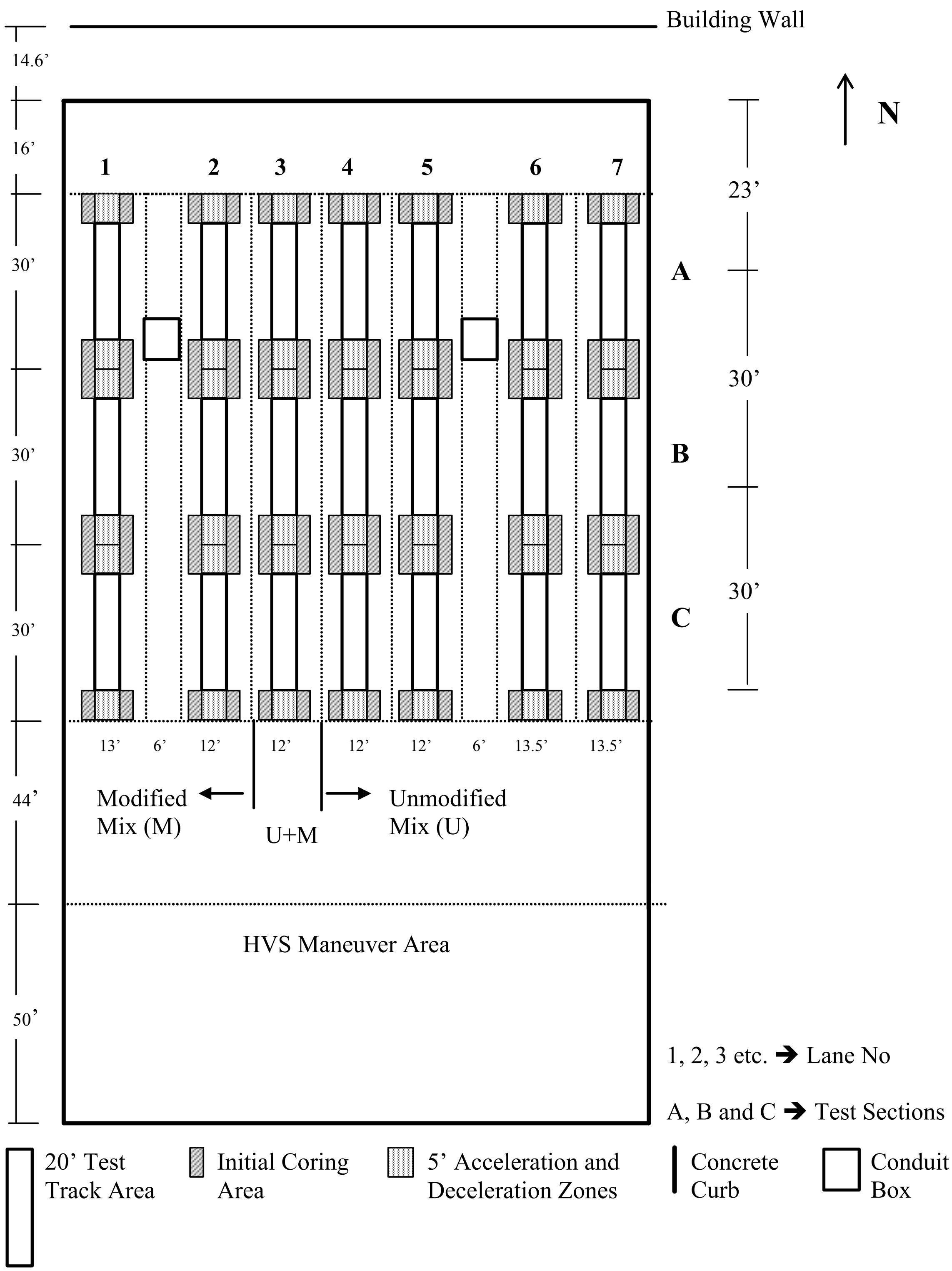


Figure 3-1. Test track layout.

3.2.2 Materials and Testing Procedures

The two asphalt mixtures placed in the test pavements were mixed with the same aggregate blend (gradation) and same effective asphalt content with two different binders – one PG67-22 and an SBS-modified PG76-22. The mixtures were 12.5 mm fine Superpave mixes, with 12.5 mm nominal maximum aggregate size and the gradation plotted above the restricted zone (obsolete). Table 3.1 shows the aggregate properties for both mixtures.

Both mixtures were designed at the Research Park facility of the FDOT State Materials Office. The optimum binder content was determined according to the Superpave mix design procedure for a design traffic level of 10 to 30 million ESAL. Table 3.2 contains the volumetric information and binder content for both mixtures.

In order to determine the optimum HVS test configuration, five trial tests were run on Lane 7. The trial runs used a super-single (wide-based radial) tire with 9,000-lbs load at 115-psi inflation pressure and a traveling speed of 8 mph. The different combinations of wheel traveling configurations (uni-directional or bi-directional), total wheel wander and wander increments are listed below:

10. Bi-directional travel with no wander
11. Uni-directional travel with no wander
12. Uni-directional travel with 4-inch wander in 2-inch increments
13. Bi-directional travel with 4-inch wander in 2-inch increments
14. Uni-directional travel with 4-inch wander in 1-inch increments

The most suitable configuration for the experiment was the uni-directional loading with four-inch wander in one-inch increments. This configuration produced wheel track profiles without the imprint of the tire treads, that were more representative of the ones observed in the field.

Table 3-1. Aggregate properties for the test mixtures.

Type Material	FDOT Code	Producer	Pit No	Date Sampled
1. S-1-A Stone	41	Rinker Mat. Corp	TM-489 87-089	9/11/00
2. S-1-B Stone	51	Rinker Mat. Corp	TM-489 87-089	9/11/00
3. Screenings	20	Anderson Mining Corp	29-361	9/11/00
4. Local Sand		V.E.Whitehurst & Sons, Inc	Starvation Hill	9/11/00

Percentage by Weight of Total Aggregate Passing Sieves								
Blend		12%	25%	48%	15%	JMF	Control Points	Restricted Zone
Number		1	2	3	4			
Sieve Size	¾" 19.0mm	99	100	100	100	100	100	
	½" 12.5mm	45	100	100	100	93	90-100	
	3/8" 9.5mm	13	99	100	100	89	-90	
	No. 4 4.75mm	5	49	90	100	71		
	No. 8 2.36mm	4	10	72	100	53	28-58	39.1-39.1
	No. 16 1.18mm	4	4	54	100	42		25.6-31.6
	No. 30 600μm	4	3	41	96	35		19.1-23.1
	No. 50 300μm	4	3	28	52	22		
	No. 100 150μm	3	2	14	10	9		
	No. 200 75μm	2.7	1.9	5.9	2.2	4.5	2-10	
G_{sb}		2.327	2.337	2.299	2.546	2.346		

Table 3-2. Volumetric properties for the test mixtures.

Mix Type	Asphalt Binder	% Binder	V_a @ N_{des}	VMA	VFA	P_{be}	G_{mm}
Superpave Mix (Compacted at 300° F)	PG67-22	8.2	4.0	14.5	72	4.97	2.276
Modified Superpave Mix (Compacted at 325° F)	PG76-22	7.9	3.8	14.2	73	4.90	2.273

3.2.3 HVS Findings

A laser profiler was used to record the pavement surface profiles of the test pavements before, during and after the HVS testing. Mr. Tom Byron of FDOT performed the analysis of the profiler data using two methods. In the first method, the initial transverse surface profile (before test) was subtracted from the final transverse surface profile to obtain the "differential surface profile." The rut depth is measured as the distance between a line connecting the two highest points of the "differential surface profile" and the lowest point of the "differential surface profile." Figure 3-2 shows the plots of change in rut depth as determined with this method.

In the second measurement method, the rut depth is calculated as the distance between a line connecting the two peaks of the final measured surface profile and the lowest point on the profile. Figure 3-3 shows the plots of change in rut depth as determined by this method.

The following observations are made from the rutting results based on Figures 3-2 and 3-3:

- Good repeatability of test results was generally observed between different test sections with the same pavement design and test temperature. Lanes 4 and 5 (two lifts of unmodified mixture) appeared to have relatively higher variability in rut development than the other test sections.
- The pavement sections with two lifts of SBS-modified mixture clearly outperformed those with two lifts of unmodified mixture. Sections with two lifts of unmodified mixture tested at 50° C – Sections 4A , 4B, 5A and 5B – experienced two to two and a half times the rut rate compared to sections two lifts of modified mixture tested at the same temperature – Sections 1B and 2B.
- The pavement sections with a lift of SBS-modified mixture over a lift of unmodified mixture (Sections 3A and 3B) had statistically the same rut rate as those with two lifts of modified mixture (1B and 2B) when tested at 50° C.

- Sections with two lifts of SBS-modified mixture – Sections 1A and 2A – tested at 65° C had much lower rutting than the test sections with the unmodified mixture and tested at 50° C – Sections 4A, 4B, 5A and 5B.

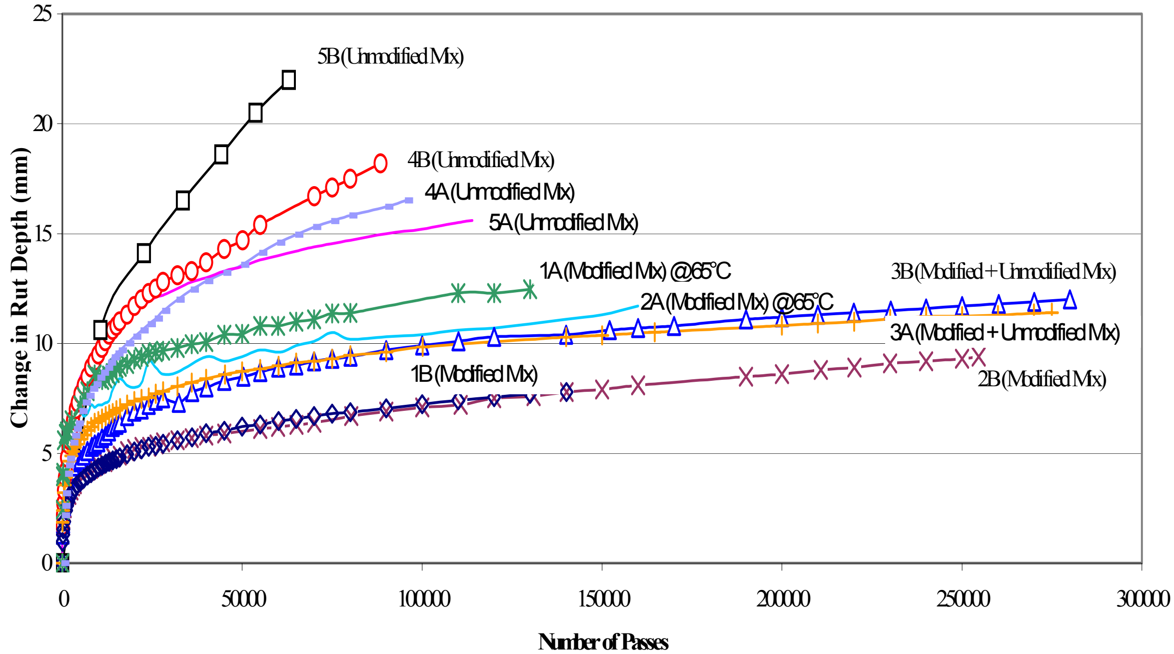


Figure 3-2. Comparison of rut depth as measured by the differential surface profile method versus number of passes.

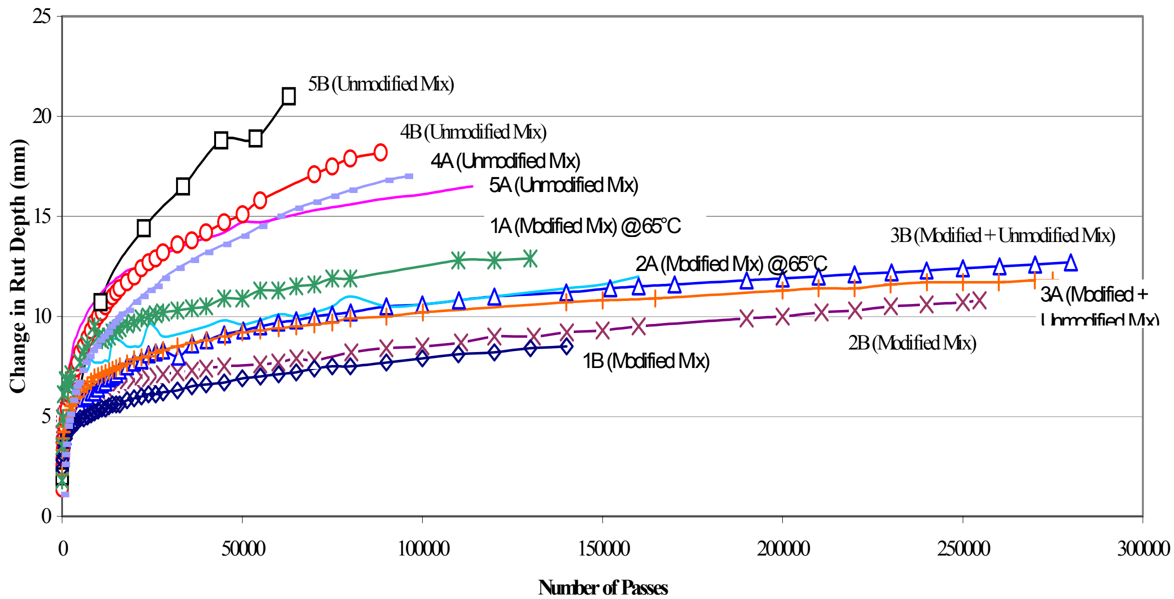


Figure 3-3. Comparison of change in rut depth as measured by the surface profile method versus number of passes.

3.3 Laboratory Testing for the HVS Experiment

The asphalt mixtures used in the HVS experiment were also evaluated in the laboratory. Mixtures were sampled from the hot-mix plant during construction of the test tracks and were subjected to Gyratory Testing Machine (GTM), and Asphalt Pavement Analyzer (APA) testing to assess their rutting potential and determine any possible relationship between mixture properties and field performance.

After the HVS experiment, cores were extracted from the test sections to evaluate the changes in properties of the pavement materials, and the possible relationship between the laboratory-measured mixture properties and the observed rutting performance. Four six-inch cores were taken for each of the test sections – two from the middle of the wheel path and two from the edge of the wheel path. The cores were further separated into Lift 1 and Lift 2 to isolate the properties of each layer and tested with the Superpave Indirect Tension Test (IDT).

3.3.1 GTM Results

Three samples from each lift of the unmodified and the SBS-modified mixtures were compacted to ultimate density (when the change in density is equal to or less than 0.5 lb/ft3 per 50 revolutions) under a 120-psi vertical ram pressure in the GTM. The SBS-modified and unmodified mixture samples were compacted at 325° F and 300° F, respectively, to simulate the actual placement temperatures. The gyratory shear resistance (Sg) of the mixture was determined at every 10 revolutions until 50 gyrations, and every 25 revolutions henceforth. The Gyratory Stability Index (GSI) – the ratio of the maximum gyratory angle to the minimum gyratory angle – was determined at the end of the test.

Table 3-3 shows the GSI value of each specimen as calculated from the GTM gyrograph. The GSI values of the SBS-modified mixtures were 1.02 for Lift 1 and 1.04 for Lift 2, whereas the unmodified mixtures had GSI values of 1.18 and 1.21 for Lift 1 and Lift 2, respectively. An increase in the GSI value beyond 1.0 usually indicates instability of the mixture under the applied ram pressure. Therefore, this indicates the unmodified mixture (with a GSI of more than 1.0) was relatively less stable compare to the SBS-modified mixture (with a GSI close to 1.0).

Table 3-3. GSI values of the four mixtures evaluated in the GTM

Sample No	**Unmodified Mix**	**Unmodified Mix**	**Modified Mix**	**Modified Mix**
	Lift 1	**Lift 2**	**Lift 1**	**Lift 2**
1	1.15	1.20	1.00	1.00
2	1.23	1.19	1.05	1.00
3	1.17	1.23	1.00	1.12
Average	**1.18**	**1.21**	**1.02**	**1.04**

3.3.2 Asphalt Pavement Analyzer Results

The APA was used to evaluate the rutting performance of the modified and unmodified asphalt mixtures in the laboratory. The APA has the ability to test six gyratory samples simultaneously in an environmentally controlled chamber. Load is applied onto a pressurized linear hose by a pneumatic loaded wheel and tracked back and forth over a testing sample to induce rutting.

Cylindrical specimens were compacted to 7% ± 0.5% air voids with the Superpave Gyratory Compactor and subjected to 8000 load cycles. Performance was calculated by subtracting the rut depth after 8025 wheel passes by the rut depth after 25 wheel passes. Six specimens of the unmodified mixture-lift 1 and four specimens of each of the remaining mixtures were evaluated in the APA. Rut-depth results, Table 3-4, indicate that the SBS-modified mixture outperformed the unmodified mixture with 30% less rutting.

Table 3-4. Rut-depth measurements in the APA evaluation of the four mixtures.

Sample No	Measurement No	**Unmodified Mix-Lift 1**			**Unmodified Mix-Lift 2**		
		Rut Measurement			Rut Measurement		
		25 Passes	8025 Passes	**Rut Depth**	25 Passes	8025 Passes	**Rut Depth**
1	1	20.2	11.8	8.4	19.8	12.6	7.2
	2	20.6	11.1	9.5	20.3	11.9	8.4
2	1	20.8	10.8	10.0	20.6	12.6	8.0
	2	20.6	11.3	9.3	20.1	13.1	7.0
3	1	20.5	9.4	11.1	20.3	13.0	7.3
	2	20.7	9.6	11.1	20.4	12.6	7.8
4	1	20.8	10.4	10.4	20.4	13.4	7.0
	2	20.0	11.0	9.0	18.5	14.5	4.0*
5	1	20.8	11.1	9.7			
	2	20.4	9.8	10.6			
6	1	20.8	10.6	10.2			
	2	21.0	12.0	9.0			
Overall Average (mm)				**9.9**			**7.5**

Sample No	Measurement No	**Modified Mix-Lift 1**			**Modified Mix-Lift 2**		
		Rut Measurement			Rut Measurement		
		25 Passes	8025 Passes	**Rut Depth**	25 Passes	8025 Passes	**Rut Depth**
1	1	20.6	14.4	6.2	21.0	16.1	4.9
	2	20.8	14.5	6.3	21.0	15.8	5.2
2	1	20.7	14.4	6.3	21.2	16.4	4.8
	2	20.9	14.8	6.1	21.0	15.6	5.4
3	1	20.5	15.4	5.1	21.1	16.0	5.1
	2	21.1	14.8	6.3	21.2	15.2	6.0
4	1	21.3	14.3	7.0	21.3	15.7	5.6
	2	20.9	14.8	6.1	21.1	15.6	5.5
Overall Average (mm)				**6.2**			**5.3**

* Not considered in the overall average because the value is an outlier

3.3.3 Superpave Indirect Tension Test

The cores from the HVS Experiment test sections contained two 2-inch HMA layers, which were separated (with a wet saw) into two lifts – bottom layer (Lift 1) and top layer (Lift 2). The sliced specimens were tested for resilient modulus at 5 and 25° C and indirect tensile strength at 25° C. The test protocol calls for three replicates for each test, however, only two replicates were available for these mixtures.

Table 3-5 shows the resilient modulus and indirect tensile strength results at 25° C. The resilient modulus of the SBS-modified mixture does not appear to be significantly different from the unmodified mixture; however, the average indirect tensile strength at 25° C of the SBS-modified mixture was higher than the unmodified mixture by approximately 10%.

Based on the density and thickness of the test-section cores, rutting in the unmodified asphalt mixture sections appears to be due to a combination of densification and shear movement, while rutting of the pavement sections with the SBS-modified mixture appears to be primarily due to densification.

Table 3-5. Resilient Modulus and Indirect Tensile Strength at 25° C.

Section	Location		Resilient Modulus		Tensile Strength	
			Gpa	Psi (10^6)	Mpa	Psi
7AW	**Lift 2**	Wheelpath	3.16	0.46	0.68	98.6
		Edge	2.84	0.41	0.67	97.1
	Lift 1	Wheelpath	3.40	0.49	0.75	108.7
		Edge	4.13	0.60	0.85	123.2
7AE	**Lift 2**	Wheelpath	3.08	0.45	0.82	118.8
		Edge	3.04	0.44	0.52	75.4
	Lift 1	Wheelpath	3.75	0.54	0.72	104.3
		Edge	3.33	0.48	0.75	108.7
7BW	**Lift 2**	Wheelpath	2.47	0.36	0.62	89.9
		Edge	2.34	0.34	0.52	75.4
	Lift 1	Wheelpath	2.93	0.42	0.68	98.6
		Edge	2.89	0.42	0.68	98.6
7BE	**Lift 2**	Wheelpath	2.91	0.42	0.60	87.0
		Edge	1.97	0.29	0.53	76.8

Section	Location		Resilient Modulus		Tensile Strength	
			Gpa	Psi (10^6)	Gpa	Psi (10^6)
	Lift 1	Wheelpath	3.17	0.46	0.67	97.1
		Edge	2.88	0.42	0.68	98.6
7C	**Lift 2**	Wheelpath	3.59	0.52	0.78	113.0
		Edge	2.89	0.42	0.63	91.3
	Lift 1	Wheelpath	4.11	0.60	0.91	131.9
		Edge	3.97	0.58	0.89	129.0
2C	**Lift 2**	Wheelpath	4.27	0.62	0.91	131.9
		Edge	2.05	0.30	0.60	87.0
	Lift 1	Wheelpath	4.22	0.61	0.89	129.0
		Edge	3.90	0.57	0.86	124.6
3C	**Lift 2**	Wheelpath	3.12	0.45	0.57	82.6
		Edge	2.38	0.34	0.70	101.4
	Lift 1	Wheelpath	3.97	0.58	0.71	102.9
		Edge	3.81	0.55	0.72	104.3
4C	**Lift 2**	Wheelpath	3.28	0.48	0.76	110.1
		Edge	1.69	0.24	0.42	60.9
	Lift 1	Wheelpath	3.33	0.48	0.76	110.1
		Edge	2.34	0.34	0.64	92.8
5C	**Lift 2**	Wheelpath	4.38	0.63	0.77	111.6
		Edge	2.75	0.40	0.55	79.7
	Lift 1	Wheelpath	3.63	0.53	0.82	118.8
		Edge	2.50	0.36	0.66	95.7
2B	**Lift 2**	Wheelpath	4.92	0.71	0.93	134.8
		Edge	2.76	0.40	0.73	105.8
	Lift 1	Wheelpath	5.57	0.81	1.04	150.7
		Edge	3.58	0.52	1.00	144.9
3B	**Lift 2**	Wheelpath	4.88	0.71	0.91	131.9
		Edge	2.85	0.41	0.78	113.0
	Lift 1	Wheelpath	5.65	0.82	1.05	152.2
		Edge	4.10	0.59	0.83	120.3
4B	**Lift 2**	Wheelpath	4.97	0.72	0.92	133.3
		Edge	2.59	0.38	0.66	95.7
	Lift 1	Wheelpath	5.60	0.81	1.01	146.4
		Edge	3.59	0.52	0.77	111.6
5B	**Lift 2**	Wheelpath	4.11	0.60	0.88	127.5
		Edge	2.41	0.35	0.60	87.0
	Lift 1	Wheelpath	5.55	0.80	1.01	146.4
		Edge	4.02	0.58	0.91	131.9
3A	**Lift 2**	Wheelpath	4.25	0.62	0.81	117.4
		Edge	2.51	0.36	0.61	88.4
	Lift 1	Wheelpath	4.91	0.71	0.87	126.1
		Edge	3.46	0.50	0.66	95.7

3.4 Laboratory Evaluation of GTR-Modified Dense-Graded Mixtures

Experience with Superpave indicated that mixtures produced with conventional asphalt binders, particularly those intended for use on high traffic volume facilities, may not have adequate resistance to cracking as a result of lower design asphalt content. The above observation is a direct result of the Superpave design procedure since higher number of gyrations (to simulate higher traffic volume) amount to lower design asphalt content.

Furthermore, recent work (fracture resistance evaluation of asphalt mixtures) showed that coarse-graded Superpave mixtures can be difficult to compact and may result in pavements with relatively high permeability. The combination of high permeability and low asphalt content may also have an adverse effect on fracture resistance.

The above observations suggest that Superpave mixtures with conventional asphalt cement may not be able to provide adequate rutting and cracking resistance when designed for higher traffic volume. This study examined the use of GTR to improve dense graded mixture performance with the following objectives:

1. To evaluate the effects of Ground Tire Rubber (GTR) modifier on the rutting and cracking performance of Superpave mixtures
2. To provide test data on mixture parameters that could provide some insight into the behavior of HMA mixes that could lead to better understanding of the field performance of asphalt pavements

3.4.1 Materials and Methods

The mixtures studied in this project used two aggregate sources – White Rock (crushed limestone) and Cabbage Grove. According to Kestory [26], cabbage grove aggregate exhibited high L.A Abrasion and further tests showed that the aggregate breaks down during mix compaction. Therefore, cabbage grove was used as part of the study to determine whether the rubber improves toughness or reduces aggregate- breakdown during compaction by volumetrically replacing the fine part of white rock (passing sieve #8 to #200) with the fines in cabbage grove.

The modified binder was produced by blending #80 GTR (crumb rubber passing sieve #80) by Rouse Rubber Industries, Inc, with AC-30 asphalt cement for approximately 20 minutes at 300°F with a Silverson L4R High Shear Mixer. Otoo (2000) [27] suggested that the optimum percentage (by weight of the straight asphalt binder) of rubber content for rubber-modified binders is 10-15%. Furthermore, FDOT currently uses 12% rubber–modified binders on its OGFC field sections constructed with modified binders. Hence, for this study the modified binders were produced at 12% by weight of the straight asphalt.

Three aggregate blends – coarse limerock, fine limerock and limerock with cabbage grove – were mixed with two asphalt binders – straight AC-30, and GTR-modified. Each mixture was designed for three different Superpave traffic levels – 3, 4, and 5 – resulting in 18 mixtures (3 gradations * 2 binders * 3 traffic levels). Table 3-6 presents the mixtures produced at the various traffic levels and their design asphalt contents.

The mixtures were then evaluated for their relative performance; the Servopac Gyratory Compactor (at 2.5° gyratory angle) was used to evaluate the rutting susceptibility and the Indirect Tension Test to evaluate the cracking resistance.

Table 3-6. HMA mixtures and designations.

Aggregate Type		Traffic Level	Design AC% (Binder) content	Designation	
				Straight AC	Modified AC (12% Rubber)
White Rock (Limestone)	Coarse Graded	3	7.2	C3	CR3
		4	6.6	C2	CR2
		5	6.1	C1	CR1
	Fine Graded	3	6.6	F3	FR3
		4	6.4	F2	FR2
		5	6.2	F1	FR1
White Rock & Cabbage Grove Fines	Coarse Graded	3	7.4	CG3	CGR3
		4	6.9	CG2	CGR2
		5	6.4	CG1	CGR1

3.4.2 Servopac Gyratory Compactor

The gyratory shear strength (G_s) is a strength parameter calculated in the Servopac during mixture compaction. Current research at UF indicates that using a 2.5° gyratory angle, the Servopac can be used to evaluate the shear resistance and stability of HMA mixes by monitoring the rate of change of shear strength and the rate of change of air voids of the mix during the compaction process. Higher gyratory angle (2.5°) yields compaction data sensitive enough that provides insight to the relative stability and shear resistance of the mixes.

To evaluate the measured gyratory shear resistance of the mixes, the SGC results were analyzed from three different perspectives outlined below:

1. A direct comparison of the gyratory shear (G_s) measurements
2. Evaluation of the areas under the gyratory shear vs. volumetric strain curve
3. The rate of change of gyratory shear resistance with respect to air voids (dGs/dAV)

3.4.2-1 Direct Comparison of Gyratory Shear Resistance (Gs)

Table 3-7 presents a summary pf the Servopac results for all tests. The rubber-modified mixtures recorded lower G_s values than the unmodified mixtures; however, the rubber-modified mixtures sustained a higher percentage of the maximum gyratory shear after the peak value. The R_{fp} parameter – ratio of the final gyratory to the peak gyratory shear – is generally higher for the modified samples. Comparing the 7.2r% and the 6.1% coarse samples it was noticed that the peak shear strength of the samples were approximately the same throughout the compaction process. This suggests that the rubber enabled the addition of more total binder with no substantial loss in the shear strength of the sample.

After the peak G_s value, the gyratory shear drops as the number of gyrations increase. This post-peak drop ($D_{p\text{-}f}$) is greater for the unmodified mixes than the modified ones. The modified samples exhibited lower percentage drop in the gyratory shear value and also retained a higher G_s value at the end of the compaction process.

3.4.2-2 Areas under the Gyratory Shear vs. Volumetric Strain Curve

Different parts of the area under the gyratory shear strength versus the volumetric strain plot present some empirical parameters that can correlate with the energy required to compact the samples during the various stages of the compaction process. Figure 3-4 is a schematic diagram with details of the four sections of a typical plot of the gyratory stress-volumetric strain curve.

There is an initial steep straight-line portion up to a breaking point, which is most probably due to the initial compression of the mixture till the aggregate particles come into contact with each other. Researchers have hypothesized that beyond this point up to the locking point (LP) aggregate effects take over the characteristics of the compaction

Table 3-7. Summary of Servopak compaction data.

Mixture		%AC	Peak Gyratory Shear, G_{sp}	Shear at final gyration, G_{sf}	Drop in G_{sp}, D_{p-f} (%)	Average D_{p-f}	R_{fp} (%) (G_{sf}/G_{sp})	Average R_{fp}
White Rock	Coarse	7.2	530	502	5.3	6.3	94.7	93.7
		6.6	537	518	3.5		96.5	
		6.1	541	487	10.0		90.0	
		7.2r	509	489	3.9	5.3	96.1	94.7
		6.6r	518	487	6.0		94.0	
		6.1r	526	494	6.1		93.9	
	Fine	6.6	547	492	10.1	8.3	89.9	91.7
		6.4	549	495	9.8		90.2	
		6.2	546	519	4.9		95.1	
		6.6r	535	489	8.6	6.1	91.4	93.6
		6.4r	542	492	9.2		90.8	
		6.2r	533	519	2.6		97.4	
White Rock + Cabbage Grove Fines	Coarse	7.4	543	483	11.0	9.5	89.0	90.5
		6.9	555	493	11.2		88.8	
		6.4	570	535	6.1		93.9	
		7.4r	539	504	6.5	4.8	93.5	95.2
		6.9r	553	524	5.2		94.8	
		6.4r	550	535	2.7		97.3	

process. The area under this portion of the curve represents the energy required to compact the sample up to the locking point. Vavrick et al., 1999 [46] have indicated that the locking point is the “preferred orientation” of the aggregates in terms of the optimum interlocking of the particles. It is assumed that beyond this point further compaction would result in shear failure of the sample.

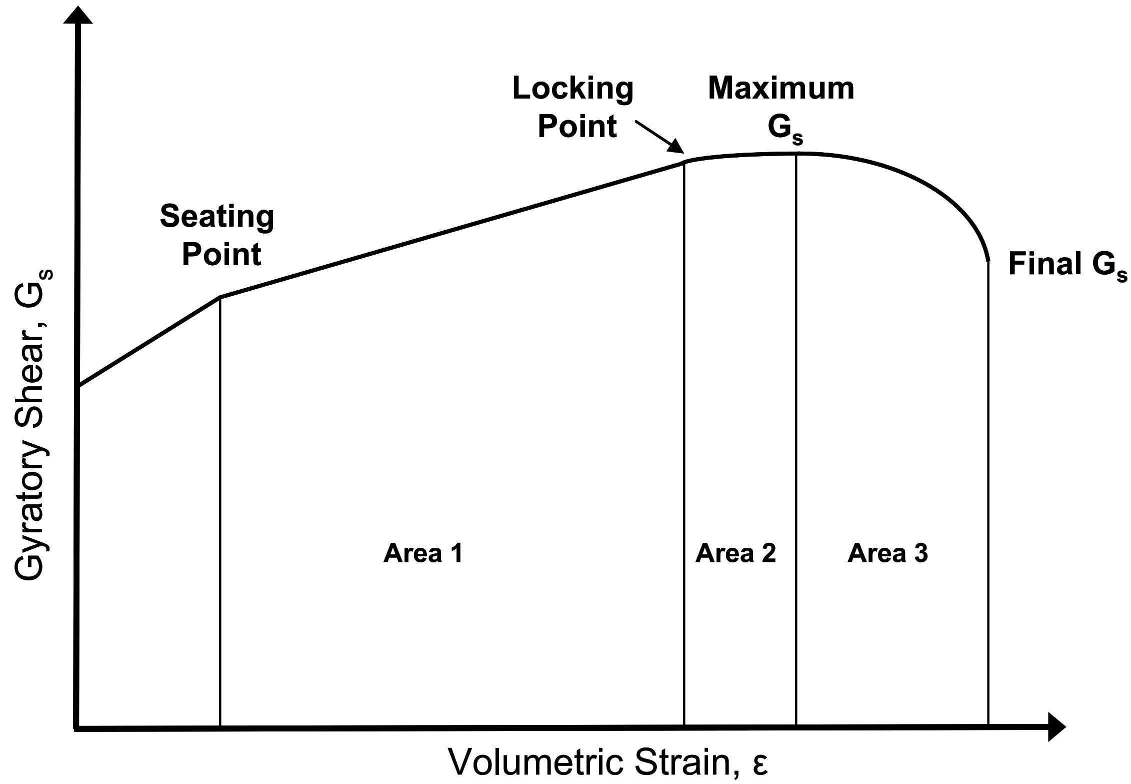


Figure 3-4. Schematic diagram of Gyratory Shear (Gs) versus Volumetric Strain (ε)

The locking point identifies the end of the second straight-line portion of the stress-strain curve. Beyond this point the curve assumes a non-linear relationship up to the peak gyratory shear, partly due to a succession of repetitive shear values indicating a zero or minimal net increase in the resistance of the sample to compaction per unit change in volumetric strain (or air voids). The final section of the curve is the post-peak drop off to the end of the compaction process. The area under this section of the curve seems to be related to the energy required to cause shear failure of the sample. Table 3-8 estimates the areas (as identified in Figure 3-4) under the gyratory shear-volumetric strain curve.

Table 3-8. Summary of estimated area values under the G_s-volumetric strain curve.

Sample		Area 1	Area 2	Area 3	Areas 1+2	Areas 2+3	Total Area
W.R Coarse	**7.2**	6923.26	396.11	1106.13	7319.37	1502.24	8425.50
	6.6	7039.25	817.75	1162.84	7857.00	1980.59	9019.84
	6.1	7119.38	895.24	1324.32	8014.62	2219.56	9338.94
	7.2r	6184.55	107.68	1401.34	6292.23	1509.02	7693.57
	6.6r	6057.65	379.32	1594.29	6436.97	1973.61	8031.26
	6.1r	6586.20	448.21	1480.53	7034.41	1928.74	8514.94
W.R Fine	**6.6**	5724.22	557.14	896.97	6281.36	1454.11	7178.33
	6.4	5486.48	921.25	891.81	6407.73	1813.06	7299.54
	6.2	5137.74	1405.79	735.96	6543.53	2141.75	7279.49
	6.6r	5186.27	484.03	948.20	5670.30	1432.23	6618.50
	6.4r	5211.67	592.66	1247.69	5804.33	1840.35	7052.02
	6.2r	4984.81	631.27	1122.49	5616.08	1753.76	6738.57

Plastic deformation in HMA pavements is attributed primarily to the lateral displacement of materials. A close examination of the areas beneath the shear-strain curve, it appears that Area relates to the ability of the material to resist permanent deformation (or shear failure). Higher area suggests higher energy is required to distort the sample, which indicates a higher (or increased) resistance to plastic deformation or rutting.

Results in Table 3-8 indicate that for Area 2 – from the breaking point to the peak – the unmodified samples have higher values than the modified samples. However, for Area 3 – from the peak to the end of the compaction process – the modified samples post higher values indicating higher amounts of energy needed to distort the sample within that region. This is in agreement with the initial observation that the post-peak drop in the gyratory shear is less for the modified samples than the unmodified samples. Nonetheless, an estimate of the total areas – Area 2 + Area 3 – indicates that the rubber-modified samples have lower total values and most probably lower total energies or overall resistance to shear failure.

3.4.2-3 Rate of Change of Gyratory Shear with respect to Air Voids (dGs/dAV)

One parameter of HMA mixes being investigated is the rate of change of gyratory shear per unit change in air voids (dGs/dAV, hereafter denoted by dGs). This is equivalent to the gradient of the straight-line portion (second segment) of the Gyratory Shear vs. Air Void plot from the compaction data.

Figures 3-5 and 3-6 (calculated dGs parameter) indicate that dGs values for the rubber-modified samples are greater than that of the straight samples. This suggests that the rate of change of gyratory shear strength per unit change in air voids is greater for the rubber-modified mixes than the straight mixes. In previous work by Birgisson et al. [47], the parameter dGs was ranked with the known field performance of some mixes and it was noticed that mixes with higher dGs did not perform well compared to those with low dGs values. In addition, these mixes had low strain tolerances (measured by their volumetric strain). Thus mixes with good field performance (low dGs) seem to gain in strength slowly over a larger range of volumetric strain where as mixes with inadequate field performance seem to gain shear resistance quickly over a lower range of volumetric strain. The parameter dGs therefore seems to suggest that the GTR might not improve the field performance of HMA mixes.

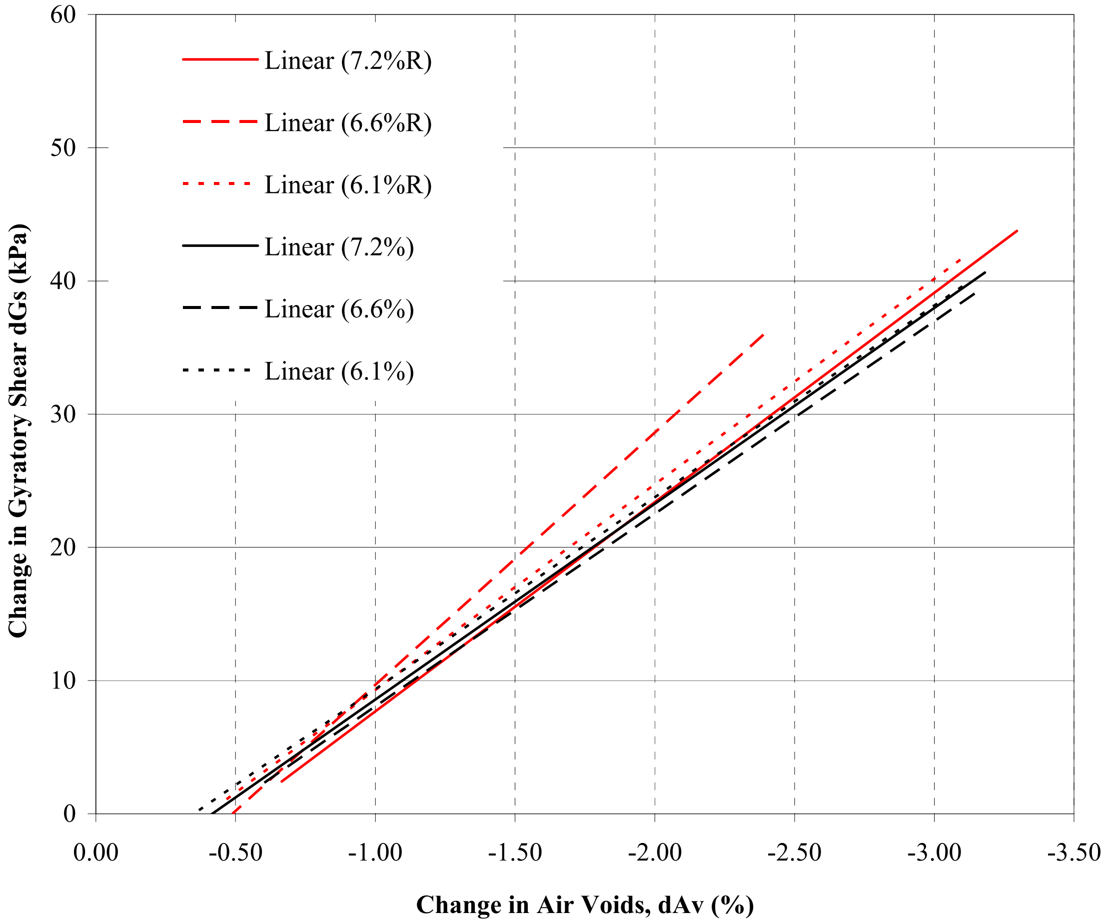


Figure 3-5. Change in G_s versus change in Air Voids for coarse mixtures.

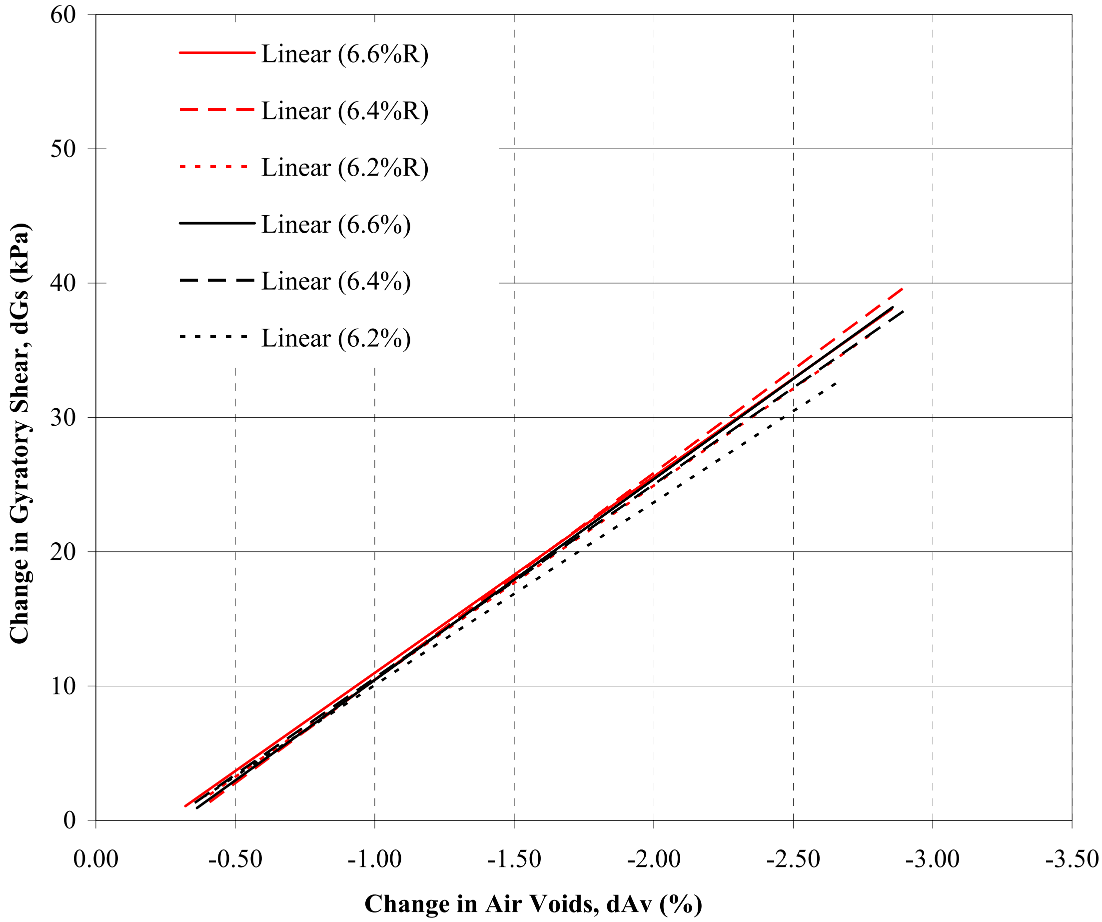


Figure 3-6. Change in G_s versus change in air voids for fine mixtures.

3.4.3 Indirect Tension Test

The Superpave Indirect Tension Test (IDT) – resilient modulus, creep compliance and indirect tensile strength – was run on the different mixtures to further evaluate the effects of GTR on mixture performance. Table 3-9 summarizes the results for test conducted at 50°F (10°C) on two-inch thick specimens, compacted to 7% air voids. The testing procedures and data reduction was done according to the IDT System developed by Roque et al. (1997) [28].

The results indicate that overall the rubber-modified binder slightly increases the resilient modulus (M_R) of the STOA mixes tested at 10° C. There was no significant difference between the M_R value of the WR1 (6.1% rubber) with the W1 sample, and the WR3 (7.2% rubber) value was slightly higher than the W3. The increase in the resilient modulus of the mixes can be attributed to the increased stiffness of the rubber-modified binder.

Table 3-9. Summary of IDT results.

Sample (%AC)	Property								
	Average Resilient Modulus (Gpa)	Average Creep compliance (1/Gpa)	Average Strength (Mpa)	Fracture Energy (kJ/m^3)	Failure Strain (με)	m-value	$\varepsilon o1$	Elastic Energy (KJ/m^3)	DE_{cs} (KJ/m^3)
Straight AC:									
6.1	11.58	5.84	2.20	4.00	2470.58	0.61	2280.60	0.21	3.79
7.2	6.91	14.61	1.64	10.00	6946.94	0.65	6709.60	0.19	9.81
Rubber:									
6.1r	11.09	3.49	1.87	2.20	1575.28	0.59	1406.66	0.16	2.04
7.2r	7.16	8.50	1.29	3.30	3002.11	0.58	2821.94	0.12	3.18

Creep compliance results indicate that after STOA, rubber-modified mixtures had lower measured compliance (@ 1000 seconds) compared to the straight samples. The decrease in creep compliance of the modified mixtures correlates well with the observed binder properties (higher viscosity for GTR modified). Reduction in creep compliance values result in lower mixture cracking resistance – ability to relieve thermal stress build-up in the pavement.

The tensile strength of a HMA sample is the maximum recorded tensile stress before fracture. There was a general decrease in the indirect tensile strength of the rubber-modified samples as compared to the non-modified samples. The increased viscosity of the modified binder does not translate into an increase in the tensile strength of the asphalt mixture. The results suggest that GTR behave like minute 'discrete' grain particles instead of being 'dissolved' in the AC-30 to form a homogeneous modified binder. The increase of the surface area in the mastic (by the rubber particles) might reduce the amount of available asphalt binder for effective bonding with the aggregates, thereby reducing the indirect tensile strength of the modified samples. Also the presence of the dispersed rubber particles could facilitate the propagation and growth of micro-cracks within the binder mastic, which effectively reduces the tensile strength of the mixture.

The fracture energy of an asphalt mixture is the energy per unit volume required to cause failure, and the failure strain is the maximum tensile strain of the material prior to fracture. Higher fracture energy and failure strain suggest higher cracking resistance for the mixture. The rubber-modified mixtures gave lower fracture energy and failure strain values compared to the straight mixtures.

3.4.4 Summary of Findings

The findings of the mixture testing for the rubber-modified asphalt mixtures can be summarized as follows:

- Servopac results indicate that the GTR prevents the mixture aggregate structure to achieve its optimum orientation, which would provide the maximum shear resistance of the HMA mixture.
- The rubber-modified mixtures seem to sustain the post-peak strength (or maximum shear strength) of the mix during or throughout the compaction process. This suggests that the GTR could increase the stability of the mix at low air void contents, even at higher asphalt contents.
- With reference to the IDT results, the rubber-modified mixtures had lower creep compliance which seems to support the point that the GTR could increase the shear resistance and hence the rutting resistance of the mix. The rubber-modified

samples also indicated a general increase in the resilient modulus (or stiffness) of the mixes, even after age hardening.

- However, the rubber-modified samples posted lower indirect tensile strength and fracture energy values at 10°C, which may lead to reduced cracking resistance at intermediate temperatures.

3.5 Evaluation of GTR-Modified Open-Graded Mixtures

In the late 1960s the Federal Highway Administration issued an instruction to State DOTs to develop “standards for pavement design and construction with specific provisions for high skid resistant pavements’’. This included an evaluation to determine whether the aggregates used in the top layer of asphalt pavements were capable of providing adequate skid resistance without polishing. A number of test sections were constructed in Florida to determine the frictional and wearing characteristics of aggregates. Performance evaluation of the test sections was instrumental in the development of specifications for open graded friction courses (OGFC).

Drain down and raveling problems observed during construction of the test sections, as well as the 1988 Florida State Senate Bill 1192 which directed the Florida Department of Transportation (FDOT) to evaluate the potential use of reclaimed tire rubber in asphalt pavement construction, prompted research into the use of ground tire rubber as an asphalt additive for the production of open graded friction courses. Research showed that finely ground tire rubber appeared promising in this regard. However, open-graded friction-course projects constructed with various asphalt-rubber binders (at various percent content of ground tire rubber) have exhibited different degrees of surface initiated cracking.

This project focused on the evaluation of the effects of asphalt rubber friction courses on surface initiated longitudinal wheel path cracking. The primary objectives that relate to the use of modified binders are summarized below:

- Determine whether the addition of higher percentages of rubber in the open-graded friction course (OGFC) can inhibit or mitigate the occurrence of surface initiated longitudinal wheel path cracking
- Evaluate the effect of rubber and binder contents on aging and its effects on the fracture resistance of OGFC

3.5.1 Materials and Methods

The coarse aggregate was crushed limestone aggregate from South Florida by Tarmac Florida Inc., (pit number 87-145), which is a standard aggregate for the FDOT FC-2 friction course. The fine sand (or local sand) used for the laboratory prepared mixtures came from North Florida by Anderson Columbia Co., Inc. The modified binder was produced by blending #80 GTR (crumb rubber passing sieve #80) by Rouse Rubber Industries, Inc, with AC-30 asphalt cement for approximately 20 minutes at 300°F with a Silverson L4R High Shear Mixer. The asphalt was blended with different rubber contents – 5%, 10% and 15% by weight of total binder.

Field-section mixture design (FDOT open-graded FC-2) was used for the laboratory prepared samples with 92 percent FC-2 stone and 8 percent local sand. The job-mix formula and a summary of the mixture designs are found in Tables 3-10 and 3-11, respectively.

The design asphalt content (6.3 %) was increased 0.5 percent for each 5 percent increase in 80-mesh rubber. Therefore, mixtures with 0%, 5%, 10% and 15% contained 6.3%, 6.8%, 7.3%, and 7.8% asphalt content, respectively. The total binder contents of the resultant mixtures were 6.3%, 7.158%, 8.111% and 9.176%. Also, control samples with asphalt contents of 6.3%, 6.8%, 7.3%, and 7.8%, were produced to assess the individual effects of the varying asphalt- and rubber-content.

Table 3-10. Job mix formula for the FC-2 friction course.

Sieve Size	Percent Passing
1/2	100
3/8	95.8
#4	35.0
#10	11.9
#40	10.2
#80	7.3
#200	2.3

Table 3-11. Mixture designation and binder contents.

Mixture Designation	Asphalt-Rubber Content	Total Binder Content (%)
R1	6.8% AC & 5% Rubber	7.2
R2	7.3% AC & 10% Rubber	8.1
R3	7.8% AC & 15% Rubber	9.2
C1	6.8% AC	6.8
C2	7.3% AC	7.3
C3	7.8% AC	7.8
T1	6.3% AC	6.3

The Indirect Tensile tests was used to determine the resilient modulus, creep compliance, strength, failure strain and fracture energy density of the mixtures at a testing temperature of 10 ºC. Testing and data reduction was performed according to the procedures established by Roque et al. [27].

3.5.2 Description of Field Test Sections

The field sections for the study were located on SR-16 starting about 1.5 miles northwest of its intersection with SR-200 (US-301) in Starke, Florida and ending at SR-233. Each section (2500 ft) is part of two-lane highways with 12-ft wide lanes and 4-ft wide paved shoulders. The pavement structure for all the sections was relatively the same, with approximately $^{5}/_{8}$ inches friction-course thickness (as determined from construction data). Figure 3-7 is a sketch of the site location of the field sections.

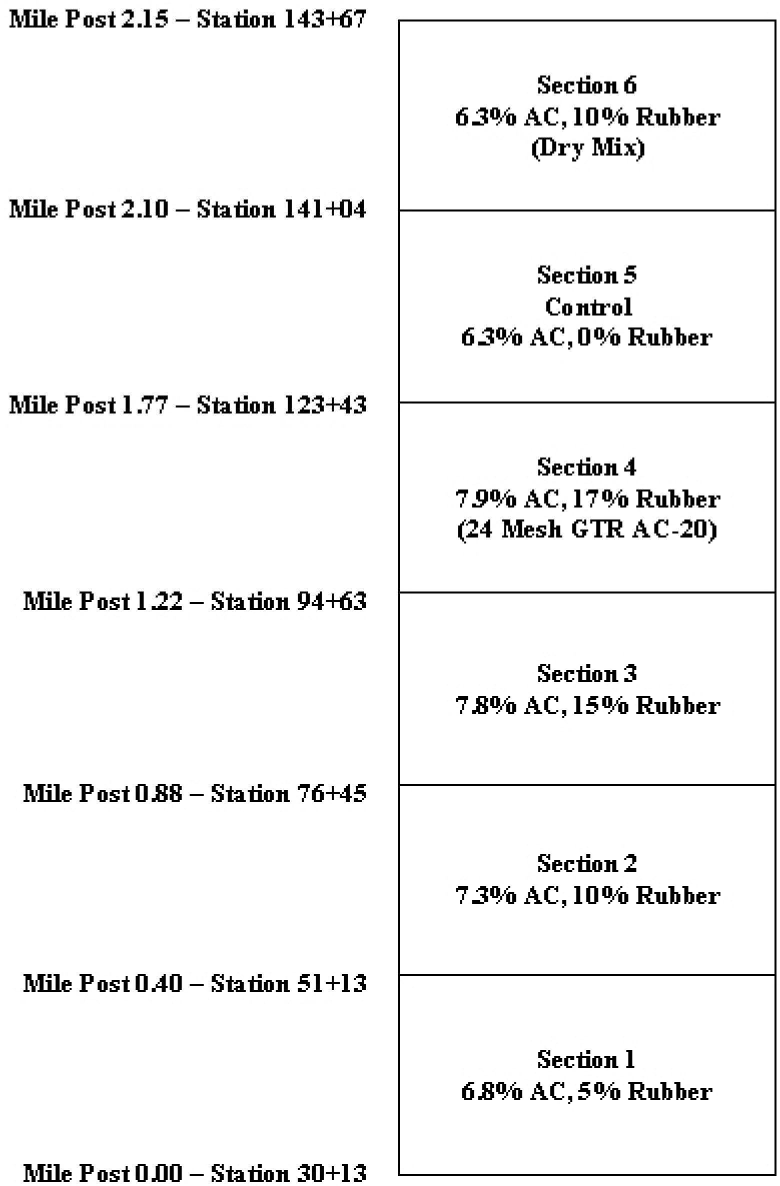


Figure 3-7. Schematic of site locations for field sections.

FDOT construction information indicated that all sections were constructed in June 1989 by the same paving contractor, using the same subcontractor for blending the GTR. The pavement structure for the sections is presented in Table 3-12.

Table 3-12. Pavements structural characteristics of field test sections

Layer	Thickness (in)
Friction Course (FC-2)	0.63
Structural Layer (Type S)	3.00
Surface Treatment	0.75
Limerock Base	5.90
Stabilized Subbase (Type-B)	11.80

The general environmental and traffic conditions for the field test sections were considered similar given the proximity to one another. Air temperatures in the vicinity of the sections were measured between 50° F (10° C) and 68° F (20° C) in the winter and 86° F (30° C) and 113° F (45° C) in the summer. The annual average daily traffic (AADT) for the sections ranged from 4,029 to 6,000 (40,681 to 61,075 ESALs) for the years 1989 until 1998.

3.5.3 Field Sections Performance

The FDOT conducted a long-term evaluation of the asphalt rubber surface mixtures constructed on SR-16 to determine their performance based on ride quality, rutting, cracking and patching, and skid resistance. Regarding skid resistance measurements the FDOT found that the friction performance of both the asphalt rubber and the control sections were not significantly different, and were constant for the first 50 months. However, the field sections with 10 and 15 percent rubber had better cracking performance (less) than the control section

Ten years after construction (1999), the sections were cored to determine the depth and width of cracking, and to help in the verification of the structural characteristics of the sections. Cores were extracted from the wheel path (WP) – through the cracks – and between the wheel paths (BWP) – undamaged. Figure 3-8 is a schematic of the core locations.

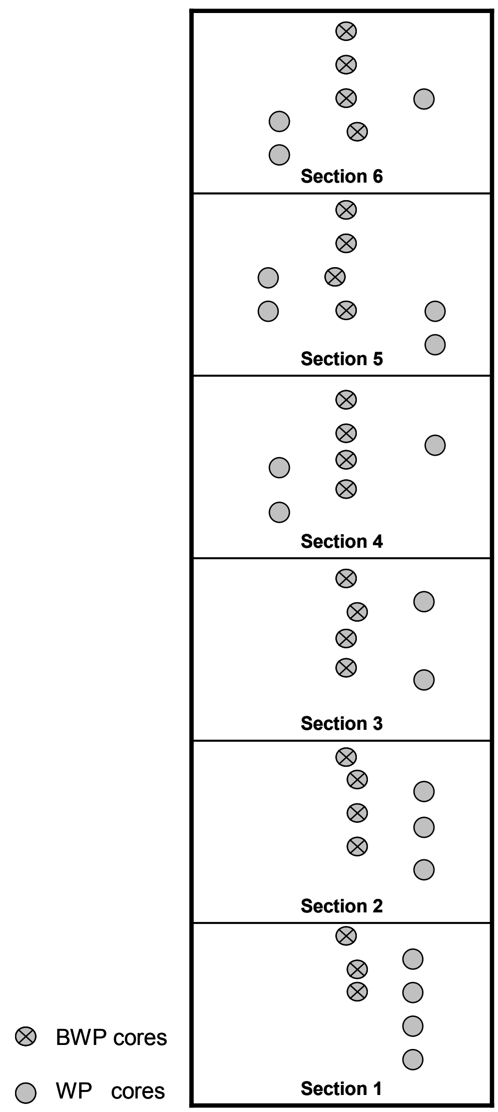


Figure 3-8. Schematic of the core locations.

The section with 5% rubber exhibited longitudinal surface cracks in both wheel paths, as well as some surface cracks between wheel paths. In general the 5% rubber section and the control section had the most longitudinal surface cracks. A closer investigation of the field cores revealed that the cracks initiated at the surface and went through the friction course to an average depth of 1.68 inches.

The section with 10% rubber content showed minimum surface hairline cracks mostly in the longitudinal direction, and cores indicated that the cracks were confined to the surface of the pavements and had not propagated through the friction course. The 15% rubber content section exhibited more cracking than the 10% rubber section; the cracks progressed through the friction course to an average depth of 1.08 inches. Finally, the control-mixture section showed extensive cracks – between wheel paths and on wheel paths – over the entire section, with average crack-depth measured at 2.05 inches.

3.5.4 Laboratory Testing Results

Binder properties measured in the lab included viscosity at 140° F (60° C), penetration at 77° F (25° C), and dynamic shear rheometer results at 77° F (25° C) and 147° F (64° C). The mixtures were evaluated with the Superpave IDT tests – resilient modulus, creep compliance, tensile strength, failure strain, and fracture energy density – to determine mixture properties. The laboratory testing results were then used to help explain the field performance of the various sections.

3.5.4-1 Summary of Binder Testing

Tables 3-13 and 3-14 show the viscosity and penetration results for the various binders, respectively, where there is an increase in viscosity and reduction in penetration with the addition of rubber. The viscosity increase because of the GTR additive prevented drain-down and made the production of higher binder-content mixtures possible.

Table 3-13. Penetration of binders at 25° C.

Trial	Binder	Pen	Mean	Standard Deviation
1	Virgin AC-30	61	60	1
2		60		
3		60		
1	RI(5% Rubber)	39	39	1
2		39		
3		40		
1	R2(10% Rubber)	37	37	1
2		37		
3		36		
1	R3(15% Rubber)	33	34	1
2		33		
3		35		

Table 3-14. Brookfield viscosity of binders at 60° C.

Trial	Binder	Viscosity (cP)	Mean	Standard Deviation
1	Virgin AC-30	3.67E+05	378200	10173
2		3.88E+05		
3		3.80E+05		
1	RI(5% Rubber)	5.90E+05	583333	6506
2		5.77E+05		
3		5.83E+05		
1	R2(10%Rubber)	9.93E+05	990033	6116
2		9.94E+05		
3		9.83E+05		
1	R3(15% Rubber)	2.20E+06	2380000	158447
2		2.50E+06		
3		2.45E+06		

DSR results, presented in Tables 3-15 and 3-16, showed the binders to be within acceptable Superpave specification limits for fatigue cracking and rutting. The modified binders showed a reduction of the phase angle (δ) and an increase of G*/Sin (δ), as well as reduced temperature susceptibility and stiffness at lower in-service temperatures. The trends reported above are conducive to better rutting and cracking performance of asphalt mixtures.

Table 3-15. Dynamic shear rheometer results of binders at 25° C.

Trial	Rubber Blend	G*(kPa)	δ	G*sinδ
1	Virgin AC-30	1110	66.7	1020
2		1070	67.4	985
3		902	67.4	833
1	RI (5% Rubber)	930	63.9	892
2		993	63.6	905
3		1010	63.8	891
1	R2 (10% Rubber)	1210	59.4	1040
2		1060	59.5	915
3		1080	59.7	931
1	R3 (15% Rubber)	1080	57.2	910
2		1060	56.8	1035
3		1050	57.0	883

Table 3-16. Dynamic shear rheometer results of binders at 64° C.

Trial	Rubber Blend	G*(kPa)	δ	G*/sinδ
1	Virgin AC-30	1.93	86.2	1.93
2		2.01	86.1	2.02
3		2.02	86.2	2.02
1	RI (5% Rubber)	2.95	83.4	2.97
2		2.94	83.5	2.96
3		3.11	83.2	3.13
1	R2 (10% Rubber)	4.98	79.3	5.06
2		4.98	79.9	5.06
3		4.10	79.2	4.17
1	R3 (15% Rubber)	7.37	76.7	7.58
2		7.00	76.2	7.20
3		7.49	76.6	7.69

3.5.4-2 Summary of Mixture Results

Mixture-testing results suggest that field performance can be correlated to the measured fracture energy density and failure strain. These two values – fracture energy density and failure strain – peaked at the 8.1% binder content (corresponding to 10 % rubber), which corresponds to the best performing mixture in the field. Figures 3-9 and 3-10 show the trend of the fracture energy and failure strain versus binder content.

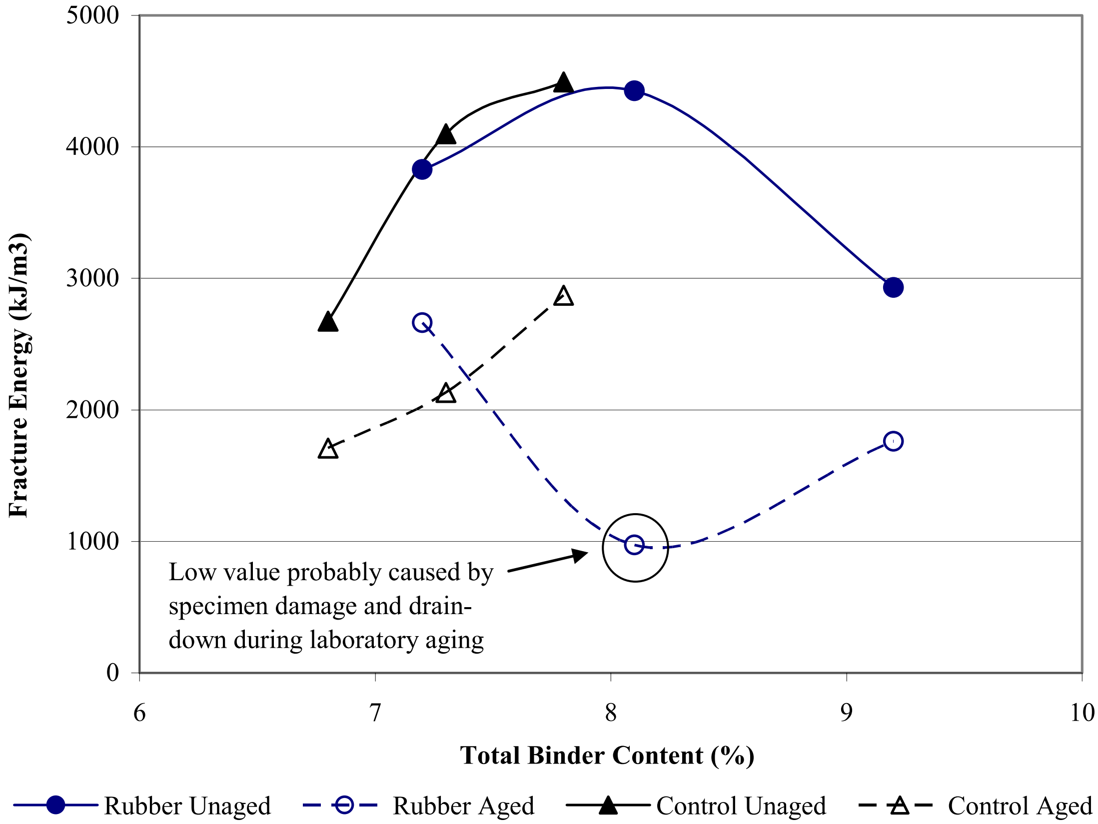


Figure 3-9. Fracture energy versus binder content.

The fracture energy and failure strain results indicate that the benefit of the rubber additive is the ability to increase the total binder in the mixture. The resilient modulus results (Figure 3-11) show that the addition of rubber provided higher values at higher binder contents, whereas the tensile strength results (Figure 3-12) indicated that the addition of rubber did not cause any significant difference in the mixture behavior.

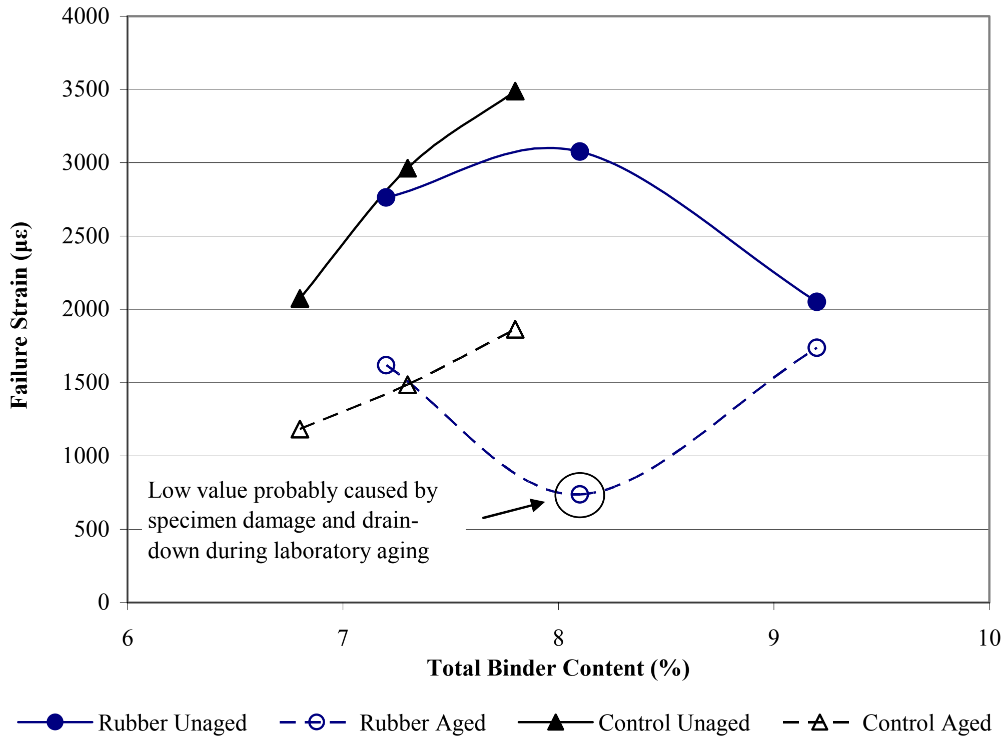


Figure 3-10. Failure strain versus binder content.

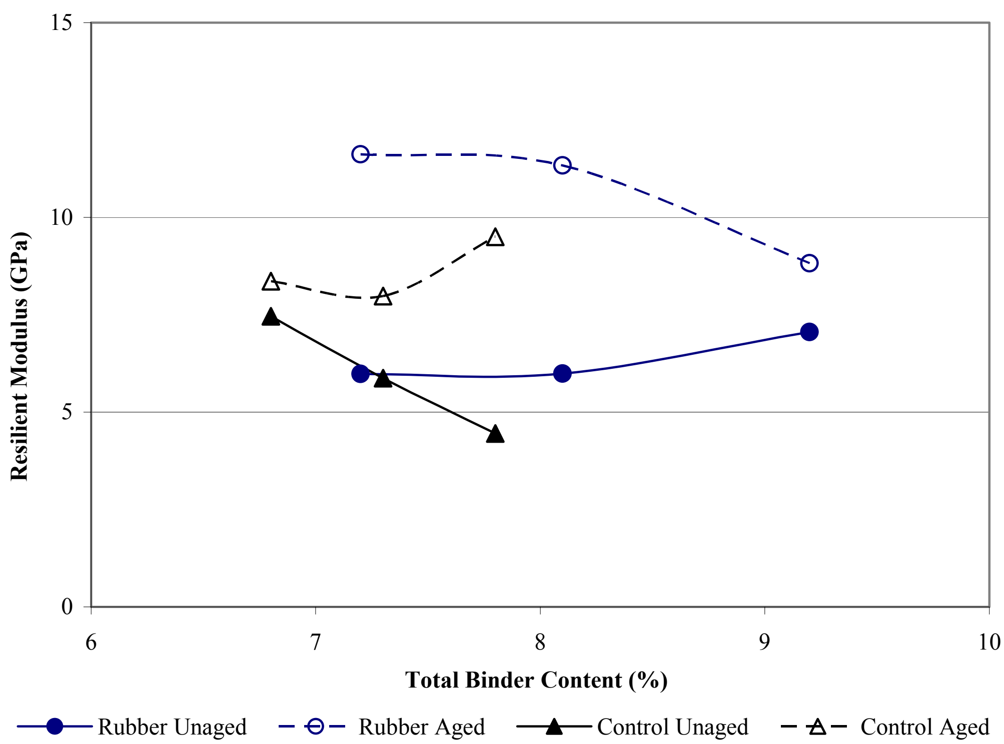


Figure 3-11. Resilient modulus versus binder content.

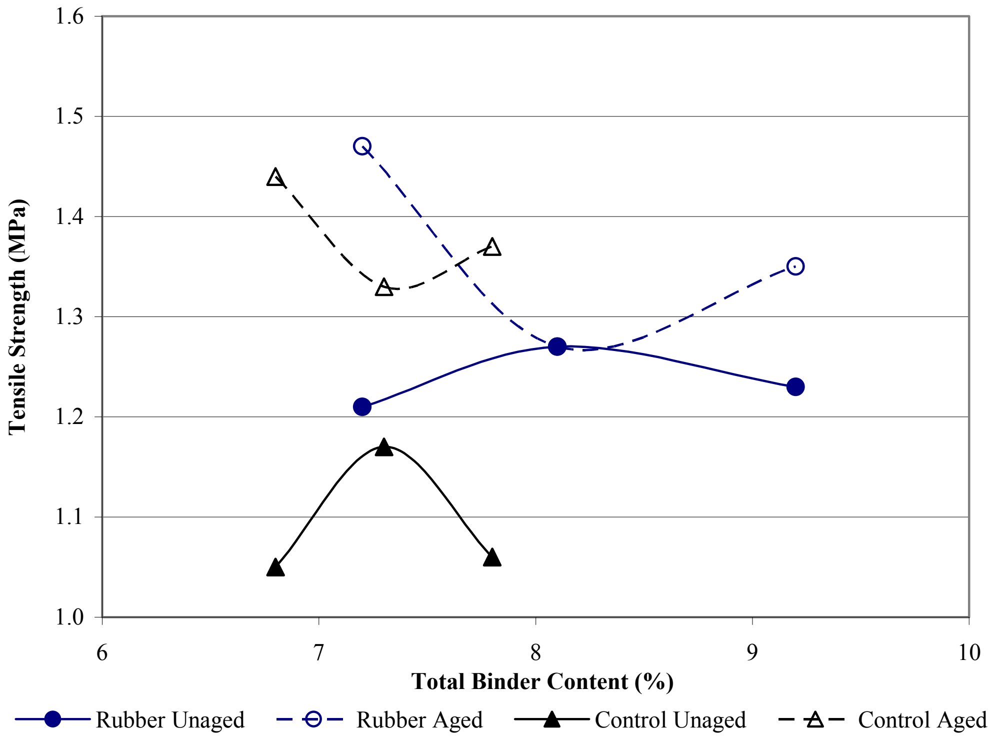


Figure 3-12. Tensile strength versus binder content.

3.5.5 Summary of Findings and Conclusions

The findings of the binder and mixture testing can be summarized as follows:

- Viscosity at 60° C increased with the addition of rubber
- Penetration at 25° C decreased significantly when added 5% rubber to the virgin binder; further increases in rubber content did not result in significant reductions in penetration values
- The rubber additive reduced the temperature susceptibility of the binder
- At lower temperatures it appeared that the dynamic stiffness of the binder was reduced by the addition of the rubber.
- The addition of rubber reduced the phase angle (δ) and increased values of G*/Sin (δ) at 25° C and 64° C which implies an improved resistance to permanent deformation
- Fracture energy density and failure strain values of unaged mixtures peaked at a total binder content of 8.1% (10% rubber) which matched the best performing field section

- Laboratory control mixtures produced with no rubber exhibited the same fracture energy density as the rubber modified mixture produced with the same total binder content. This indicates that the primary benefit of the rubber is to allow the production of mixtures with higher total binder content.

- Resilient modulus values indicate that the addition of the rubber also helped the mixture retain higher resilient modulus at higher binder contents and higher temperature.

- The addition of the rubber does not appear to have a significant effect on the tensile strength of the mixtures

Based on the findings above, the following conclusions were made:

- The potential benefits associated with the addition of rubber are realized through the reduced temperature susceptibility and higher stiffness at lower temperatures

- The addition of rubber also allowed the introduction of higher binder contents without drain down during construction

- The addition of rubber and subsequent higher binder content appears to improve the cracking performance of OGFC

3.6 Fatigue Cracking Evaluation of SBS-Modified Mixtures

As mentioned earlier, the Superpave mix design procedure being implemented today essentially remains a volumetric design procedure, devoid of validated performance-based mechanical property tests for asphalt mixtures. The volumetric design procedure assumes that the gyratory compactor number of gyrations represent the traffic conditions to which the mixture will be subjected. The design asphalt is determined as the asphalt content required to obtain 4% air voids at the design number of gyrations for a particular level of traffic and environment. No other mechanical testing is currently required to ensure the adequate mixture performance.

Therefore, Superpave mix design has essentially placed stricter requirements on the shear resistance of the mixtures at higher temperatures, but no appropriate checks to guarantee cracking resistance of mixtures. Observations made over the past few years in the use of Superpave design procedure, indicate that it may not be possible to produce Superpave mixtures with conventional asphalt cement for certain levels of traffic and environment to have both adequate rutting and cracking resistance. According to results of recent research on modifiers, one way to achieve the above objective of producing a mixture with sufficient fracture resistance as well as desirable rutting resistance is through the use of asphalt modifiers.

The overall objective of this study was to achieve a better understanding of how SBS polymer modification affects the cracking resistance of asphalt mixtures. A clearer understanding of these effects will lead to better guidelines for their use, as well as improved methods to characterize their benefit and to evaluate their potential benefit in specific mixtures and loading environments.

3.6.1 Materials and Methods

The aggregate used for this study was limestone, and aggregate gradation was coarse gradation (C1) and fine gradation (F1), which passes below and over the Superpave restricted zone, respectively. The coarse gradation (C1) acted as the main aggregate structure for the majority of the tests in this study; the fine gradation (F1), was used to evaluate the effect of SBS polymer on cracking for different aggregate gradation. Figure 3-13 shows the gradation chart of the mixtures and the restricted zone and control points, and Table 3-17 shows the blend proportion of the aggregates.

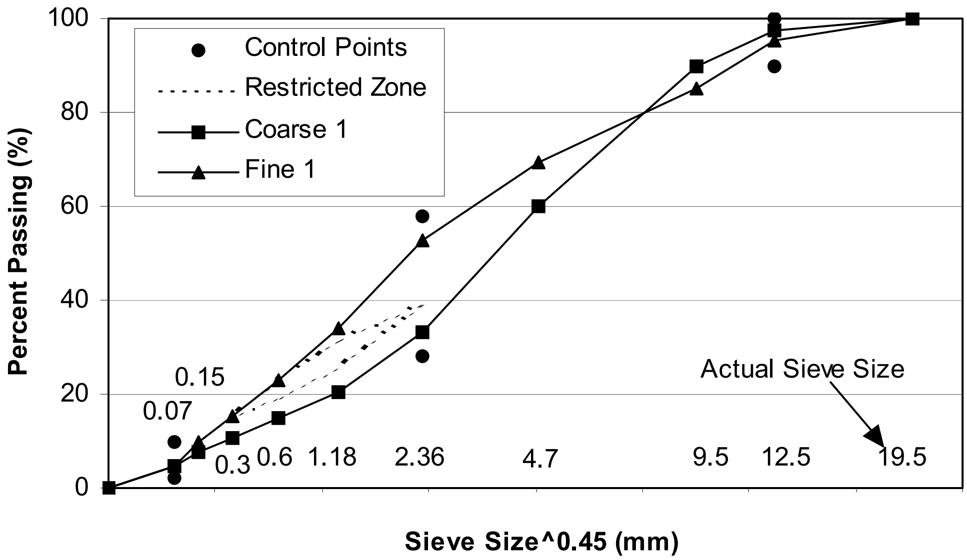


Figure 3-13. Aggregate gradations (Coarse 1 and Fine 1).

Table 3-17. Aggregate blend proportions.

	S1A (%)	S1B (%)	Screenings (%)	Filler (%)
C1	10.20	63.27	25.51	1.02
F1	20.30	25.37	53.29	1.03
Bulk Specific Gravity	2.43	2.45	2.53	2.69

Two binders were involved in this study – one control and one SBS polymer modified asphalt. According to the information provided by CITGO Asphalt Refining Company, the control asphalt binder is characterized as PG 67-22 or AC 30, and the modified asphalt binder as PG 76-22. SBS polymer (3%) was blended with the control asphalt in the process of high shear milling to produce the SBS modified asphalt.

The laboratory asphalt mixtures produced for testing and evaluation were designed with the Superpave Volumetric Mix Design procedure, which bases its selection for design asphalt content on a set of criteria on the volumetric properties of the mixture (VMA, VFA, density) at 4 % air voids. Apart from the above procedure that determines the design asphalt content, the aggregates need to fulfill a set of criteria for the consensus and source properties that aim to prevent the use of substandard aggregates in producing asphalt mixture. Table 3-18 shows the compaction effort for the various traffic levels in the Superpave mix design procedure. Four different traffic levels – Level 3, 4, 5, and 6 – which cover more than 90 % of traffics running on arterial road, interstate highway, and turnpike, were selected for the mixture design.

Table 3-18. Traffic levels and gyratory compaction effort.

Traffic level (Millions of EASL's)	N_{ini}	N_{des}	N_{max}
1 (<0.3)	6	50	75
2 (0.3-1.0)	7	75	115
3 (1-3)	7	75	115
4 (3-10)	7	75	115
5 (10-30)	8	100	160
6 (30-100)	9	125	205
7 (>100)	9	125	205

Standard Superpave IDT tests were performed on all mixtures to determine resilient modulus, creep compliance, m-value, D_1, tensile strength, failure strain, fracture energy, and dissipated creep strain energy to failure. A significant number of additional tests were also performed including: (1) repeated load fracture tests to evaluate measured crack growth behavior, and (2) longer-term creep tests to failure to evaluate the potential of this type of test to uniquely characterize SBS polymer modification.

3.6.2 General IDT Mixture Properties

Table 3-19 presents the Superpave IDT results for tests run at 10° C to obtain asphalt mixture properties. As expected, the mixtures had higher resilient modulus at lower binder contents, but SBS modification had relatively little effect on resilient modulus at either binder content. Modification had no effect on resilient modulus at 7.2% binder content and reduced the resilient modulus by about 20% at 6.1% binder content. This seems to indicate that the polymer has little effect on response at small strain or short loading times, which implies that polymer modification does not reduce the mixture's effectiveness from a structural point of view.

Conversely, the SBS modifier dramatically reduced the creep compliance of mixtures at both low and high asphalt contents. Thus, the SBS polymer appears to have a much greater influence on the time-dependent response, and perhaps specifically the creep response, than on the elastic response of the mixture. The lower rate of creep response is more clearly reflected in the much lower m-value of the modified mixtures at both asphalt contents, as shown in Figure 3-14. Prior research, which resulted in the development of HMA fracture mechanics model, clearly showed that there is a direct relationship between the rate of creep and the rate of micro-damage accumulation in asphalt mixtures.

Table 3-19. Superpave IDT results (Coarse 1, STOA).

Sample	Property							
	Resilient Modulus (Gpa)	**Creep compliance at 1000 seconds (1/Gpa)**	**Tensile Strength (Mpa)**	**Fracture Energy (kJ/m^3)**	**Failure Strain (10^{-6})**	**m-value**	**D_1**	**$DCSE_f$ (kJ/m^3)**
	Temperature: 0 °C							
6.1	15.42	0.77	2.67	1.50	809.9	0.51	1.93E-07	1.27
7.2	11.74	1.66	2.54	2.00	1184.4	0.52	2.99E-07	1.73
6.1SBS	14.28	0.90	3.00	2.40	1219.1	0.45	2.63E-07	2.08
7.2SBS	13.03	1.08	2.89	2.70	1349.3	0.44	3.48E-07	2.38
Temperature: 10 °C								
6.1	11.56	5.90	1.87	4.00	2467.6	0.61	6.04E-07	3.85
7.2	7.18	13.44	1.69	4.90	3756.3	0.62	1.21E-06	4.70
6.1SBS	9.26	3.04	1.95	3.80	2291.2	0.45	7.69E-07	3.59
7.2SBS	7.37	5.20	1.93	5.10	3725.7	0.47	1.33E-06	4.85
Temperature: 20 °C								
6.1	5.80	20.37	1.61	7.20	7430.0	0.61	1.61E-06	6.98
7.2	4.72	50.98	0.90	7.20	9634.1	0.63	3.68E-06	7.11
6.1SBS	6.04	9.33	1.69	5.30	5098.5	0.52	1.55E-06	5.06
7.2SBS	4.98	12.41	1.59	9.70	9838.5	0.48	4.10E-06	9.45

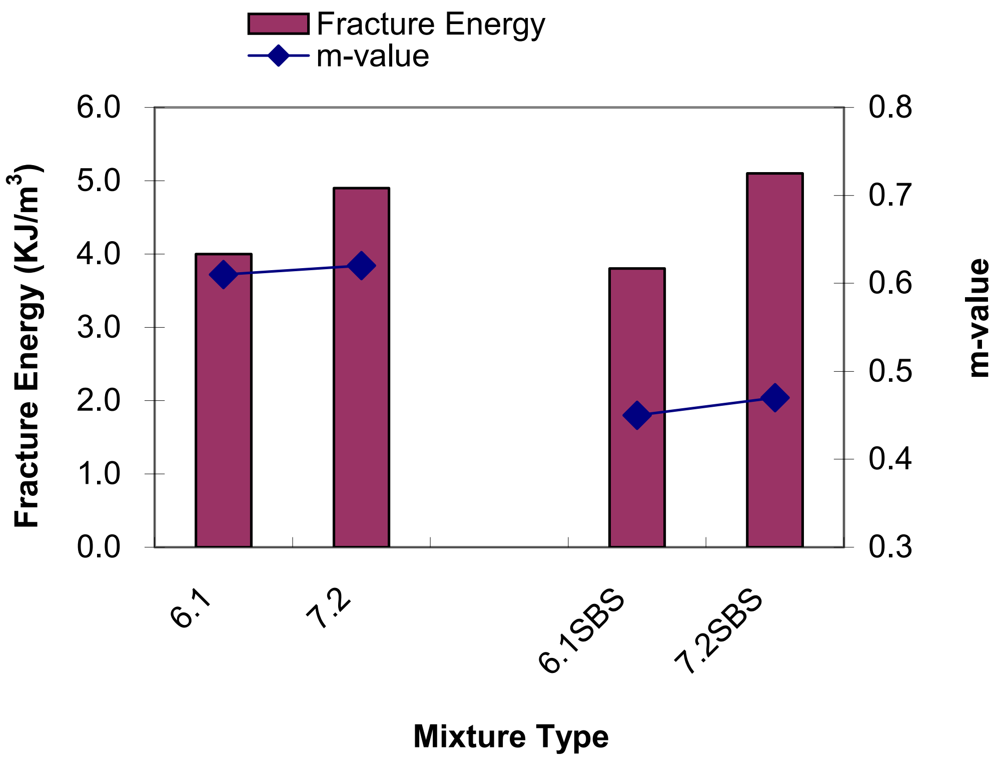


Figure 3-14. Fracture energy and m-value from Superpave IDT.

The results presented in Table 3-19 also indicate the polymer had almost no effect on tensile strength, failure strain, fracture energy (FE), dissipated creep strain energy to failure ($DCSE_f$), or creep parameter D_1. Therefore, it appears that the benefit of the polymer is primarily, and almost exclusively, reflected in the reduced m-value, which indicates a reduced rate of micro-damage accumulation. Fracture and longer-term creep test results presented later will further confirm this point.

Unfortunately, lower m-values are not uniquely related to the addition of polymers. For example, age-hardening a mixture will reduce its m-value, but it will also reduce its FE and $DCSE_f$, which would counteract and likely overwhelm any benefit gained by reducing the m-value in this way. The benefit of the polymer comes from the fact that the m-value is reduced without affecting FE or $DCSE_f$. It is thought that the network, or secondary structure of the polymer phase, reduced the m-value, which is related to the viscous response of the mixture. However, the polymer does not have sufficient time to affect FE during the strength test (around 4 to 5 seconds). Therefore, further research was conducted to evaluate other tests and/or interpretation procedures that may be used to uniquely characterize the effect of the SBS polymer.

3.6.3 Temperature Sensitivity

Standard Superpave IDT tests were also conducted at 0° C and 20° C to identify the temperature sensitivity of Superpave IDT properties and present the mixture properties for further analysis. As shown in Figure 3-15, at all range of temperatures tested, the mixtures at lower binder contents exhibited the higher resilient modulus than those at higher binder content, but SBS modification had relatively little effect on resilient modulus at either binder content. This seems to indicate that the polymer has little effect, while the amount of asphalt binder has a relatively bigger effect, on response at small strain or short loading times across all of the temperatures tested. Figure 3-15 also indicates that temperature sensitivities of resilient modulus of mixtures are almost identical, as shown in the exponential constant that ranges from –0.0489 to –0.043. It appears that temperature sensitivity of the response at small strain or short loading time is likely not affected by the modification of binder for this particular asphalt mixture.

The results presented in Figure 3-16 to 3-18 also indicate the polymer had almost no effect on the temperature sensitivity of tensile strength, failure strain, fracture energy (FE), or dissipated creep strain energy to failure ($DCSE_f$). Therefore, it appears that regardless of the modification of the binder, the temperature sensitivity of the response at short loading time is almost identical for a particular base asphalt and mixture type.

On the other hand, the SBS polymer dramatically reduced the creep compliance at all temperatures between 0° and 20° C. In addition, the SBS polymer reduced the temperature sensitivity of creep compliance, which can be represented as the slope of exponential curve in creep compliance versus temperature, at both low and high asphalt contents, as shown in Figure 3-19. The reduced creep compliance and temperature sensitivity of the modified mixtures (6.1SBS and 7.2SBS) are primarily from the reduced m-value shown in Figure 3-20, while those of the low binder content mixtures (6.1 and 6.1SBS) are primarily results of the reduced D_1 shown in Figure 3-21. Thus, it appears that among the creep parameters represented in power model, m-value reflects primarily the binder characteristics, while D_1 reflects the structural characteristics of mixture.

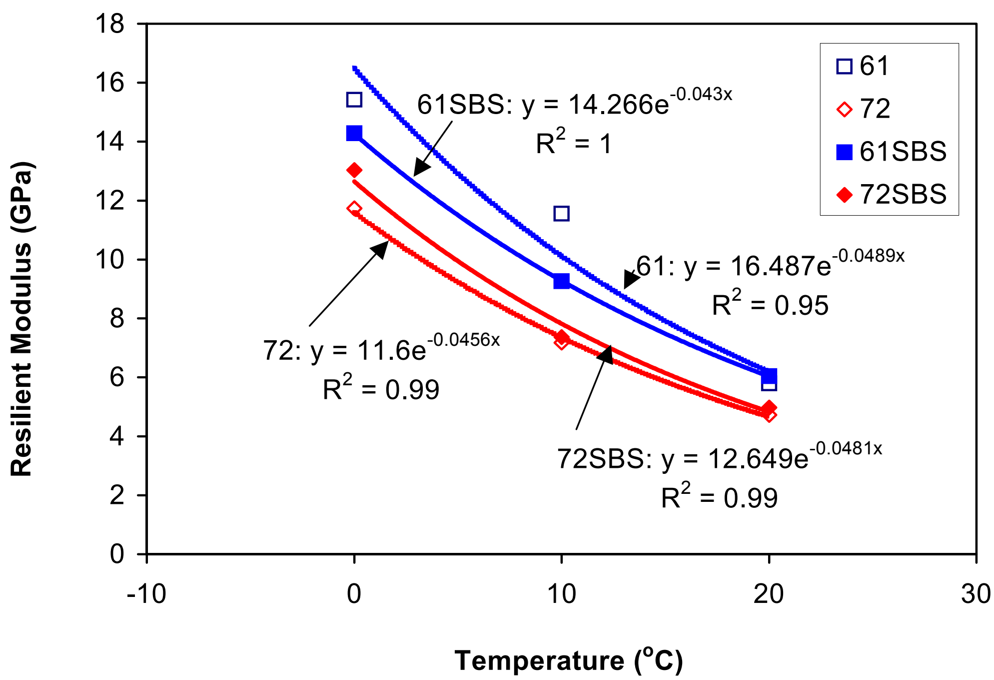


Figure 3-15. Temperature sensitivity of resilient modulus.

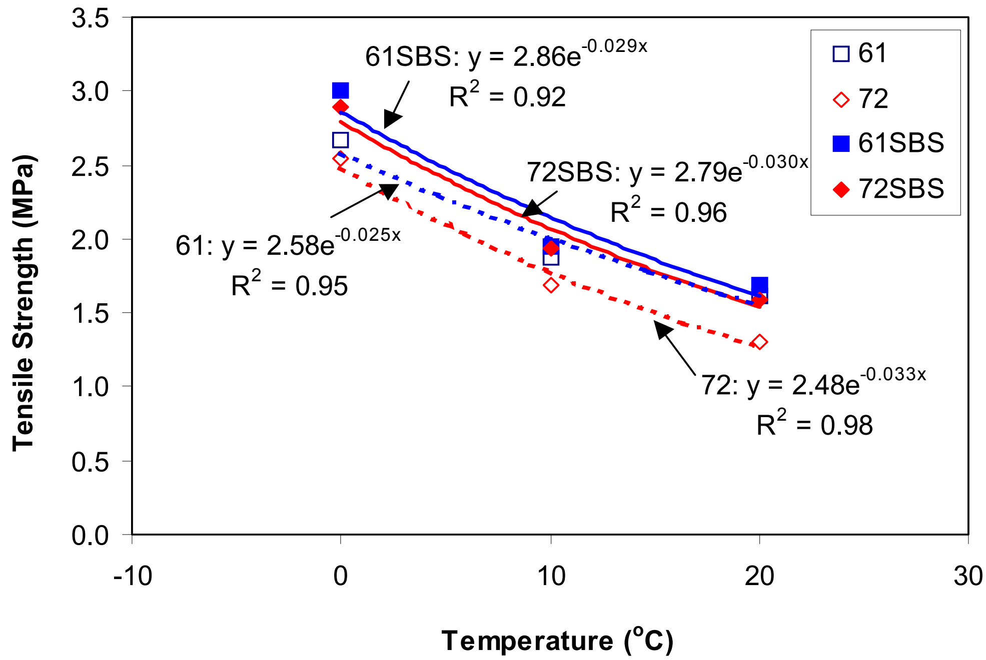


Figure 3-16. Temperature sensitivity of tensile strength.

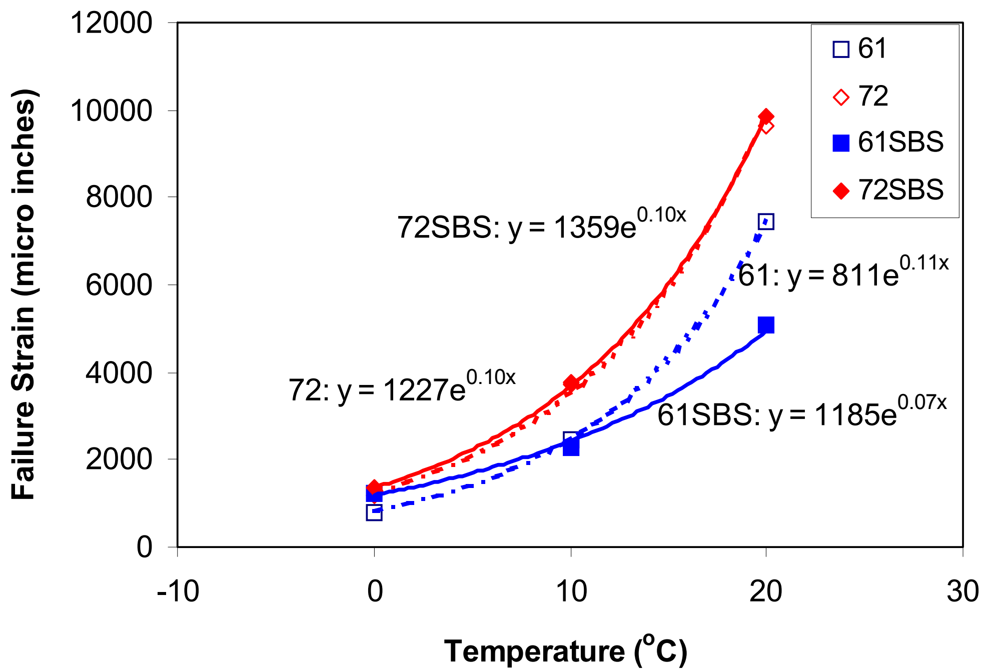


Figure 3-17. Temperature sensitivity of failure strain.

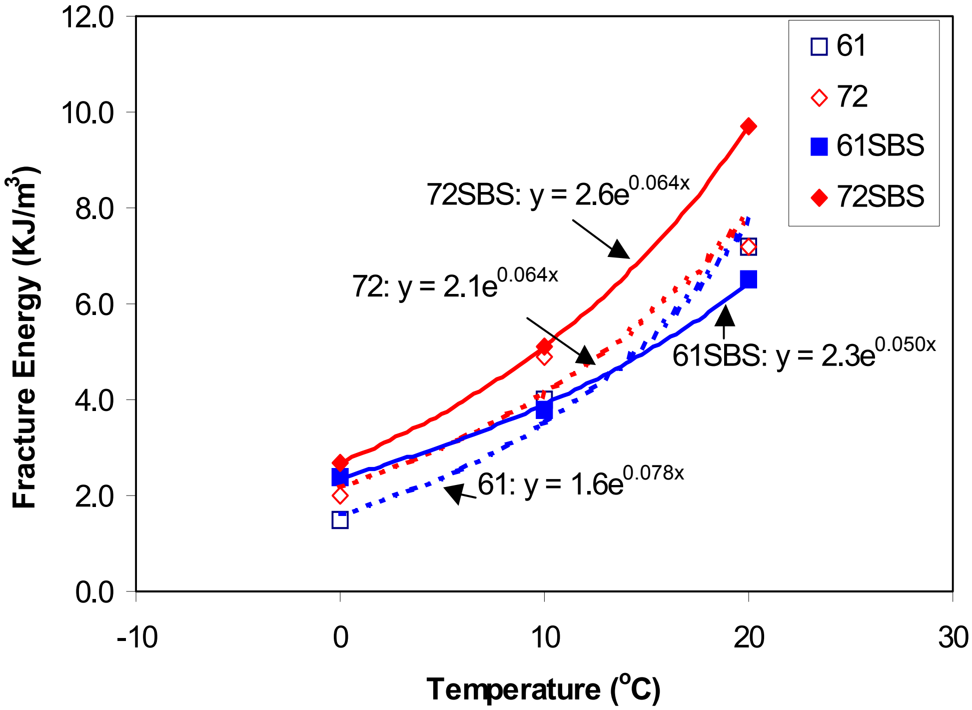


Figure 3-18. Temperature sensitivity of fracture energy.

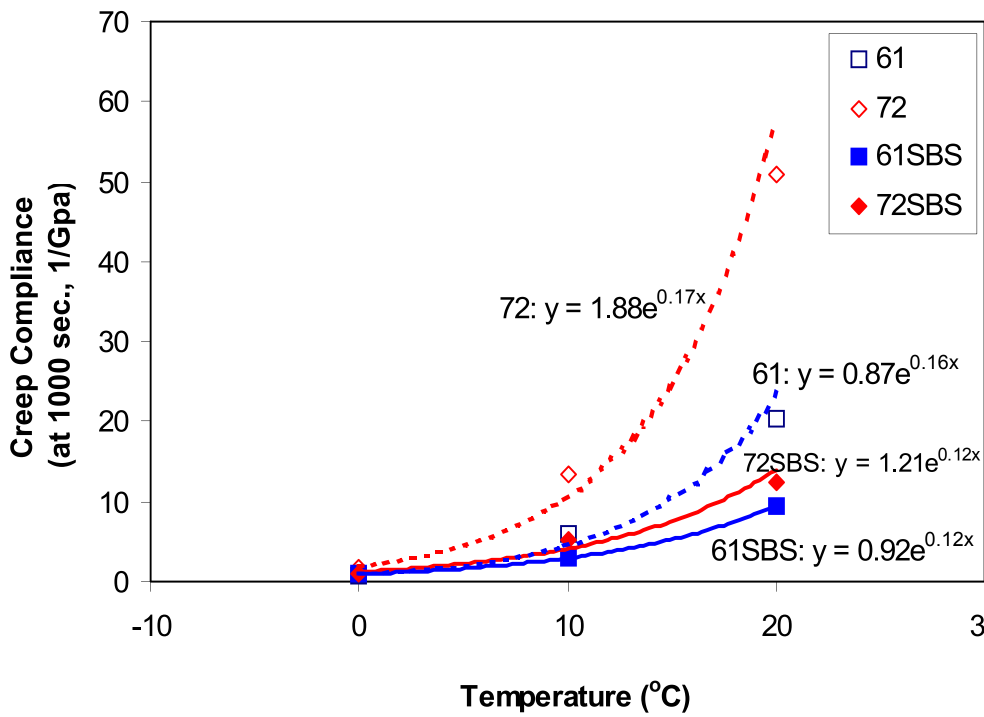


Figure 3-19. Temperature sensitivity of creep compliance.

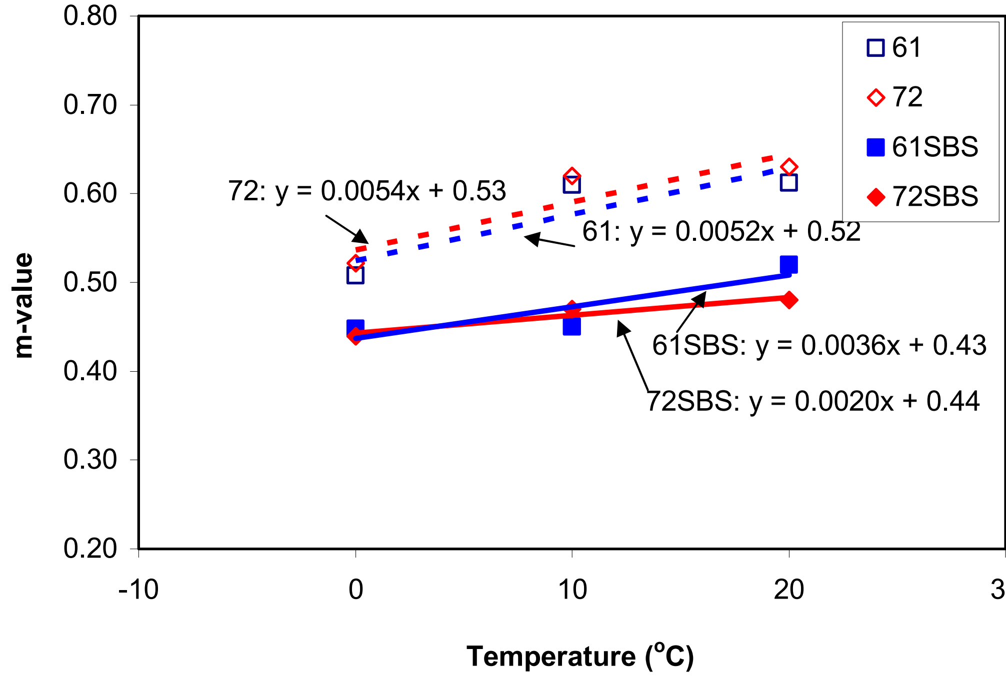


Figure 3-20. Temperature sensitivity of m-value.

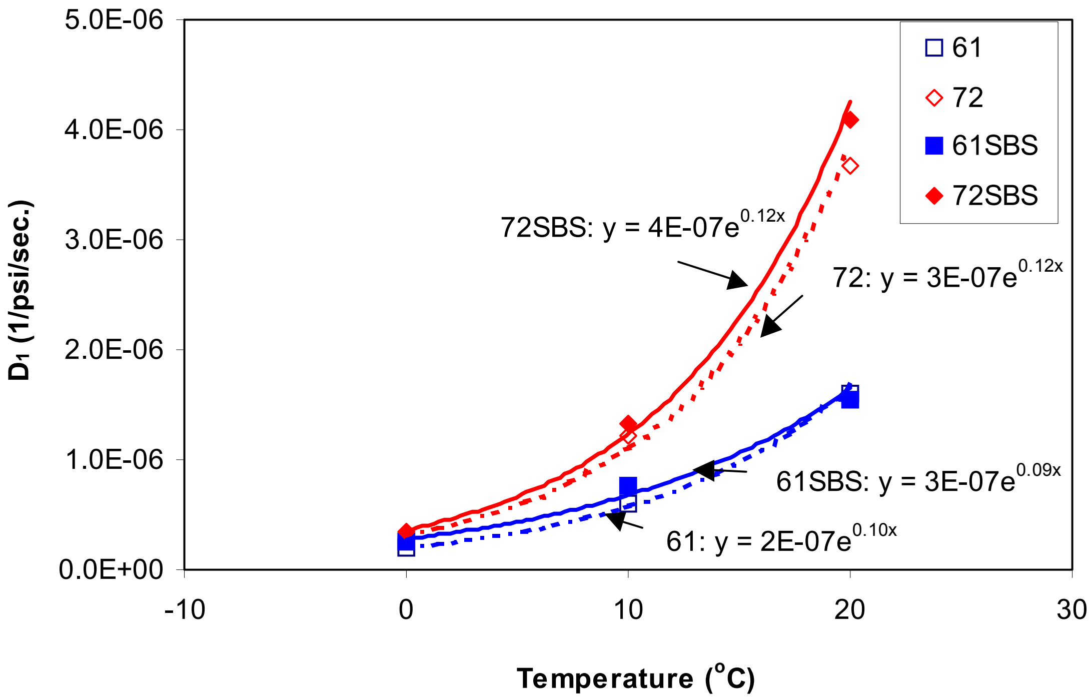


Figure 3-21. Temperature sensitivity of D_1.

3.6.4 Repeated Load Fracture Test

There were implications that SBS modifier seemed to improve cracking resistance as determined from mixture properties obtained from the Superpave IDT. Superpave IDT results the fundamental material properties from which fracture can be predicted, but it was felt that greater confidence would be achieved by performing actual fracture tests. Fracture tests are necessary to simulate closely the fracture state, and thus to identify the effect of SBS modifier and the binder content on rate of micro-damage development and rate of macro-crack growth. Tests were performed and analyzed according to the procedures described by Roque et al. [28].

After conducting and analyzing of fracture tests, the resulting resilient deformation was plotted versus number of load repetitions. Figure 3-22 shows the horizontal resilient deformation (δ_H) during the fracture test. There was a jump in first part of the resilient deformation of all samples. This jump is caused by steric softening and probably by increases in temperature, which reduces the stiffness of the asphalt mixture resulting in higher resilient deformation. However, this jump in resilient deformation has no physical meaning from the damage point of view (Zhang, 2000) [29]. One way of eliminating the effect of the initial increase in resilient deformation is to shift the initial resilient deformation (δ_i) to the original resilient deformation (δ_o). The original resilient deformation can be determined by extrapolating the linear portion of the crack growth back to determine the intercept at zero load cycles, as shown in the Figure 3-22.

Figure 3-23 shows normalized resilient deformation (δ_H/δ_o) as a function of load repetitions. As explained by Roque et al. (1999) [30], an increase in normalized resilient deformation is directly related to the development of damage in the mixture. When the rate of change of δ_H/δ_o is linear (early in the test), the mixture is undergoing micro-damage development. The initiation of macro-damage (macro-crack) occurs when the rate of change of δ_H/δ_o no longer linear.

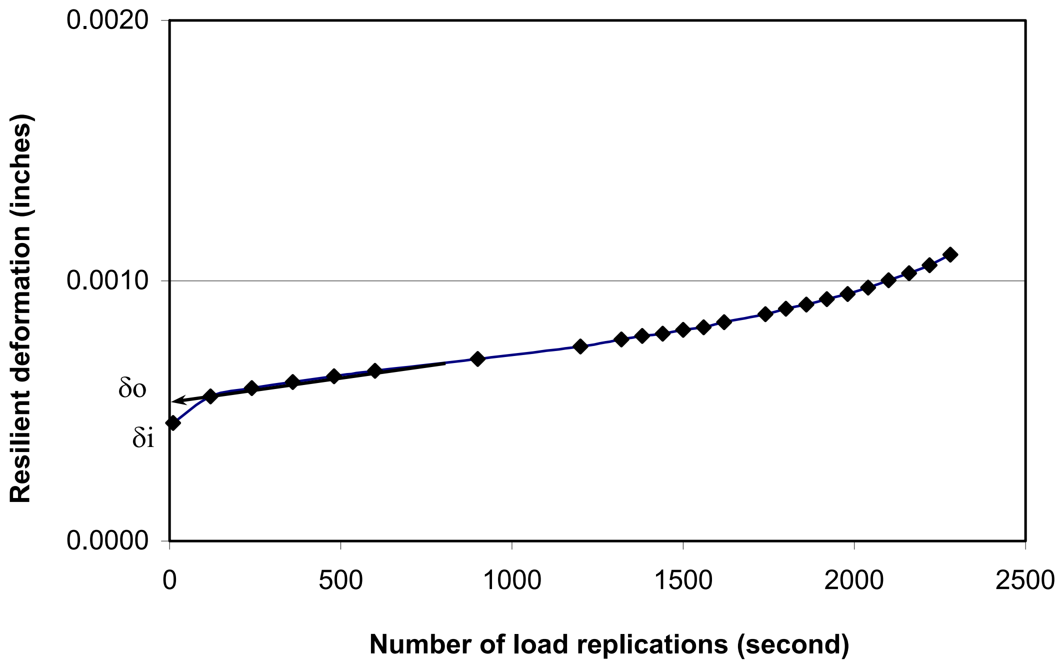


Figure 3-22. Determination of initial resilient deformations (δ_i) and original resilient deformation (δ_o).

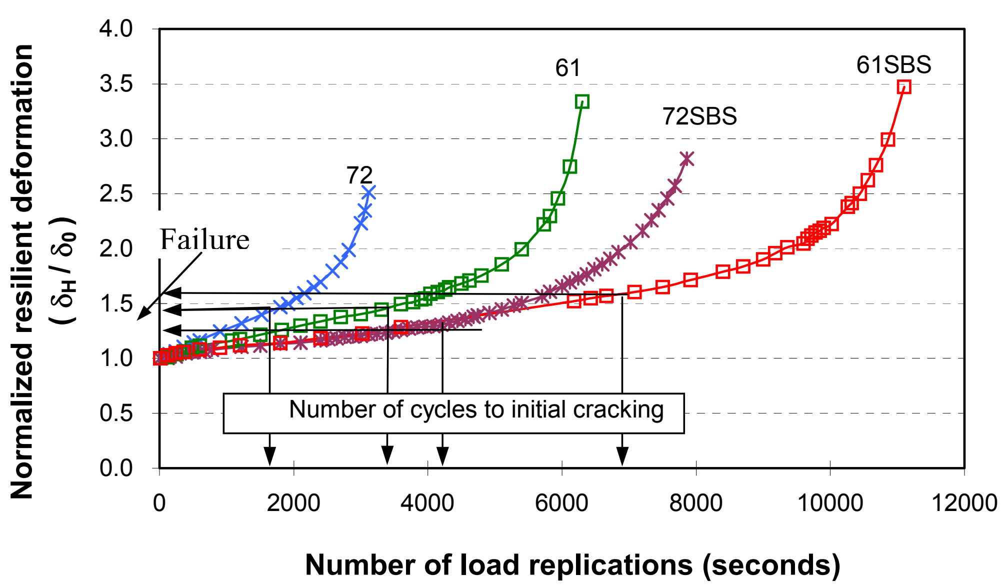


Figure 3-23. Fracture test results (STOA, Coarse 1, and 10^{o}C).

The results presented in Figure 3-23 clearly show that the SBS polymer reduced the rate of micro-damage development and consequently increased the number of load repetitions required for crack initiation. This is consistent with the lower m-value determined for the modified mixtures. The Figure also shows that δ_H/δ_o prior to crack initiation was about the same for all mixtures, modified or unmodified. This is consistent with the fact that the failure limits (FE and $DCSE_f$) were relatively unaffected by the SBS modifier. It should also be noted that the mixtures with lower binder content, which have lower creep than the mixtures with higher binder content (Figure 3-24), exhibited greater resistance to fatigue-type crack growth (modified and unmodified). This trend is consistent with the test results of the long-term oven aged mixture and the fine-graded mixtures, which will be further presented later in this chapter. Therefore, it appears that fatigue-type crack growth observed in fracture test is better represented in the viscoelastic response measured in creep tests than in the critical responses measured in strength tests.

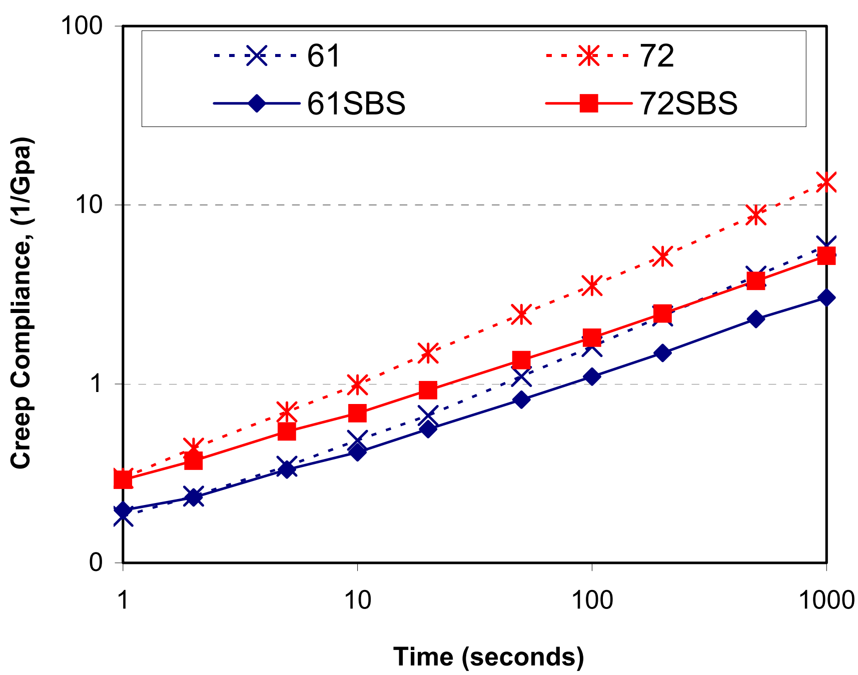


Figure 3-24. Creep compliance of coarse-graded mixtures (10°C, STOA).

3.6.5 Longer-Term Creep Tests to Failure

Research performed in recent years related to the development of an HMA fracture model has clearly indicated that microdamage is directly related to the development of creep, and that the initiation of cracking in asphalt mixture can be defined in terms of the dissipated creep strain energy threshold ($DCSE_f$). This implies that the mechanism of crack initiation and propagation for sub-critical loading conditions is associated with the development of creep, regardless of the loading pattern used to induce creep. Therefore, it was hypothesized that the effects of SBS polymer modification on cracking could be uniquely identified by performing creep tests to crack initiation. The DCSE of creep test was calculated by simply multiplying the creep strain by the stress applied during the test. The results of longer-term creep tests presented in Figure 3-25 clearly show the following:

- SBS modifiers reduced the rate of DCSE accumulation.
- The $DCSE_f$ was about the same for modified and unmodified mixtures, which agrees with the results of strength tests as well as with the idea that the SBS polymer primarily reduced the rate of micro-damage, but does little to increase the threshold energy required to crack the mixture.

These results seem to indicate that the time to crack initiation as determined from this type of test may be suitable for uniquely characterizing the presence and benefit of SBS modification in asphalt mixture.

Figure 3-26 also shows that this test provides an alternative way to determine the $DCSE_f$ of asphalt mixtures. As shown in the Figure, the $DCSE_f$ obtained from creep tests was almost identical to the value determined from independent strength tests performed on the same mixtures. $DCSE_f$ was determined from the creep tests as the energy to the point where the rate of DCSE became nonlinear.

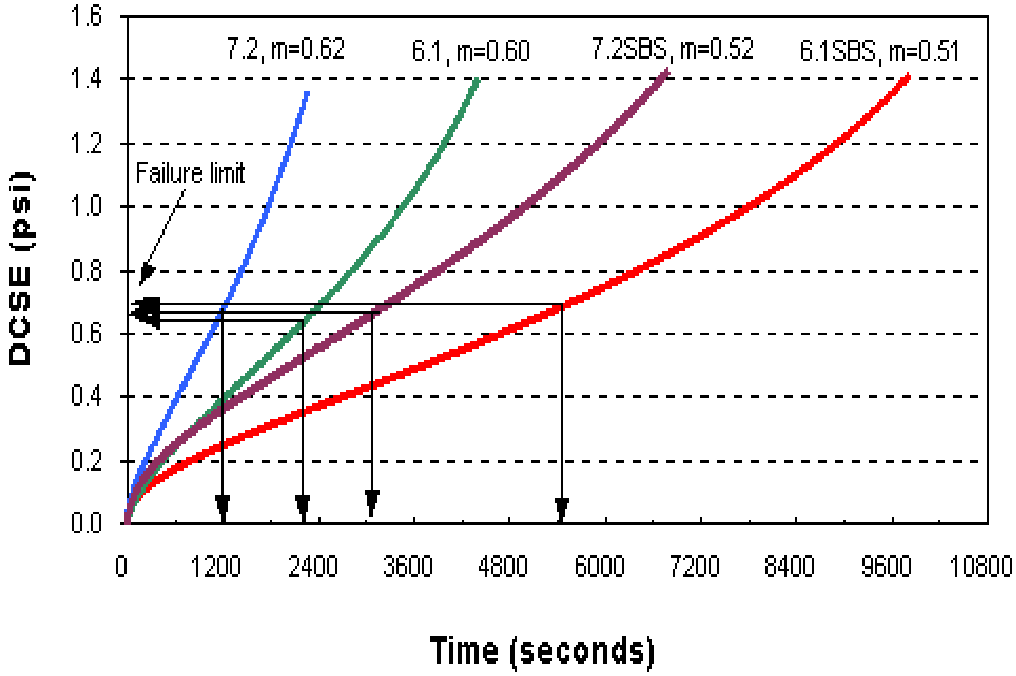


Figure 3-25. Creep test results (DCSE vs. time).

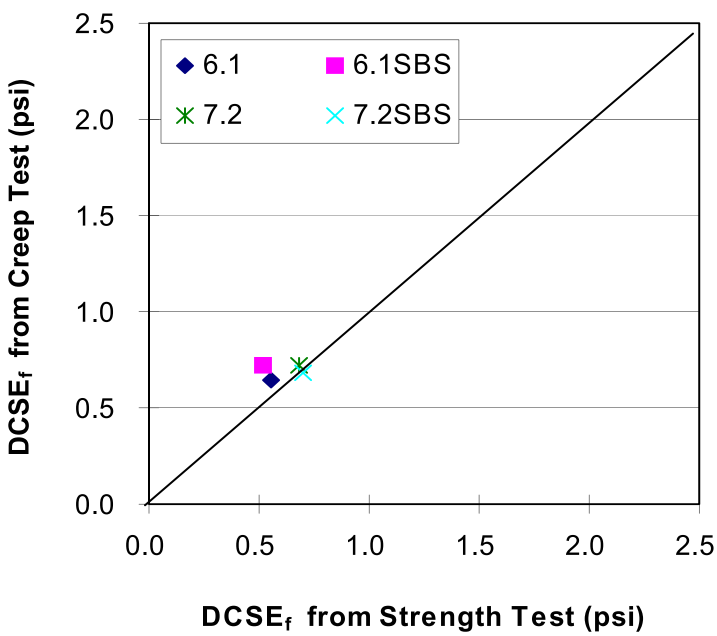


Figure 3-26. Comparison of $DCSE_f$ between creep and strength test.

3.6.6 Summary of Findings

This study was conducted to evaluate the effects of SBS polymer modification on cracking resistance and healing characteristics of Superpave mixtures. The investigation also focused on identifying mixture properties and/or characteristics, as well as specific test methods that can be used to uniquely characterize the presence and beneficial effect of SBS modifiers in asphalt mixtures. The findings of this study may be summarized as follows:

- SBS polymer modification appears to improve the cracking performance of asphalt mixtures by reducing the rate of creep accumulation, which has been shown to be directly related to the rate of micro damage development, without reducing the threshold fracture energy of the mixture. Therefore, one must determine both the creep properties and the fracture energy limit of mixtures to reveal the beneficial effect of SBS polymer modification on cracking performance.

- The reduced rate of creep accumulation in modified mixtures appears to be mainly and perhaps almost exclusively, reflected in a lower m-value.

- The HMA fracture model developed at the University of Florida appears to accurately reflect the beneficial effects of SBS polymer modification on the cracking performance of asphalt mixtures. The model uses creep compliance parameters determined from a 1000-second creep test and the threshold fracture energy determined from a tensile strength test, both of which are performed with the Superpave IDT, to predict crack initiation and growth in asphalt mixtures.

- The relative effect of SBS modifiers was increased at higher binder contents and temperatures. It implies that the effect of SBS polymer would be increased in the mixtures with higher asphalt contents such as open graded friction courses.

3.7 Open Graded Friction Course Study

At the request of Bruce Dietrich, State Pavement Design Engineer, the University of Florida and the FDOT State Materials Office performed a comparison study for the cracking performance of open graded friction courses (OGFC) prepared with ground tire rubber (GTR) modified asphalt and with styrene-butadiene-styrene (SBS) modified asphalt binder. Current practice in Florida requires the use asphalt binder containing 12 percent of ground rubber (ARB 12) for all open graded friction courses. The primary objective of the study was to determine the performance benefits, if any, of changing the OGFC binder requirement to SBS modified binder.

3.7.1 Scope and Research Approach

The FC5 mixtures were prepared with Nova Scotia granite aggregate (NS 315) and the three selected binders – control (PG 67-22), GTR modified (ARB 12), and SBS modified (PG 76-22). The optimum asphalt was determined with the pie-plate method using the PG 67-22 binder and was then adjusted (times 1.12) for the ARB 12. For comparison purposes, a one-to-one substitution was decided for the ARB 12 and PG 76-22 modified binders. All mixtures contained 0.4 percent (by total mixture weight) of mineral fiber.

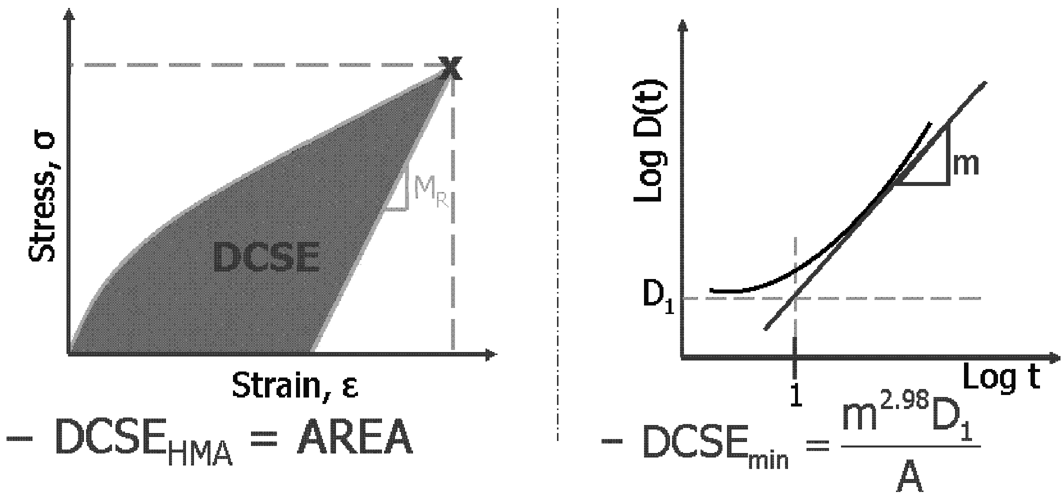
The laboratory mixtures were compacted with the Pine gyratory compactor to 50 gyrations to achieve the target air-void content. The 150-mm diameter compacted specimen were then cut to two-inch-thick specimens for the indirect tension test (IDT). Bulk specific gravity results from the CoreLok device verified the air-void content (16-19%).

Gage points were glued on the dry specimens before they were placed in the testing chamber and soaked at the testing temperature for eight hours prior to testing. The testing was performed at our (UF-FDOT) standard testing temperature of 10°C. The Superpave IDT tests included Resilient Modulus (M_R), Creep Compliance (D_t), and finally a Strength test (S_t). After testing, the acquired data files were compiled and modified for input into the analysis package. The analyzed data was then collected in tabular form and further analyzed en mass to extract a comprehensive assessment of the materials fracture resistance.

3.7.2 Results

The IDT results were reduced with the help of the analysis package to determine the resilient modulus, creep compliance, and strength of each material. The life-expectancy comparison was based on the Energy Ratio (ER) values calculated for each mixture. Energy Ratio is the ratio of the Dissipated Creep Strain Energy of the mixture ($DCSE_{HMA}$) with the minimum DCSE ($DCSE_{MIN}$).

Figure 3-27 illustrates the theory behind the energy ratio calculation. The area under the stress-strain curve is the Fracture Energy (FE) of the material, and the $DCSE_{HMA}$ is the FE minus the Elastic Energy (the slope of the resilient modulus). By curve-fitting the creep compliance data we can get the D_1 and m-value parameters that are used to calculate the $DCSE_{MIN}$ for that particular mixture. The $DCSE_{MIN}$ is defined as the minimum energy that would require 6000 cycles (in the HMA Fracture Mechanics Model) to propagate a crack for 2 inches.



Where:

$$A = \frac{(6.36 - S_t)}{33.44 \times \sigma_t^{3.1}} + 2.46 \times 10^{-8}$$

S_t = Tensile Strength
σ_t = Tensile Stress

Figure 3-27. Energy Ratio calculation.

Prior experience in Florida indicated that poor performance was associated with mixtures with DCSE<0.75 and ER<1. The IDT results (Table 3-20) showed that the PG 76-22 mixture had the highest ER (1.10), followed by the ARB 12 (0.74), and the PG 67-22 with the lowest energy ratio (0.64). The results clearly show the instant benefit of the SBS modified binder by increasing the ER of the mixture by 45% compared to the ARB 12 binder. In terms of increase of life-expectancy, there is not enough information to make such a prediction at this point. However, based on past experience – the evaluation of 22 field sections – the energy ratio concept proved to be an accurate method to determine a mixtures ability to resist cracking.

Table 3-20. Energy Ratio results for the three mixtures.

Mixture	m-value	D_1	Resilient Modulus, M_R (GPa)	Strength, St (MPa)	Fracture Energy, FE (KJ/m^3)	$DCSE_{HMA}$, (KJ/m^3)	$DCSE_{MIN}$, (KJ/m^3)	Energy Ratio, ER
Control	0.7607	8.89E-07	4.690	0.95	4.7	4.60	7.19	0.64
ARB	0.6571	1.72E-06	5.240	1.16	6.9	6.77	9.18	0.74
SBS	0.7141	9.32E-07	4.246	1.17	7.2	7.04	6.38	1.10

CHAPTER 4
COST ANALYSIS FOR MODIFIER USE

4.1 Overview

A parametric study was conducted to analyze the cost effectiveness for use of SBS polymer modified mixture in asphalt pavement. The study consisted of three steps: (1) design of three types of pavement structures (conventional asphalt pavement with crushed stone base, full depth asphalt pavement, and HMA overlay on the conventional asphalt pavement), (2) calculation of energy ratio as a fatigue cracking criterion for designed pavement structures, and (3) cost analysis for pavement structures with and without SBS modified mixture.

4.2 Pavement design

For structural pavement design (layer thickness design), we selected three typical asphalt pavement types – conventional asphalt pavement with crushed stone base, full depth asphalt pavement, and HMA overlay on the conventional asphalt pavement – and three traffic levels – low (<3 million), medium (3-10 million), and high (> 10 million). Based on FDOT traffic data from 1997, low traffic levels (traffic level 1 to 3 of Superpave mix design) cover 39.5% of total estimated design ESALs in Florida, medium traffic levels (traffic level 4 to 5 of Superpave mix design) cover 58.9%, and high traffic levels (traffic level 6 to 7 of Superpave mix design) cover 1.6% of total estimated design ESALs in Florida. For pavement design in this study, three million ESAL was set as upper limit of low traffic levels, 10 million ESAL as average of medium traffic levels, and 30 million ESAL as a low limit of high traffic levels.

For the selected pavement type and traffic levels, the AASHTO design guide was used for the design, and the designed AC layer thickness was checked by AI (Asphalt Institute) method. As input values, asphalt concrete (AC) modulus was determined from the Superpave IDT, and typical moduli of base and subgrade in Florida were selected. Structural coefficient (a_i) and drainage coefficient (m_i) were determined according to the AASHTO design guide as shown in Table 4-1. The following design inputs were assumed in all cases: 95% of reliability (R), 0.4 of standard deviation (S_o), and 2.0 of design serviceability loss (□PSI).

Table 4-1 shows the resulting design layer thickness and the resulting tensile stresses at the bottom of AC layer. The tensile stresses were calculated for each design pavement structure using multi-layer elastic analysis program, BISAR using 9000 (lb) single axle loads with 6 inch radius. These stresses were used for calculating the energy ratio.

4.3 Calculation of Energy Ratio

A parameter, Energy Ratio (ER), which represents the fracture toughness of asphalt mixtures, was recently developed by Roque et al. [31]. This parameter allows for the evaluation of cracking performance for different pavement structures by incorporating the effects of mixture properties and pavement structural characteristics. In this study, the energy ratio was calculated for the design pavement structures as a fatigue cracking criterion. The energy ratio is expressed in Equation (4-1).

Table 4-1. Designed layer thickness and calculated stresses.

				Low Traffic	Medium Traffic	High Traffic
Conventional	Modulus (psi)	a_i	m_i	LAYER THICKNESS (inches)		
AC	1,200,000	0.40	1	6.0	7.0	8.5
Crushed stone base	40,000	0.14	1.2	8.5	10.5	10.0
Subgrade	10,000					
σ_t (psi) at the bottom of AC layer				204.0	165.0	129.0

				Low Traffic	Medium Traffic	High Traffic
HMA Full Depth	Modulus (psi)	a_i	m_i	LAYER THICKNESS (inches)		
AC	1,200,000	0.40	1	10.0	12.0	14.0
Subgrade	10,000					
σ_t (psi) at the bottom of AC layer				119.0	87.6	67.1

				Low Traffic	Medium Traffic	High Traffic
HMA Overlay	Modulus (psi)	a_i	m_i	LAYER THICKNESS (inches)		
AC Overlay	1,200,000	0.40	1	3.0	3.5	4.5
AC	1,200,000	0.40	1	6.0	7.0	8.5
Crushed stone base	40,000	0.14	1.2	8.5	10.5	10.0
Subgrade	10,000					
σ_t (psi) at the bottom of AC layer				121.0	94.7	68.0

$$ER = \frac{DCSE_{HMA}}{DCSE_{MIN}} = \frac{a \times DCSE_f}{m^{2.98} \times D_1} \tag{4-1}$$

Where: $\alpha = 0.00299 \cdot \sigma^{-3.1}(6.36 - S_t) + 2.46 \times 10^{-8}$
σ is the tensile stress of the asphalt layer in psi
S_t is the tensile strength of the asphalt layer in MPa
$DCSE_f$ is the Dissipated Creep Strain Energy in KJ/m^3
D_1 and m are creep parameters in 1/psi

The primary benefit of SBS polymer to mixture cracking resistance is derived from a reduced rate of micro-damage accumulation, which was reflected in a lower m-value for modified mixtures. Thus, by varying the variables in equation (4-1), one can evaluate the effect of modifiers on the ER. The factorial design for the parametric study is shown in Table 4-2. Variables in Table 4-2 were selected based on Superpave IDT test results at 10° C for various mixtures. For example, the typical m-value was 0.6 for unmodified mixtures, and 0.45 for modified mixtures. Also, D_1 values ranged from 14.0 $*10^{-7}$ for a poor-performing mixture to 6.0 $*10^{-7}$ for a good-performing mixture. The ER calculated for each pavement structure and the selected factorial design are presented in Table 4-3 through Table 4-5.

Table 4-2. Factorial design for parametric study.

m-value		0.60			0.45		
D_1*10^{-7} (1/psi)		14.0	10.0	6.0	14.0	10.0	6.0
$DCSE_f$ (KJ/m^3)	1.0						
	2.0						
	3.0						
	4.0						

Jajliardo (2003) [32] recommended a minimum required ER (ER_{min}) for various traffic levels. He recommended an ER_{min} of 1.1 for 3 million ESAL, 1.3 for 10 million ESAL, and 1.7 for 30 million ESAL. Comparing the ER in Table 4-3 with the ER_{min}, most of AC layers in conventional pavement structures with unmodified binder (typically m-value of 0.6) could not meet the ER_{min} for all traffic levels. Even though AC layers modified with polymer (typically m-value of 0.45) having low $DCSE_f$ or high D_1 (generally means low quality mixtures) could not meet the ER_{min}, the modified AC layer with high $DCSE_f$ or low D_1 (generally means high quality mixtures) met the ER_{min} for all traffic levels.

As far as full depth AC pavement structures, the ER of modified AC was enough to meet the criteria, while the ER of unmodified AC layers was not enough to meet the criteria for low traffic level as shown in Table 4-4. However, the ER for medium and high traffic levels was over the ER_{min} regardless of modification, except for the cases of low $DCSE_f$. Very similar trends resulted in HMA overlay pavement structures as shown in Table 4-5.

In summary, when the ER is considered as a criterion of fatigue cracking, it appears that conventional pavements, which consists of AC surface, crushed stone base, and subgrade, designed according to the AASHTO procedure, do not have a sufficient resistance against fatigue cracking, even though SBS polymer modified asphalt mixtures are used in AC layer, except for the pavement structures with a very high quality asphalt mixture, which has a higher $DCSE_f$ and lower creep. Thus, a thicker AC layer than that designed according to the AASHTO procedure is necessary to have the sufficient fatigue cracking resistance. Conversely, unmodified AC layers with higher quality asphalt mixtures (higher $DCSE_f$ and lower creep) and polymer modified AC layers have a sufficient fatigue cracking resistance in full depth and HMA overlay pavement structures, while unmodified AC layers with lower quality asphalt mixtures (lower $DCSE_f$ and higher creep) are not sufficient to tolerate the fatigue cracking.

Table 4-3. Energy ratios calculated for conventional pavement structures.

(a) For low traffic

m-value		0.60			0.45		
D_1*10^{-7} (1/psi)		14.0	10.0	6.0	14.0	10.0	6.0
$DCSE_f$ (KJ/m^3)	1.0	0.11	0.16	0.26	0.26	0.37	0.61
	2.0	0.22	0.31	0.52	0.52	0.73	1.22
	3.0	0.33	0.47	0.78	0.78	1.10	1.83
	4.0	0.44	0.62	1.03	1.04	1.46	2.44

(b) For medium traffic level

m-value		0.60			0.45		
D_1*10^{-7} (1/psi)		14.0	10.0	6.0	14.0	10.0	6.0
$DCSE_f$ (KJ/m^3)	1.0	0.14	0.19	0.32	0.33	0.46	0.76
	2.0	0.28	0.39	0.65	0.65	0.92	1.53
	3.0	0.42	0.58	0.97	0.98	1.37	2.29
	4.0	0.56	0.78	1.30	1.31	1.83	3.05

(c) For high traffic level

m-value		0.60			0.45		
D_1*10^{-7} (1/psi)		14.0	10.0	6.0	14.0	10.0	6.0
$DCSE_f$ (KJ/m^3)	1.0	0.21	0.29	0.48	0.48	0.68	1.13
	2.0	0.41	0.58	0.96	0.97	1.36	2.26
	3.0	0.62	0.86	1.44	1.45	2.04	3.39
	4.0	0.82	1.15	1.92	1.94	2.71	4.52

Table 4-4. Energy ratios calculated for full depth AC pavement.

(a) For low traffic level

m-value		0.60			0.45		
D_1*10^{-7} (1/psi)		14.0	10.0	6.0	14.0	10.0	6.0
$DCSE_f$ (KJ/m^3)	1.0	0.24	0.34	0.56	0.57	0.80	1.33
	2.0	0.48	0.68	1.13	1.14	1.59	2.65
	3.0	0.72	1.01	1.69	1.70	2.39	3.98
	4.0	0.96	1.35	2.25	2.27	3.18	5.30

(b) For medium traffic level

m-value		0.60			0.45		
D_1*10^{-7} (1/psi)		14.0	10.0	6.0	14.0	10.0	6.0
$DCSE_f$ (KJ/m^3)	1.0	0.50	0.69	1.16	1.17	1.64	2.73
	2.0	0.99	1.39	2.31	2.34	3.27	5.45
	3.0	1.49	2.08	3.47	3.50	4.91	8.18
	4.0	1.98	2.78	4.63	4.67	6.54	10.90

(c) For high traffic level

m-value		0.60			0.45		
D_1*10^{-7} (1/psi)		14.0	10.0	6.0	14.0	10.0	6.0
$DCSE_f$ (KJ/m^3)	1.0	1.03	1.44	2.40	2.43	3.40	5.66
	2.0	2.06	2.88	4.80	4.85	6.79	11.32
	3.0	3.09	4.32	7.20	7.28	10.19	16.98
	4.0	4.12	5.76	9.61	9.70	13.58	22.64

Table 4-5. Energy ratios calculated for HMA overlay.

(a) For low traffic level

m-value		0.60			0.45		
D_1*10^{-7} (1/psi)		14.0	10.0	6.0	14.0	10.0	6.0
$DCSE_f$ (KJ/m^3)	1.0	0.23	0.33	0.54	0.55	0.77	1.28
	2.0	0.47	0.65	1.09	1.10	1.54	2.56
	3.0	0.70	0.98	1.63	1.65	2.31	3.84
	4.0	0.93	1.31	2.18	2.20	3.08	5.13

(b) For medium traffic level

m-value		0.60			0.45		
D_1*10^{-7} (1/psi)		14.0	10.0	6.0	14.0	10.0	6.0
$DCSE_f$ (KJ/m^3)	1.0	0.41	0.57	0.95	0.96	1.34	2.24
	2.0	0.81	1.14	1.90	1.92	2.68	4.47
	3.0	1.22	1.71	2.85	2.87	4.02	6.71
	4.0	1.63	2.28	3.79	3.83	5.37	8.94

(c) For high traffic level

m-value		0.60			0.45		
D_1*10^{-7} (1/psi)		14.0	10.0	6.0	14.0	10.0	6.0
$DCSE_f$ (KJ/m^3)	1.0	0.99	1.39	2.31	2.34	3.27	5.45
	2.0	1.98	2.77	4.62	4.67	6.54	10.90
	3.0	2.97	4.16	6.94	7.01	9.81	16.35
	4.0	3.96	5.55	9.25	9.34	13.08	21.80

4.4 Cost analysis

A parametric study was conducted to compare the construction costs of AC layers with and without polymer modifier. Based on the FDOT Item Average Unit Costs (item no. 2334 and 2337), the unit cost of unmodified HMA and polymer modified HMA were assumed at 50 dollars per ton and 70 dollars per ton, respectively. Currently (2005) the price premium for an SBS-modified mix ranges from \$6 to \$10 per ton; well below the \$20 price difference assumed for this study. Consequently, the results of this cost analysis based on current prices further benefit the polymer-modified mixtures. The cost analysis was performed for two cases as follows:

- Calculated ER ≥ Minimum Required ER
- Calculated ER of Unmodified HMA ≥ Calculated ER of Modified HMA

4.4.1 Case 1 ($ER_{HMA} \geq ER_{min}$)

In this case, the minimum required ER (ER_{min}) was used as a criterion to determine the construction cost. The ER_{min} criteria were first presented by Jajliardo [32] for various traffic levels – 1.1 for three million ESAL, 1.3 for 10 million ESAL, and 1.7 for 30 million ESAL. When the calculated ER, which is based on the thickness designed according to the AASHTO procedure, is less than the ER_{min}, the ER was increased to meet the ER_{min} by increasing the thickness of AC layer resulting in the decreased tensile stress at the bottom of AC layer. Otherwise, the designed thickness was use to calculate the construction cost. In other words, the design thickness of AC layer meeting the ER_{min} was maintained, regardless of redundant margin between the ER_{min} and the ER calculated, since the thickness designed according to AASHTO procedure was assumed as the minimum allowable thickness.

Table 4-6 shows the cost of AC layer meeting the ER_{min} for conventional pavement structures. AC layer thickness and resulting tensile stress meeting the ER_{min} are presented in Appendix A. Table 4-7 and Table 4-8 show the cost of AC layer for HMA full depth and HMA overlay pavement structures, respectively. AC layer thickness and resulting tensile stress are also presented in Appendix A.

As shown in Table 4-6, the cost was reduced by up to 30% for conventional pavements by using polymers. However, the cost reduction was decreased as traffic level increased. Therefore, the result indicates that if AC layer of conventional asphalt pavement should have the minimum fatigue cracking toughness, using SBS modifiers in AC layer, as compared with unmodified AC layer, can reduce the construction cost by around 5% to 30%, in depending on traffic level. On the other hand, in some cases of, presented in Table 4-6 (for example, D_1 is 6 and $DCSE_f$ is 4 for low traffic), the cost of modified AC layer was increased. This increased cost is because the design thickness determined in Table 4-1 is used as a minimum thickness in this analysis.

Conversely, Table 4-7 shows that by using modifiers, the cost of AC layer was decreased by 10% for HMA full depth pavement with mixtures having lower $DCSE_f$ and higher D_1 (lower quality asphalt mixtures). However, cost for modified mixtures increased by 8% for higher $DCSE_f$ and lower D_1 (higher quality asphalt mixtures). It appears that for thicker AC layers, as in full-depth HMA pavements, the cost savings due to thickness reduction is relatively small compared to the cost of mixture modification. Therefore, the benefits of SBS-modification are less significant for full-depth pavement structures compared to traditional layer design.

As far as the HMA overlay shown in Table 4-8, the variation of the cost difference was from 30% of reduction in AC layers with lower $DCSE_f$ and higher D_1 (lower quality asphalt mixtures) to 27% of increase in AC layers with higher $DCSE_f$ and lower D_1 (higher quality asphalt mixtures). Therefore, in the pavement with a thick AC layer such as HMA full depth and HMA overlay, it appears that the cost effectiveness largely depends on the quality of asphalt mixtures. In other words, if a relatively lower quality asphalt mixture is used, the construction cost of AC layer can be largely saved by using modifiers.

Table 4-6. Cost of AC layer to meet ER_{min} in conventional pavement

(a) Low traffic (unit: $\$/m^2/in$)

m-value		0.60 (unmodified)			0.45 (modified)		
D_1*10^{-7} (1/psi)		14	10	6	14	10	6
$DCSE_f$ (KJ/m^3)	1.0	37.8	35.3	31.0	33.3	30.5	26.2
	2.0	32.4	29.6	25.7	27.6	24.5	19.2
	3.0	29.3	27.1	21.7	23.7	19.2	19.2
	4.0	26.8	24.0	18.0	20.0	19.2	19.2

(b) Medium traffic

m-value		0.60 (unmodified)			0.45 (modified)		
D_1*10^{-7} (1/psi)		14	10	6	14	10	6
$DCSE_f$ (KJ/m^3)	1.0	38.6	36.7	32.4	34.4	31.6	27.4
	2.0	33.3	31.0	26.8	29.1	25.7	22.0
	3.0	29.6	27.6	23.1	25.4	22.0	22.0
	4.0	27.9	25.1	19.7	22.0	22.0	22.0

(c) High traffic

m-value		0.60 (unmodified)			0.45 (modified)		
D_1*10^{-7} (1/psi)		14	10	6	14	10	6
$DCSE_f$ (KJ/m^3)	1.0	40.9	38.1	34.1	37.2	33.8	29.6
	2.0	35.3	32.7	28.8	31.0	28.2	26.2
	3.0	32.4	29.6	25.4	27.6	26.2	26.2
	4.0	29.9	27.4	24.0	26.2	26.2	26.2

Table 4-7. Cost of AC layer to meet ER_{min} in HMA full depth pavement

(unit: $/m^2/in)

(a) Low traffic

m-value		0.60 (unmodified)			0.45 (modified)		
D_1*10^{-7} (1/psi)		14	10	6	14	10	6
$DCSE_f$ (KJ/m^3)	1.0	40.0	37.5	33.6	35.5	32.7	30.5
	2.0	34.7	31.9	28.2	30.5	30.5	30.5
	3.0	31.6	29.0	28.2	30.5	30.5	30.5
	4.0	29.3	28.2	28.2	30.5	30.5	30.5

(b) Medium traffic

m-value		0.60 (unmodified)			0.45 (modified)		
D_1*10^{-7} (1/psi)		14	10	6	14	10	6
$DCSE_f$ (KJ/m^3)	1.0	41.5	38.6	34.7	36.9	36.1	36.1
	2.0	35.8	33.8	33.8	36.1	36.1	36.1
	3.0	33.8	33.8	33.8	36.1	36.1	36.1
	4.0	33.8	33.8	33.8	36.1	36.1	36.1

(c) High traffic

m-value		0.60 (unmodified)			0.45 (modified)		
D_1*10^{-7} (1/psi)		14	10	6	14	10	6
$DCSE_f$ (KJ/m^3)	1.0	43.7	40.9	39.5	41.7	41.7	41.7
	2.0	39.5	39.5	39.5	41.7	41.7	41.7
	3.0	39.5	39.5	39.5	41.7	41.7	41.7
	4.0	39.5	39.5	39.5	41.7	41.7	41.7

Table 4-8. Cost of AC layer to meet ER_{min} in HMA overlay pavement

(unit: $\$/m^2/in$)

(a) Low traffic

m-value		0.60 (unmodified)			0.45 (modified)		
D_1*10^{-7} (1/psi)		14	10	6	14	10	6
DCSE$_f$ (KJ/m^3)	1.0	20.9	18.3	14.1	16.4	13.5	10.7
	2.0	15.5	12.7	8.7	10.7	10.7	10.7
	3.0	12.4	10.2	8.5	10.7	10.7	10.7
	4.0	9.9	8.5	8.5	10.7	10.7	10.7

(b) Medium traffic

m-value		0.60 (unmodified)			0.45 (modified)		
D_1*10^{-7} (1/psi)		14	10	6	14	10	6
DCSE$_f$ (KJ/m^3)	1.0	18.9	16.9	12.7	14.7	12.1	12.1
	2.0	13.5	11.3	9.9	12.1	12.1	12.1
	3.0	9.9	9.9	9.9	12.1	12.1	12.1
	4.0	9.9	9.9	9.9	12.1	12.1	12.1

(c) High traffic

m-value		0.60 (unmodified)			0.45 (modified)		
D_1*10^{-7} (1/psi)		14	10	6	14	10	6
DCSE$_f$ (KJ/m^3)	1.0	16.9	14.1	12.7	15.0	15.0	15.0
	2.0	12.7	12.7	12.7	15.0	15.0	15.0
	3.0	12.7	12.7	12.7	15.0	15.0	15.0
	4.0	12.7	12.7	12.7	15.0	15.0	15.0

4.4.2 Case 2 ($ER_{HMA} \geq ER_{PM\text{-}HMA}$)

In this case, the ER of polymer modified HMA (ER_{PMHMA}) was used as the criterion to determine the construction cost of AC layer. The typical m-value of polymer modified and straight asphalt mixture is 0.45 and 0.6, respectively as presented in the factorial design. As shown in Table 4-3 through Table 4-5, the resulting $ER_{PM\text{-}HMA}$ is higher than that of the ER of unmodified HMA (ER_{HMA}) at the same conditions. Therefore, in this case, the ER_{HMA} was increased to meet the $ER_{PM\text{-}HMA}$ by increasing the thickness of AC layer resulting in the decreased tensile stress at the bottom of AC layer, which gives an equivalent fatigue cracking resistance of polymer modified AC layer.

Table 4-9 shows the cost of unmodified AC layer and polymer modified AC layer having the equivalent ER. The cost of polymer modified AC layer was calculated in two cases to identify how much cost was reduced by varying the thickness of modified mixture: (1) the case where top two inches of AC layer was constructed with polymer modified mixture, and (2) the case where whole AC layer was constructed with polymer modified mixture. AC layer thickness and resulting tensile stresses meeting the ER of modified AC layer are presented in Appendix A.

As shown in Table 4-9, there is little difference of cost in conventional pavement structures. The cost of modified AC layer (full depth replacement) is 12% lower for low traffic level and 8% higher for high traffic level than those of unmodified AC layer. However, in HMA full depth pavement structures, the cost of modified AC layer (full depth replacement) is 14% to 19% higher for all traffic levels than those of unmodified AC layer, and in HMA overlay structures, polymer modified mixture resulted in lower cost (3% to 24% lower cost of modified AC layer than those of unmodified AC layer). These results are due to the thickness of AC layer. That is, the thinner the AC layer, the higher the cost reduction by applying polymer modifier. These results indicate that there is a break point below which the construction cost of the AC layer can be saved by using modifiers. As shown in Table 4-1 and Table 4-9, the positive cost reduction appears to happen below 7 inches of AC layer thickness in conventional pavement structures, and 4.5 inches of HMA overlay thickness.

Finally, another cost comparison was performed to identify how much cost was reduced by varying the thickness of modified mixtures. That is, the case when the top two inches of AC layer is replaced by the modified mixture and the pavement still has an equivalent ER. The top two inches of AC layer were selected, since top-down cracking is the most prevalent type of cracking and two inches is the typical length of cracks in Florida. As shown in Table 4-9, the construction cost was reduced for all cases and up to 30% cost reduction was induced by the use of polymer in this case.

Table 4-9. Costs of AC layer with the equivalent ER

(Unit: $)

		Low Traffic	Medium Traffic	High Traffic
Conventional Structure	Unmodified	26.8	28.2	31.0
	Modified (2" replacement)	19.2	22.0	26.2
	Modified (Full depth replacement)	23.7	27.7	33.6
HMA Full Depth	Unmodified	34.7	40.6	46.5
	Modified (2" replacement)	30.5	36.1	41.7
	Modified (Full depth replacement)	39.5	47.4	55.3
HMA Overlay	Unmodified	15.5	16.9	18.3
	Modified (2" replacement)	10.7	12.1	15.0
	Modified (Full depth replacement)	11.9	13.8	17.8

CHAPTER 5
SUMMARY AND RECOMMENDATIONS

5.1 Summary

It may not be possible to produce Superpave mixtures with conventional asphalt cement for certain levels of traffic and environment that to have both adequate rutting and cracking resistance. Mixtures designed for high traffic may be susceptible to cracking due to the lower design asphalt content. One way to achieve sufficient fracture resistance is through the use of asphalt modifiers. Styrene Butadiene Styrene (SBS) and Ground Tire Rubber (GTR) polymer modifiers have become increasingly popular because of their apparent success in mitigating rutting and cracking in the field. However, the specific effects of polymer modified asphalts on mixture performance of Superpave mixtures are not clear yet. This report serves as a summary of the work done to evaluate the effects of SBS and GTR polymer modification on mixture performance; the main findings are summarized below.

5.1.1 Modifiers

- GTR has been used in open and dense graded friction courses. The basic benefit of GTR is that it can increase binder content without drain-down; the increased binder content subsequently improves cracking resistance.
- For dense-graded mixtures (tested with 12% rubber) GTR creates problem with the aggregate structure; it prevents the aggregate structure of achieving maximum shear strength orientation. Hybrid binder should be explored as a potential candidate for dense graded mixtures.

- Based on literature, most polymer evaluations focused on rutting performance, where SBS-modified mixtures performed better relative to conventional binder and other modifiers which reported premature cracking – EVA, Gilsonite, and Lake Asphalt.

- FDOT, recognizing the benefits of SBS modified binder, now specifies SBS for locations that there has been a history of rutting.

<u>5.1.2 Performance in Florida</u>

- Field data is mostly anecdotal, not involving controlled scientific studies. In most cases observations indicate good performance relative to prior history in existing location.

- HVS experiment clearly indicated SBS-modified binder (PG 76-22) out-performed the control binder (PG 67-22) in rutting performance.

- Laboratory: APA, GTM, Servopac, have all clearly illustrated the benefit in rutting performance.

- HMA fracture model (Superpave IDT) showed that the SBS modified mixtures could benefit cracking, mainly because of reduced creep rate.

- However, SBS modifiers do not have any effect on stiffness or the DCSE of the material. A short loading time test, including complex modulus, will not be able to capture the benefits of the modifier.

<u>5.1.3 Cost Benefit</u>

- The added cost of SBS-modified binder amounts to $100 per ton of liquid binder that results to $6 to $8 per ton of HMA (10 to 15% price increase in total cost). This may be reduced with continued use; case by case scenario results in higher costs because contractors need to use different storing tanks with agitators, purchase the binder in smaller quantities, etc.

- Based on the HMA Fracture Model, for pavements with sufficient structure (i.e. HMA thickness not required for SN, or assuming SN obtained from base) then SBS reduces required thickness that results to 5-30% reduction in initial cost depending on traffic level; not considering improved life-cycle cost.

<u>5.1.4 Reported Construction Considerations</u>

- Absence of tender zone may increase plant production by 30 to 40 %.

- However, for highly absorptive aggregate, production may be reduced by 40% for extra drying time (moisture less able to escape polymer coated aggregate); this may need more careful investigation.

- Higher temperatures in production (325° F).

5.2 Recommendations

- Proceed with controlled field studies for rutting and cracking.
- Need to identify (develop) a design procedure that accurately reflects benefits of PMA; complex modulus will not be able to capture the SBS modifier benefits.
- Further investigate production issues, especially the part with absorptive aggregate.
- Use of SBS modified asphalts appears warranted and cost effective. High traffic volume Superpave mixtures result in lower asphalt binder content because of the increased design gyrations. If sufficient structure is present, for traffic level D or higher, use of SBS-modified asphalt is recommended.
- SBS-modified asphalt is highly recommended for intersections (high volume, slow moving traffic) and OGFC; however, environmental benefit of use of rubber in pavements cannot be overlooked. Project to investigate a hybrid binder – combination of SBS polymer with rubber – is already under way.

APPENDIX A
AC LAYER THICKNESS FOR COST ANALYSIS

Table A-1. Tensile stress at the bottom of AC layer to meet ER_{min} for conventional pavement.

(a) Low traffic ERmin = 1.1

m-value		0.60			0.45		
D_1*10^{-7} (1/psi)		14	10	6	14	10	6
$DCSE_f$ (KJ/m^3)	1.0	33.3	37.2	44.0	30.3	33.8	40.0
	2.0	17.1	19.0	22.4	15.5	17.3	20.4
	3.0	11.5	12.9	15.2	10.5	11.7	13.8
	4.0	8.8	9.8	11.5	8.0	8.9	10.5

(b) Medium traffic ERmin = 1.3

m-value		0.60			0.45		
D_1*10^{-7} (1/psi)		14	10	6	14	10	6
$DCSE_f$ (KJ/m^3)	1.0	31.5	35.2	41.6	28.7	32.0	37.9
	2.0	16.2	18.0	21.3	14.7	16.4	19.4
	3.0	10.9	12.2	14.4	10.0	11.1	13.1
	4.0	8.3	9.3	10.9	7.6	8.4	9.9

(c) High traffic ERmin = 1.7

m-value		0.60			0.45		
D_1*10^{-7} (1/psi)		14	10	6	14	10	6
$DCSE_f$ (KJ/m^3)	1.0	28.9	32.3	38.1	26.3	29.4	34.7
	2.0	14.8	16.5	19.5	13.5	15.1	17.8
	3.0	10.0	11.2	13.2	9.1	10.2	12.0
	4.0	7.6	8.5	10.0	6.9	7.7	9.1

Table A-2. AC layer thickness to meet ER_{min} for conventional pavement.

(a) Low traffic

m-value		0.60			0.45		
D_1*10^{-7} (1/psi)		14	10	6	14	10	6
$DCSE_f$ (KJ/m^3)	1	13.4	12.5	11.0	11.0	10.0	8.5
	2	11.5	10.5	9.1	9.0	7.9	5.3
	3	10.4	9.6	7.7	7.6	6.5	6.0
	4	9.5	8.5	6.4	6.3	6.0	6.0

(b) Medium traffic

m-value		0.60			0.45		
D_1*10^{-7} (1/psi)		14	10	6	14	10	6
$DCSE_f$ (KJ/m^3)	1	13.7	13.0	11.5	11.4	10.4	8.9
	2	11.8	11.0	9.5	9.5	8.3	7.0
	3	10.5	9.8	8.2	8.2	7.0	7.0
	4	9.9	8.9	7.0	7.0	7.0	7.0

(c) High traffic

m-value		0.60			0.45		
D_1*10^{-7} (1/psi)		14	10	6	14	10	6
$DCSE_f$ (KJ/m^3)	1	14.5	13.5	12.1	12.4	11.2	9.7
	2	12.5	11.6	10.2	10.2	9.2	8.5
	3	11.5	10.5	9.0	9.0	8.5	8.5
	4	10.6	9.7	8.5	8.5	8.5	8.5

Table A-3. AC layer thickness to meet ER_{min} for HMA full depth pavement.

(a) Low traffic

m-value		0.60			0.45		
D_1*10^{-7} (1/psi)		14	10	6	14	10	6
$DCSE_f$ (KJ/m^3)	1	14.2	13.3	11.9	11.8	10.8	10.0
	2	12.3	11.3	10.0	10.0	10.0	10.0
	3	11.2	10.3	10.0	10.0	10.0	10.0
	4	10.4	10.0	10.0	10.0	10.0	10.0

(b) Medium traffic

m-value		0.60			0.45		
D_1*10^{-7} (1/psi)		14	10	6	14	10	6
$DCSE_f$ (KJ/m^3)	1	14.7	13.7	12.3	12.3	12.0	12.0
	2	12.7	12.0	12.0	12.0	12.0	12.0
	3	12.0	12.0	12.0	12.0	12.0	12.0
	4	12.0	12.0	12.0	12.0	12.0	12.0

(c) High traffic

m-value		0.60			0.45		
D_1*10^{-7} (1/psi)		14	10	6	14	10	6
$DCSE_f$ (KJ/m^3)	1	15.5	14.5	14.0	14.0	14.0	14.0
	2	14.0	14.0	14.0	14.0	14.0	14.0
	3	14.0	14.0	14.0	14.0	14.0	14.0
	4	14.0	14.0	14.0	14.0	14.0	14.0

Table A-4. AC layer thickness to meet ER_{min} for HMA overlay.

(a) Low traffic

m-value		0.60			0.45		
D_1*10^{-7} (1/psi)		14	10	6	14	10	6
$DCSE_f$ (KJ/m^3)	1	7.4	6.5	5.0	5.0	4.0	3.0
	2	5.5	4.5	3.1	3.0	3.0	3.0
	3	4.4	3.6	3.0	3.0	3.0	3.0
	4	3.5	3.0	3.0	3.0	3.0	3.0

(b) Medium traffic

m-value		0.60			0.45		
D_1*10^{-7} (1/psi)		14	10	6	14	10	6
$DCSE_f$ (KJ/m^3)	1	6.7	6.0	4.5	4.4	3.5	3.5
	2	4.8	4.0	3.5	3.5	3.5	3.5
	3	3.5	3.5	3.5	3.5	3.5	3.5
	4	3.5	3.5	3.5	3.5	3.5	3.5

(c) High traffic

m-value		0.60			0.45		
D_1*10^{-7} (1/psi)		14	10	6	14	10	6
$DCSE_f$ (KJ/m^3)	1	6.0	5.0	4.5	4.5	4.5	4.5
	2	4.5	4.5	4.5	4.5	4.5	4.5
	3	4.5	4.5	4.5	4.5	4.5	4.5
	4	4.5	4.5	4.5	4.5	4.5	4.5

Table A-5. AC layer thickness and tensile stress of unmodified AC layer to meet ER of modified mixture.

			TRAFFIC 1	TRAFFIC 2	TRAFFIC 3
Conventional	Modulus(psi)	a & m	AC layer thickness to meet $ER_{PM\text{-}HMA}$		
AC (unmodified)	1,200,000	0.40	9.5	10.0	11.0
CRUSHED STONE BASE	40,000	0.14 & 1.2	8.5	10.5	10.0
SUBGRADE	10,000				
σ_t (psi) at the bottom of AC layer			114.6	103.6	88.4

			TRAFFIC 1	TRAFFIC 2	TRAFFIC 3
Full Depthl	Modulus(psi)	a & m	AC layer thickness to meet $ER_{PM\text{-}HMA}$		
AC (unmodified)	1,200,000	0.40	12.3	14.4	16.5
SUBGRADE	10,000				
σ_t (psi) at the bottom of AC layer			83.2	64.2	50.1

			TRAFFIC 1	TRAFFIC 2	TRAFFIC 3
HMA Overlay	Modulus(psi)	a & m	AC layer thickness to meet $ER_{PM\text{-}HMA}$		
AC (unmodified) Overlay	1,200,000	0.40	5.5	6.0	6.5
AC (unmodified)	1,200,000	0.40	6.0	7.0	8.5
CRUSHED STONE BASE	40,000	0.14 & 1.2	8.5	10.5	10.0
SUBGRADE	10,000				
σ_t (psi) at the bottom of AC layer			84.2	68.8	50.8

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