

Final Report

**GUIDELINES FOR USE OF MODIFIERS
IN SUPERPAVE MIXTURES**

**VOLUME 3 OF 3 VOLUMES:
EVALUATION OF GROUND-TIRE-RUBBER (GTR)**

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16. Abstract <p>A laboratory investigation was conducted to evaluate the effect of ground-tire-rubber (GTR) on Superpave mixes as part of an ongoing research project to investigate the effects of asphalt modifiers on Superpave mixes. This report evaluates the effects of GTR on Superpave™ mixes, and its possible usage in dense graded structural layers.</p> <p>The laboratory investigation was conducted with GTR passing mesh size #80 with AC-30 as base binder. Aggregates used include white rock and Cabbage Grove limestone. Hot Mix Asphalt (HMA) mixes were produced for three traffic levels (low, intermediate, and high) using the Superpave mix design procedure. The samples were tested with the Servopac gyratory compactor and also the Superpave Indirect Tensile Test method. Parameters of the samples that were tested and evaluated include the resilient modulus, indirect tensile strength, creep compliance, fracture energy density, and failure strain.</p> <p>The test results indicate that the addition of GTR to asphalt mixes seems to improve its resistance to permanent deformation and also permits the addition of more binder without substantial loss in the shear resistance of the HMA mixes, and also without binder drain-down, which sometimes occurs during the construction of HMA pavements. However, the addition of GTR reduces the fracture energy and failure strain at intermediate temperatures, which seems to suggest that the GTR reduces the cracking performance (or resistance) of HMA mixes.</p>					
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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol	When You Know	Multiply By	To Find	Symbol
LENGTH								
in	inches	25.4	millimeters	mm	millimeters	0.039	inches	in
ft	feet	0.305	meters	m	meters	3.28	feet	ft
yd	yards	0.914	meters	m	meters	1.09	yards	yd
mi	miles	1.61	kilometers	km	kilometers	0.621	miles	mi
AREA								
in ²	square inches	645.2	square millimeters	mm ²	square millimeters	0.0016	square inches	in ²
ft ²	square feet	0.093	square meters	m ²	square meters	10.764	square feet	ft ²
yd ²	square yards	0.836	square meters	m ²	square meters	1.195	square yards	yd ²
ac	acres	0.405	hectares	ha	hectares	2.47	acres	ac
mi ²	square miles	2.59	square kilometers	km ²	square kilometers	0.386	square miles	mi ²
VOLUME								
fl oz	fluid ounces	29.57	milliliters	ml	milliliters	0.034	fluid ounces	fl oz
gal	gallons	3.785	liters	l	liters	0.264	gallons	gal
ft ³	cubic feet	0.028	cubic meters	m ³	cubic meters	35.71	cubic feet	ft ³
yd ³	cubic yards	0.765	cubic meters	m ³	cubic meters	1.307	cubic yards	yd ³
NOTE: Volumes greater than 1000 l shall be shown in m ³ .								
MASS								
oz	ounces	28.35	grams	g	grams	0.035	ounces	oz
lb	pounds	0.454	kilograms	kg	kilograms	2.202	pounds	lb
T	short tons (2000 lb)	0.907	megagrams	Mg	megagrams	1.103	short tons (2000 lb)	T
TEMPERATURE (exact)								
°F	Fahrenheit temperature	5(F-32)/9 or (F-32)/1.8	Celsius temperature	°C	Celsius temperature	1.8C + 32	Fahrenheit temperature	°F
ILLUMINATION								
fc	foot-candles	10.76	lux	lx	lux	0.0929	foot-candles	fc
fl	foot-Lamberts	3.426	candela/m ²	cd/m ²	candela/m ²	0.2919	foot-Lamberts	fl
FORCE and PRESSURE or STRESS								
lbf	poundforce	4.45	newtons	N	newtons	0.225	poundforce	lbf
psi	poundforce per square inch	6.89	kilopascals	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)

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CHAPTER 1 INTRODUCTION

1.1 Background

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 budget of the SHRP was channeled to the development of performance based asphalt mix specifications with direct correlation between laboratory analysis and field performance (Asphalt Institute). Superpave™ (*Superior Performing Asphalt Pavements*) 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 as being implemented today is the Level 1 mix design (a purely volumetric design procedure), it has the following major advantages over the traditional Marshall and Hveem mix design procedures:

- Additional requirements that essentially eliminates the use of substandard or unacceptable aggregates;
- Specifications and/or selection of binders using fundamental properties that incorporates or takes into account a broader range of in-service temperatures of asphalt pavements;
- Gyrotory compaction that closely simulates field compaction and traffic conditions.

In spite of the above numerated advantages above, experience gained over the past few years in the use of Superpave™ design procedure has indicated that mixtures produced with conventional asphalts 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 for such road facilities. The above observation is a direct result of the Superpave design procedure since the higher the number of gyrations (which simulates higher traffic volume) the lower the design

asphalt content, thus increasing its susceptibility to cracking at low in-service temperatures. Furthermore, recent work on an on-going Florida Department of Transportation (FDOT) research project to develop methods to evaluate the fracture resistance of asphalt mixtures has indicated that coarse-graded Superpave™ mixtures are difficult to compact and may result in pavements with relatively high permeability. The combination of high permeability and low asphalt content indicate that such mixtures may also have low fracture resistance (Roque et al. 1997).

The above observations points to the fact that producing Superpave™ mixtures with conventional asphalt cement for certain levels of traffic and environment to have both adequate rutting and cracking resistance may not be possible. One way to achieve the above objective of producing a mix with desirable rutting resistance, yet having sufficient fracture resistance at low in-service temperatures is by the use of asphalt additives or modifiers.

1.2 Study Objectives

The primary objectives of the research are as follows:

1. To evaluate the existing Superpave™ mixture design procedure (i.e., Level 1 mix design) to identify situations in which the design procedure would achieve rutting resistance but low or undesirable fracture resistance.
2. To evaluate the effects of ground tire rubber (GTR) modifier on the Superpave™ volumetric design procedure.
3. To evaluate the effects of GTR modifier on the rutting and cracking resistance of Superpave™ mixtures.
4. To provide recommendations/guidelines for a modified Superpave™ mixture design procedure that points to the need for additives early in the mixture design process.

1.3 Scope of Study

A wide variety of modifiers currently exist in the asphalt industry today. As a result the study focused on GTR modifier (GTR, sieve # 80), which is referred to as a thermoset in the literature. The GTR was combined with PG-graded asphalt PG 64-22 to produce the modified binder (rubber-asphalt). Aggregates used in the study include white rock, Cabbage Grove, and Calera, which are all currently being used by the FDOT.

Superpave™ mixes were produced for design traffic levels 3, 4, and 5. Both modified and unmodified samples were produced to serve as controls. The physical properties of the samples were then analyzed by comparing their resistance to rutting and cracking as measured by the Servopac gyratory compactor and the indirect testing machine, respectively (MTS). Aged samples were also produced to determine the effect of the selected modifiers with age hardening.

CHAPTER 2 LITERATURE REVIEW

2.1 Introduction

The purpose of this chapter is to review the various distresses in hot mix asphalt (HMA) pavements, and the use and effect of modifiers on such distresses in HMA pavements. Also some of the major concerns of the SuperpaveTM mix design method, which have given rise to further research, will be discussed.

2.2 Distresses in Hot Mix Asphalt Pavements

Asphalt pavement distresses or defects can be broadly categorized into three main types namely cracking, permanent deformation (which includes rutting), and surface disintegration (Paterson 1987). These may develop individually or interactively at the different stages of the life of an asphalt pavement. Another type of pavement distress that does not fall directly within these broad categories is stripping or moisture damage. The primary causes of these distresses are traffic loading, environment, age hardening (gradual oxidation of pavement), or a combination of two or more of these factors. Other possible causes include poor construction, poor mix design, and unsuitable climate during construction, or even opening the road facility to traffic too early (Paterson 1987).

2.2.1 Cracking

Cracking can be further classified as load associated or non-load associated. Load-associated cracking, primarily fatigue cracking (alligator cracking), is the phenomenon whereby pavement fracture occurs under repeated or fluctuating stress with a maximum value less than the tensile strength of the HMA surface. Major factors that influence the initiation and development of load associated fatigue cracking include structural section of the pavement,

traffic, environment, consistency of the asphalt cement, asphalt content, and properties of road sub grade (Finn et al. 1978).

One type of non-load associated cracking which is of considerable interest to engineers in the paving industry is thermal cracking or low-temperature cracking, which normally manifests itself in the form of transverse cracks (Khandal 1978). Other types of non-load associated cracking are longitudinal, reflective, and block cracking, which is a combination of longitudinal and transverse cracks.

Low-temperature cracking occurs when the pavement experiences a temperature drop, which causes tensile stresses to develop. If these stresses exceed the strength of the HMA pavement layer, then transverse cracks develop. The cracks propagate further under the influence of the adverse cyclic environmental conditions, which eventually leads to the deterioration of the pavement.

2.2.2 Permanent Deformation

Unrecoverable deformation or permanent deformation is the broad term used to describe pavement distresses such as rutting, shoving, and depressions (Paterson 1987), of which rutting is of prime importance due to its predominance in HMA pavements. Rutting has been defined as the progressive movement of materials under static or cyclic loads either in the asphalt layer or the underlying road structure. This progressive movement is caused by either the consolidation (i.e., further compaction) of the pavement due to traffic, or lateral plastic flow under vehicular wheel paths (Roberts et al. 1991). Factors that affect rutting include, among others, the consistency or viscosity of the asphalt cement, and the percentage asphalt content of the mix. Research and experience indicate that excessive asphalt cement, which causes loss of internal friction between the aggregate particles within the mix, is a common cause of rutting. Also

asphalt cements with low viscosity could lead to rutting at high temperatures. However, the consistency of the asphalt cement plays a minor role in the resistance of a pavement structure with a well-graded, angular and rough-textured aggregates (Roberts et al. 1991). Generally, poor mix design in relation to the environment in which the pavement is to be located, and a weakened base structure would most probably lead to rutting of the pavement.

2.2.3 Moisture Damage

Moisture-induced damage, or stripping, is the weakening or eventual disruption of the adhesive bond, usually in the presence of moisture, between the aggregate surface and the asphalt cement in an HMA pavement (Roberts et al. 1991, Crossley and Hesp 2000). In 1999, Maupin reported that in a survey conducted in Virginia on 74 pavement sites, 30 to 50% of the roads inspected showed a significant degree of stripping indicating the significant occurrence of stripping as a form of distress in HMA pavements. The presence of moisture eventually leads to the loss of cohesion of binder and aggregates, thereby resulting in a reduction of the strength of the asphalt matrix (Roberts et al. 1991, Crossley and Hesp 2000).

Six types of stripping have been identified in the literature (Abed El Halim et al. 1993), namely, emulsification, detachment, displacement, film rupture, pore pressure, and hydraulic scouring. Contributing factors to the phenomenon of stripping include the chemical and physical properties of the aggregates and asphalt cement, traffic, environment and construction practice. Even though the occurrence of the different types of stripping is initiated by different mechanisms, the underlying factor is the presence of moisture at the asphalt/aggregate interface of the pavement (Roberts et al. 1991, Crossley and Hesp 2000). A summary of the pavement distresses and their most probable causes is presented in Table 2.1 (Roberts et al. 1991).

Table 2.1 Summary of Distresses and Possible Causes in HMA Pavements

Type of Distress	Most Probable Cause
1. Cracking: a. Load associated - Fatigue (alligator) cracking - Premature cracking b. Non-load associated - Thermal/low temperature cracking - Longitudinal cracking - Reflective cracking - Block cracking	- Traffic load and environment - Early opening of road to traffic - Environment (temperature) - Poor construction - Combined effects
2. Permanent Deformation a. Rutting b. Shoving/heaving c. Depressions d. Roughness e. Wear	- Poor mix design/poor construction, weakened base structure - Poor mix design/poor construction, weakened base, early traffic - Weakened support, environment - Combined distresses - Traffic load/ aggregate type
3. Disintegration a. Raveling b. Potholes c. Edge breaks	- Age hardening - Cracking + raveling - Traffic + moisture
4. Stripping a. Emulsification b. Detachment c. Displacement d. Film rupture e. Pore pressure f. Hydraulic scouring	- Combined effects of chemical, physical properties of aggregates, asphalt cement, and environment

2.3 The Use of Modifiers in Asphalt Pavements

2.3.1 The Need For Modified Asphalt Binders in HMA Pavements

Asphalt modifiers (or additives) have been used in the road construction industry as far back as the early 1950's (Roberts et al. 1991). 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 1950's, i.e., to improve the performance of asphalt pavements in terms of increased resistance to pavement distresses (notably cracking, rutting, and stripping) thereby prolonging its service life. Modifiers have been used more extensively in Europe as compared to the United

States partly due to the contractor guarantees that some European countries demand on asphalt pavements (Roberts et al. 1991). However, the resurgence in the use of modifiers in the United States within the past decade could be attributed to the following factors:

- a) Increasing demand on HMA pavements due to higher traffic volume and traffic loads.
- b) The recently developed Superpave™ binder Performance Grade (PG) specification based on the SHRP program, recommended in March 1993. This specification requires asphalt binders to meet stiffness requirements at low as well as high pavement service temperatures. However, the peculiar properties of most conventional asphalts inhibit their ability to meet these requirements in varying conditions and therefore may need modification.
- c) Recently, public agencies are willing to pay a higher cost for pavements with a prolonged service life or a reduced risk of premature failure, and
- d) Environmental and economic pressure to dispose of some waste materials and industrial byproducts by using pavement additives, notably vehicular rubber tires.

From a more technical perspective, modifiers are added to asphalt pavements to improve its resistance to the major distresses a pavement may experience during its service life, namely fatigue, rutting, and moisture damage. Haas et al. (1982) gave a comprehensive or during mix production, to improve the properties and/or performance of definition of a modifier as follows:

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.

It follows from the above definition that specific technical reasons for using modifiers in HMA mixes include, among others,

- To obtain stiffer mixes at high service temperatures to enhance its resistance, as much as possible, to rutting (Roberts et al. 1991);
- 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 bonding to reduce the incidence of stripping or moisture damage;
- To improve the resistance to abrasion which also reduces other forms of surface disintegration;
- To rejuvenate aged asphalt binders; and
- To improve the overall performance of HMA pavements.

An ‘ideal’ HMA pavement binder would be said to have been obtained if it exhibited the above-mentioned properties after being modified with an additive (Lukanen 1987). Even though significant successes have been reported in the literature in the use of (Crossley and Hesp 2000, Shuler et al. 1987, Terrel and Walter 1986) a few specific issues still need to be addressed with regards to the use of modifiers such as the procedures or method of incorporation of the modifier into the binder and/or mixture, the recycleability of the modified mixture (such as in the case of ground tire rubber, GTR), health and safety issues, specifications for modified binders, and finally initial and life cycle cost considerations (Roberts et al. 1991).

2.3.2 Classification and Effects of Modifiers on HMA Pavements

In the asphalt industry today, one can find a variety of substances referred to as modifiers, which in one way or another attempts to provide remedial measures for one (or more) of the pavement distresses previously discussed. These products range from naturally occurring substances such as rubber, Gilsonite, sulphur, and lime, to complex engineered substances such as styrene-butadiene-styrene (SBS), styrene-ethylene-butylene-styrene (SEBS), and ethyl-vinyl-acetate (EVA) (Roberts et al. 1991). More recently Silane-functionalized polydiamines have been investigated by Crossley and Hesp (2000) as potentially effective against moisture damage

and low temperature failure. A generic classification of modifiers has however been presented by Terrel and Walter and a few other researchers (Roberts et al. 1991, Haas et al. 1982, Button 1992). A modified version of Terrel and Walter's classification is presented in Table 2.2.

Table 2.2 Generic Classification of Asphalt Modifiers Currently Being Used or Tested In Hot Mix Asphalt Pavements

Type	Generic Example
1. Filler	Carbon Black Mineral Fillers: - Fly ash - Crusher fines - Lime - Portland cement
2. Extenders	Sulphur Lignin
<i>Polymers</i>	
3. Elastomers a. Natural latex b. Synthetic latex c. Block, diblock copolymers (reactive polymers) d. Reclaimed tire rubber	Natural rubber Styrene-butadiene rubber (SBR) Styrene-butadiene-styrene (SBS), styrene-isoprene-styrene (SIS), styrene-butadiene di-block copolymers Crumb rubber
4. Plastomers (thermoplastics)	Ethyl-vinyl-acetate (EVA) Polyvinyl chloride (PVC) Ethylene propylene (EPDM) Polyethylene / Polypropylene Ethylene Acrylate Copolymer
<i>Others</i>	
5. Antistripping agents	Amines Lime
6. Hydrocarbons (natural asphalts)	Gilsonite Trinidad Lake Asphalt Recycling and rejuvenating oils
7. Antioxidants	Lead compounds Carbon Calcium salt
8. Oxidants	Manganese salts
9. Miscellaneous	Deicing calcium chloride Silicones

2.3.2.1 Natural asphalts

Among the naturally occurring asphalts, Gilsonite and Trinidad Lake Asphalt have been used as additives in HMA pavements. Sam Gilson first discovered Gilsonite in the Utah Basin

of Eastern Utah in 1885. It is a naturally occurring asphalt, with a penetration of 0 to 3 at 77° F, and ring and ball softening point between 250 and 350° F (Roberts et al. 1991). Gilsonite is known to increase the viscosity of HMA mixes, the rutting resistance at high service temperatures, and also substantially increases the Marshall stability of mixes. However, it also results in mixes with high viscosities at low service temperatures (Button 1992; Tia 1999). Trinidad Lake asphalt has a penetration range of 3-10 at 77° F and a softening point in the range of 200-207° F. It has been used in construction for high stress areas such as intersections and access to toll booths (Roberts et al. 1991).

2.3.2.2 Mineral fillers

Numerous researchers have investigated the use of mineral filler as an additive in the past (Button 1992; Tia 1999; Vallerga and Gridley 1980; Yao and Monismith 1987; Khosla and Zahran 1987; Tayebali et al. 1991). 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. It is reported that the use of these substances, especially carbon black, is beneficial to the durability, the wear resistance, and the temperature susceptibility of the mix.

They 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 (Roberts et al. 1991, Crossley and Hesp 2000).

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 in the pavement industry as Microfil 8 (Roberts et al. 1991; Huang 1994), Yao and Monismith (1987) 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. (1987) 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.

2.3.2.3 Polymers

A polymer is a large molecule that is formed by chemically reacting many (poly) smaller molecules (monomers) to one another in long linear or branched chains. The process of forming these polymeric compounds is known as Polymerization. The properties of the resultant polymer depend on the sequence and chemical structure of the constituent monomers (Roberts et al. 1991; Remp and Merrill 1986). Polymeric materials can be engineered to have peculiar physical and chemical properties depending on the initial properties of the constituent monomers. Researchers have generally categorized polymers into two major groups, elastomers and plastomers (thermoplastics). However a third category of polymers known as thermosets are mentioned in the literature.

Elastomers

Elastomers (or rubbers) are lightly cross-linked polymers that can be defined as materials capable of fairly large elastic deformation (Kinloch and Young 1983). They resist deformation from applied loading or stress by stretching and recovering their shape quickly when the applied load or stress is removed. From the standpoint of modulus, elastomers are also referred to as substances with limited extensibility and incomplete retraction (Huang 1994). Even though they are known to add very little strength to asphalt cement until they are stretched, their strength increases with elongation. Some examples of elastomers are styrene-butadiene-rubber (SBR),

styrene-butadiene-styrene (SBS), styrene-isoprene-styrene (SIS), natural rubber and crumb rubber modifier that is produced from ground tire rubber (GTR)¹ (Roberts et al. 1991).

SBR was developed under the name “BUNA-S” in Germany, and in North America as “GR-S” (government rubber-styrene) during the Second World War (Charrier 1991). It is produced by the free radical copolymerisation of styrene and butadiene in emulsion. Studies have shown that SBR improves the elastic characteristics of the binder without increasing the stiffness or rigidity of binders at low service temperatures. However, a marked increase is noted in the rigidity of the modified binder as compared to the straight asphalt cement. The modification of the binder with SBR may thus result in the increased resistance to rutting and also improve the cracking resistance of the mix at low service temperatures (Verhaeghe et al. 1994; Collins et al. 1991; Armijo 1993; Kraier et al. 1988.).

Ground Tire Rubber

Ground tire rubber is a polymer that is often referred to as a thermoset. In the literature thermosets are differentiated from elastomers in that they are more rigid, tightly cross-linked polymers that degrade rather than melt upon the application of heat (Kraier et al. 1988). Crumb rubber produced from ground tire rubber (GTR) is a modifier to which some attention has been given in recent years in the United States, partly due to the need for a solution to the increasing number of huge discarded tire piles in the country. It is reported that 285 million tires are discarded every year in the United States (Roberts et al. 1991). Out of these, 97 million are retreated, or used for combustion or some other applicable purpose, while the remaining 188 million are added to stockpile, landfills, etc. Several states, including Florida, have enacted legislation to address the issue. In 1988, the Florida Senate Bill 1192, directed the Florida

¹GTR is sometimes classified as a *Thermoset*

Department of Transportation (FDOT) to evaluate the potential use of reclaimed tire rubber in the construction of asphalt pavements.

GTR has been blended with asphalt in various types of pavement construction such as seal coats, inter-layer, and open-graded friction courses (Collins et al. 1991; Piggott et al. 1977; Huffman 1980; Schnormeier 1980; Vallerga and Bagley 1980; Ford 1982). It is reported that when GTR is mixed with asphalt (usually at temperatures between 135 and 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. This leads to a significant increase in the viscosity of the asphalt-rubber mixture. Researchers have reported that the resulting modified binder has a lower temperature susceptibility, increased resistance to plastic deformation at high service temperatures, an improved resistance to age hardening (Kortschot and Woodhams 1984; Scoffield 1986; Lalwani et al. 1982) and an increased resilient modulus which increases the ability of the pavement to resist cracking at low service temperatures. Notwithstanding potential benefits in the use of crumb rubber, one major issue of rubber-modified binders is its questionable suitability as RAP (Roberts et al. 1991) and relatively higher initial cost (Khosla and Zahran 1987; Estakhri et al. 1992; Takallou and Sainton 1992).

Plastomers

A plastomer (or thermoplastic) is a material that is solid and possesses significant elasticity at room temperature and turns into a viscous fluid material at higher temperatures. When cooled these substances would regain their original or rubber-like nature (Charrier 1991). Typical examples of plastomers include ethyl vinyl acetate (EVA), polyvinyl chloride (PVC), ethylene propylene (EPDM), and styrene-ethylene-butylene-styrene (SEBS). Plastomers have a tough, rigid, three-dimensional network, which is resistant to deformation. As such, they are

noted for imparting early strength to the pavement structure on loading but may fracture under strain. Therefore such polymers provide a pavement with a high stiffness modulus while elastomers give a more resilient, flexible pavement (Roberts et al. 1991). A thermoplastic would generally be appropriate as a modifier when small strains and maximum stability are required for the asphalt mixture (Huang 1994).

Styrene-ethylene-butadiene-styrene is formed by the polymerization of ethylene-butadiene rubber to block copolymer styrene (Charrier 1991). This is different from the randomly co-polymerized system like styrene-butadiene rubber (SBR) which gives a response intermediate between the polymers polystyrene and polybutadiene. SBS is reported to substantially increase the strength of the mix at high service temperatures (Huang 1994). 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 (1985) used SBS and SEBS as modifiers to upgrade pavements in surface dressing applications and found that the addition of SEBS or a combination of these polymers to asphalt binders can provide reduced penetration, increased ring and ball softening points, improved low temperature ductility, increased toughness and tenacity, and increase the viscosity at service temperatures.

2.4 Superpave Mix Design

2.4.1 Background

Since the early 1940s and 1950s asphalt pavements have been designed using the Hveem and Marshall mixture design methods. These design methods are still practiced in some parts of the world today. The Hveem and Marshall mixture design methods being essentially empirical

in nature do not provide the necessary assurance of field performance needed for asphalt pavements today. With the increase in traffic and heavier loads over the years, an improved method of design, which could be related directly to field performance, was needed. This design method should allow for the rational design of asphalt mixtures for various traffic loads, environments, and also provide tests with models to predict rutting, thermal cracking, and fatigue cracking (Roberts et al. 1991).

In 1987, the US government committed \$150 million to the five year SHRP road research program, out of which \$50 million was channeled towards the development of asphalt pavement specifications, tests and models that directly relate to field performance and prediction (Asphalt Institute). At the end of the research a new system of asphalt mixture design referred to as SuperpaveTM (for *superior performing pavements*) was developed. The new system has the following key elements,

- A new grading system for asphalt binders, called Performance Graded (PG) grading system
- Performance related aggregate specifications,
- A new mixture design process (which involves a gyratory method of compaction) and a new analysis procedure (Roberts et al. 1991).

Superpave consists of three levels of design, Levels 1, 2, and 3. Level 1 is an improved material selection and volumetric mix design procedure. Level 2 picks up from the volumetric mix design and conducts other tests for the prediction of field performance. Level 3 is a more comprehensive array of tests to achieve a more reliable level of performance prediction. Since Levels 2 and 3 have not been completely developed yet, the design procedure as being implemented today is purely Level 1 mix design, i.e., the rigorous selection of materials and volumetric mix design (Asphalt Institute).

2.4.2 Major Concerns of Superpave Mix Design

Since 1996 quite a number of US states have adopted the Superpave mix design system and have implemented it in one form or the other. It was estimated that by 1998 thirty-seven states (including Florida) had implemented the new binder specification. By the end of the year 2000, it is estimated that 46 states would have implemented the binder specification and 39 states the Level 1 mix design method (Tia 1999) Notwithstanding the above figures, a few pertinent concerns have been raised with regards to the validity of some of the binder specifications and the mix design process. Some of these are briefly discussed below.

2.4.2.1 Fine aggregate angularity (FAA)

The validity of the uncompacted void content, or fine aggregate angularity (FAA) criteria has been questioned since the implementation of the Superpave design system. The criteria was introduced in the design system to screen out smooth and/or rounded fine aggregates that may supposedly result in mixtures with low rutting resistance. The presumption is that fine aggregates with lower FAA values, according to ASTM C1252, will have lower shear strength (internal friction) and subsequent low resistance to permanent deformation. However, recent research at the University of Florida (Fernandes et al. 1999) shows that there is not enough correlation between the shear resistance of an asphalt mix and the uncompacted void ratio of the fine portion of the aggregates to validate this specification. The study reported that even though the FAA did contribute to the shear strength of a mix to some extent, its importance is overridden by other factors such as the toughness, the overall gradation, and the packing characteristics of the aggregates. Apart from such studies, it has also been the concern of some DOT's such as the FDOT that local aggregates do not meet the FAA specification.

2.4.2.2 Aggregate gradation – “restricted zone”

One of the aggregate selection criteria is based on the gradation of the aggregates.

Plotted on a 0.45 power gradation chart the gradation of the aggregates is controlled by a set of control points and a “restricted zone” which is a no-go area for the aggregate gradation curve (Asphalt Institute; Roberts et al. 1991). Mixtures that pass through the restricted zone are said to be over-sanded. Such tend to possess compaction problems during construction and more importantly offer little resistance to rutting. However, there are some aggregates that would pass through the restricted zone that are well crushed and meet the minimum VMA requirements. Also results of some of studies have shown that such mixtures perform equally well, or even better, than those that do not (Tia 1999).

2.4.2.3 Voids in mineral aggregates (VMA)

Voids in the mineral aggregates (VMA) of a mix is defined as the volume of intergranular void space between the aggregate particles of a compacted paving mixture that includes the air voids and volume of asphalt not absorbed by the aggregates. Quantitatively, it is expressed as the ratio of the volume of air plus asphalt and the bulk volume of the compacted mix expressed as a percentage (Roberts et al. 1991). The VMA parameter is very important in the volumetric analysis of asphalt mixtures due to the fact that it controls the amount of binder and/or air voids within an asphalt mixture. The amount of binder and/or air voids in an asphalt mixture correlates directly to its overall durability and its potential to bleed (Tia 1999).

The continued implementation and evaluation of the Superpave system in Florida and other states have raised questions regarding the validity of the VMA criteria. There are reports of increased difficulty in meeting the minimum VMA requirement. Also some producers may have to gap-grade the aggregates in the coarser aggregate range to meet the specification, which

may subsequently have a detrimental effect on the permeability and shear strength of the mix. In order to have adequate durability aggregates for HMA must have a minimum film thickness for adequate cohesion within the matrix of the HMA. The minimum VMA requirement specified in the Superpave design procedure is based on the rationale of incorporating at least a minimum permissible asphalt content into the mix to ensure its durability. However, the criterion is only dependent on the nominal maximum aggregate size of the mixture and not the overall gradation of the aggregate (Asphalt Institute; Tia 1999). On-going studies at the University of Florida tend to suggest that the VMA criteria need to be modified to incorporate the effect of the overall gradation of the aggregate of a mix and not only the nominal maximum aggregate size. Also other researchers have shown that based on the requirement of a minimum film thickness, mixtures with a coarser aggregate gradation should have a lower VMA requirement because of its lower surface area (Bensa et al. 2001).

2.4.2.4 Protocols for modified asphalt binders

The concept of modifying asphalt binders and mixtures, which is gradually taking prominence in the construction industry today, dates as far back as the 1950's. Asphalt binders are modified with the sole aim of improving some particular physical characteristic of the HMA mix thereby increasing its resistance to one (or more) of the major pavement distresses, namely rutting, cracking and moisture damage.

With the advent of Superpave, characterization of asphalt binders has shifted from the Pen-Viscosity System to the Performance Graded (PG) System, by which asphalt binders are graded (or classified) in relation to their rheological properties at high, intermediate and low temperatures. Important qualitative parameters used in this classification are the parameters G^*

(complex modulus) and δ (phase angle). The use of these parameters in characterizing asphalt binders hinges on the two basic assumptions (Bahia) that:

1. the evaluation of the binder is based on properties within the linear visco-elastic range, during which the behavior is independent of the strain or stress level; and
2. the binder behavior is independent of the film thickness and sample geometry.

These assumptions permits the evaluation of asphalt binders as simple systems that can be analyzed using linear visco-elasticity and simple geometries within which the stress and strain curves are simple to evaluate.

Modifiers are substances, which, by virtue of their physical and chemical properties, change the rheological properties of asphalt binders by varying degrees when utilized in the production of HMA. Some of these changes include increase in viscosity and the particulate characteristics of the binder. As a result of these changes the rheological properties of modified binders violate the assumptions on which the PG-grade is based on. Therefore, the classification of modified asphalts with the PG-grade system may result in modified binders being graded incorrectly, particularly so for the high-temperature PG-grade. Studies by Bahia reveal that some modified binders (termed “Complex Binders”) violate the assumptions of the Superpave protocol as stated above. Other independent studies by FHWA also showed that a modified binder having the same $G^*/\sin \delta$ value as the unmodified binder had as much as 10 times more rutting resistance than the unmodified binder. This demonstrates that the current Superpave binder protocols and PG-grading system should be modified to capture the effect of modifiers on asphalt binders, and the differences between the rheological and performance related properties of straight and modified asphalts, respectively.

CHAPTER 3 RESEARCH PROGRAM AND INSTRUMENTATION

3.1 Introduction

This chapter provides information on the type of materials and procedures for the production of HMA samples in the laboratory. Also presented is a summary of the testing procedures and instrumentation.

3.2 Materials

This section provides information on the aggregates and the asphalt binder used in the production of the HMA samples. Properties of the ground tire rubber (GTR), the modified binder and unmodified (control) asphalt binder are also provided.

3.2.1 Aggregates

Two main aggregates were used in this study, namely, white rock (crushed limestone) and Cabbage Grove. The limestone is mined from Miami, South Florida by White Rock Quarries, Inc. White rock is one of the major aggregates currently used in the state of Florida and approved by the FDOT for road construction and rehabilitation projects. It is also a standard aggregate that the FDOT uses for its FC-2 friction course mixture. Coarse and fine gradations of the limestone aggregate were developed for the production of the HMA laboratory mixes. Also Cabbage Grove fines (i.e., fine aggregate below sieve #8) were used in this study. According to Kestory, 2000, Cabbage Grove has been noted as high L.A. abrasion that breaks down during compaction of the mix. Therefore, Cabbage Grove was used as part of the study primarily to determine whether the rubber improves toughness or reduces the breakdown of the aggregate during compaction. Cabbage Grove was used by volumetrically replacing the fine part of white

rock (passing sieve #8 to #200) with the fines in Cabbage Grove. The resulting aggregate structure was then tested and evaluated.

3.2.2 Rubber Modified Binder

Ground tire rubber (GTR) or crumb rubber passing sieve #80 was blended with AC-30 to produce a rubber-modified binder for the production of the modified HMA mixes. Rouse Rubber Industries, Inc supplied the GTR modifier. Details of their chemical constituent as provided by the supplier are provided in Tables 3.1 and 3.2, respectively.

Table 3.1 Chemical Constituents of #80 Mesh Ground Tire Rubber (GTR)

Constituent	Percentage
Acetone Extract	16.3
Ash	4.8
Carbon Black	30.9
Rubber Hydrocarbon	48
Moisture	0.80
Specific Gravity	1.145

Table 3.2 Gradation of Ground Tire Rubber (GTR)

Sieve Size	Percent Passing
#40	100.0
#60	100.0
#80	98.0
#100	2.0

3.3 Preparation and Handling of Materials

3.3.1 Aggregates

The specific gravity of the aggregates was determined in accordance with ASTM C127 and ASTM C128. Also the gradations of the aggregate were determined according to ASTM C136. Before the aggregates were used in producing the HMA they were oven dried for 8 hours

at 225° F and sieved into their individual sieve sizes ready to be batched according to the job mix formula to be used.

3.3.2 Preparation of Rubber-modified Binder

The modified binder was produced by blending the GTR with AC-30 for approximately 20 minutes at 300° F with a Silverson L4R High Shear Mixer. The AC-30 was heated to the blending temperature for about 3 hours. During the blending process the temperature of the asphalt-rubber mix was maintained (for a consistent blend) by means of a hot plate heated prior to the process to an approximate temperature of 300° F. Previous studies at the University of Florida by Otoo (2000) 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 some of its field sections constructed with modified binders.

Hence the modified binders were produced at 12% by weight of the straight asphalt for this study. The rheological properties of the modified (and virgin) binder were also determined after the blending process.

3.4 Design and Production of Mixtures

A flowchart summarizing the major activities in the design, production, and testing of mixes is presented in Figure 3.1. The laboratory HMA samples produced for testing and evaluation were designed with the Superpave™ volumetric mix design procedure, which bases its selection for design asphalt content (AC% content) on a set of criteria on the volumetric properties of the mix (VMA, VFA, density) at 4% air voids. Apart from the above procedure that determines the AC% content, the aggregates of the HMA mix need to fulfill a set criteria for

the consensus and source properties that aim to prevent the use of substandard aggregates in HMA mix design.

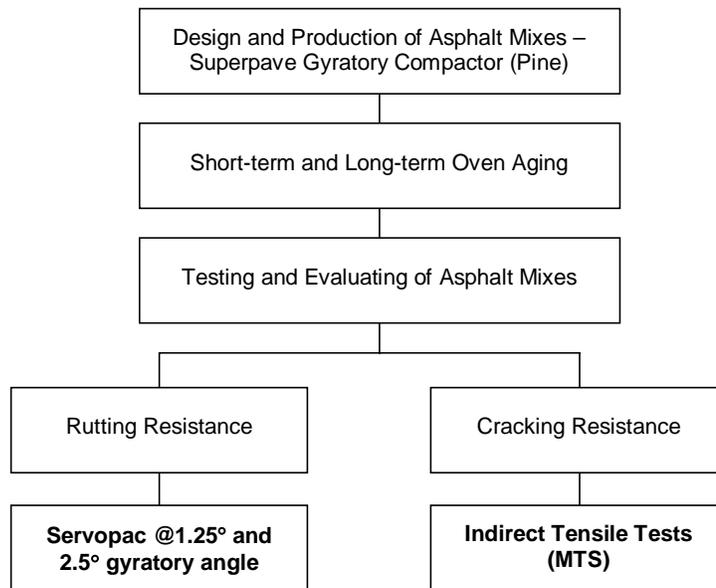


Figure 3.1 Flowchart of major activities.

3.4.1 Types of HMA Mixtures

Nine types of HMA mixtures were produced. These were produced from the coarse and fine blend of the limestone aggregate, and a combination of limestone and Cabbage Grove. The combined gradation was formed by replacing the fine portion of the limestone with Cabbage Grove. For each of these two blends HMA mixes were produced for three different Superpave™ traffic levels, namely Levels 3, 4, and 5, respectively. Requirements for these traffic levels as specified in the Superpave™ mix design manual are presented in Table 3.3 below. The mixtures produced at the various traffic levels and their design asphalt contents are also presented in Table 3.4 below.

Table 3.3 Superpave Traffic Levels and Gyrotory Effort

Traffic level/Gyrotory effort (Millions of EASL's)	N_i	N_d	N_{max}
3(1-3)	7	75	115
4(3-10)	8	100	160
5(10-30)	8	100	160
6(30-100)	9	125	205

Table 3.4 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	7.1	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 Preparation of Mixtures

The Superpave mix design procedure consisted of determining the design (optimum) asphalt content for the various JMF's. An estimate of the design asphalt content, $P_b\%$ was determined using knowledge of similar mixes produced with the same aggregates. Four

aggregate samples, 4500 g each, were batched out for asphalt mixes at $P_b\%$, $P_b \pm 0.5\%$, and $P_b + 1\%$. Three other aggregate samples of approximately 1500 g were also batched out for the determination of the theoretical maximum density (TMD) of the resulting asphalt mixes. The bulk density of the compacted 4500 g samples was also determined. With these results, volumetric analysis was used to determine the actual design content for the mixtures.

3.4.3 Batching and Mixing of Aggregates

Aggregate batching sheets were prepared for 4500 g samples and 1500 g samples based on the JMF's for the aggregates. The 4500 g samples batched out were heated in an oven at a temperature of 300° F (mixing temperature) for approximately 2½ hours. The mixing bucket and asphalt to be used were also heated to the mixing temperature. The aggregates were then removed from the oven and mixed for approximately 5 minutes (or until the aggregates were well coated) with asphalt corresponding to the estimated percentages (by weight of total mix) of P_b , and $P_b \pm 0.5$. The mixed samples were then spread out in pans and heated in an oven for 2 hours at a temperature of 275° F to simulate the short-term aging (STOA) of HMA. Each of the mixes was stirred after 1 hour to obtain a uniformly aged sample. The 950 g samples were used to determine the theoretical maximum density (TMD) of the various mixes according to AASHTO T 209-94.

3.4.4 Compaction of Samples

After STOA the 4500 g samples were then removed and quickly compacted using the Superpave gyratory compactor (pine). The samples were compacted with a ram pressure of 600 kPa at a gyratory angle of 1.25° to a maximum number of gyrations of 205, corresponding to N_{max} for Superpave Traffic Level 6 (TL-6). The compaction data of the samples were used in determining the design asphalt content for the various traffic levels. After the determination of

the design asphalt contents described above, straight (unmodified) and modified mixture samples were produced for testing on the Servopac and on the Superpave Indirect Tensile System (IDT). Samples tested on the Superpave IDT were compacted to 7% air voids at the design asphalt content for the relevant traffic level.

3.5 Testing of HMA Mixes

To evaluate the rutting and cracking resistance of the mixtures the samples were tested by compacting on the Servopac gyratory compactor (at gyratory angles of 1.25° and 2.5°, respectively), and also by performing resilient modulus, creep compliance and indirect tensile strength using the system developed by Roque et al. (1997).

3.5.1 Servopac Gyratory Compactor

The Servopac compactor is a HMA gyratory compactor developed in Australia. It uses a constant angle of gyration throughout the compaction process that is comparable to a strain-controlled loading system. Even though a gyratory angle of 1.5° is commonly used for compaction (e.g., SuperpaveTM mixes), it can be varied up to a maximum of 3°. The Servopac gyratory compactor provides a strength parameter (in the compaction data) termed the gyratory shear strength, G_s . The gyratory shear G_s in a specimen of Area A , and height h is given by Butcher 1998),

$$G_s = \frac{2PL}{Ah}$$

where P = average pressure measured in the gyratory actuators; and

L = the distance to the midpoint of the gyratory actuators.

Current research at the University of Florida tends to indicate that with a gyratory angle of 2.5° the Servopac can be used to evaluate the shear resistance and stability of HMA mixes

based on the mixture parameters derived from the compaction data. Mixture parameters referred to above include the rate of change of shear strength and the rate of change of air voids of the mix during the compaction process. Also, it was noted that compacting various mixes at a gyratory angle of 2.5° yielded compaction data sensitive enough that provides insight into the relative stability and shear resistance of the mixes.

For testing on the Servopac, 4500 g samples were compacted at gyratory angles of 1.25° and 2.5°, respectively. Samples were produced with the various aggregate types with the modified and unmodified binders. The mixes were subjected to short-term oven aging (STOA) for approximately two hours at 275° F (135° C) just as in the mix design. They were then compacted to the maximum number of gyrations of 205 gyrations at 275°F (135° C) for all the different design asphalt contents on the Servopac gyratory compactor.

3.5.2 Indirect Tensile Tests

To further evaluate the effects of GTR on the rutting and cracking resistance of the mixes, the resilient modulus, creep compliance and the indirect tensile strength tests were conducted on the different mixture types. The tests were conducted at 50° F (10° C) on two-inch thick specimens compacted to 7% air voids. The specimens were obtained by slicing off one inch from the ends of 4500 g-samples (compacted to 7% air voids) and then dividing the resulting sample into two equal parts. The testing procedures and data reduction was done according to the IDT System developed by Roque et al. (1997).

The two-inch thick specimens (subjected to STOA during mix design) were initially dehumidified for at least 48 hours prior to the placement of the gage points (for LVDT's) on the specimen. After the gage points were placed, the LVDT's were positioned on the gages and the specimen further conditioned at the test temperature of 50° F (10° C) for about 8 hours in an

environmental chamber. Care was taken to align the loading head of the system with the vertical axis marked on the specimen. To evaluate the effects of long-term aging on the specimens, separate mixtures were produced and subjected to long-term oven aging (LTOA) for 5 days at 185° F or 85° C (per AASHTO PP2-94). Two-inch samples were then produced from the aged gyratory-compacted samples and tested on the IDT System.

The resilient modulus tests were performed on the samples by applying a repeated peak-load of resulting in horizontal deformations within the range of 200-300 micro inches. Each load cycle consisted of 0.1-second load application followed by a 0.9-second rest period. After allowing the sample to re-stabilize (usually 5 to 10 minutes) the creep compliance test was performed. This consisted of a 1000-second test by the application of a constant load on the specimen. The load was chosen such that it produced a horizontal deformation within the range of 150-200 micro inches after 30 seconds of loading. The strength test was performed immediately after the creep test by applying a constant rate of displacement of 50 mm/min until the specimen failed.

CHAPTER 4 SUMMARY AND ANALYSIS OF LABORATORY TEST RESULTS

4.1 Introduction

This chapter provides a discussion of the binder and laboratory mixture testing results. A summary on the binder properties obtained, namely the viscosity, penetration, and the dynamic shear rheometer are also presented. Mixture properties obtained were the gyratory shear resistance, resilient modulus, creep compliance, tensile strength, failure strain and fracture energy density of the mixtures.

4.2 Summary of Binder Properties

4.2.1 Viscosity and Penetration

A summary of the binder properties is presented in Tables 4.1 and 4.2. The viscosity and penetration tests were performed at 64° C (147° F) and 25° C (77° F), respectively, for both the straight and rubber-modified binder. Comparisons of the results for the two binders indicate that the modified binder has a higher viscosity and a lower penetration. The addition of GTR to the straight AC-30 thus results in a binder with higher viscosity due to the presence of the rubber particles. This increase in viscosity could be beneficial to HMA pavements since it could lead to a decrease in permanent deformation at intermediate (or high) in-service temperatures. However, since the binder tests alone do not capture the actual interaction of GTR in HMA mixes, the full effects of the modifier would be evaluated from the mixture tests.

4.2.2 Dynamic Shear Rheometer

Results of the dynamic shear rheometer (presented in Table 4.2) indicate that the rubber-modified binder has a lower phase angle (δ) at 25° C and higher G^* values, and subsequently,

Table 4.1 Penetration and Viscosity Test Results

Penetration at 25° C (77° F)				
Binder Type	Replicate	Penetration	Average	Standard Deviation
Virgin AC-30	1	61	60	1
	2	60		
	3	60		
AC-30 + 12% Rubber	1	36	36	1
	2	36		
	3	35		
Viscosity at 60° C (140° F)				
Binder Type	Replicate	Viscosity (cp)	Average	Standard Deviation
Virgin AC-30	1	3.67E+05	3.78E+05	10173
	2	3.88E+05		
	3	3.80E+05		
AC-30 + 12% Rubber	1	6.83E+05	6.81E+05	3960
	2	6.85E+05		
	3	6.76E+05		

$G^*/\sin(\delta)$ 64° C. The G^* and $G^*\sin(\delta)$ falls within the limits of 1.0 kPa and 5000 kPa respectively set by the Superpave binder specifications. Even though these values seem to suggest a potential improvement in the rheological properties of the rubber-modified binder in relation to the crack and rutting resistance of HMA mixes, such a conclusion is only possible after the modified HMA mixes have been evaluated from the mixture tests in order to capture the complete interaction of the GTR modified binder with the aggregate structure of the mixes.

Table 4.2 Dynamic Shear Rheometer Results

Dynamic Shear Rheometer at 25° C (77° F)					
Binder Type	Replicate	G^* (kPa)	δ	$G^* \times \text{Sin}(\delta)$	Average
Virgin AC-30	1	1110	66.7	1020	946
	2	1070	67.4	985	
	3	902	67.4	833	
AC-30 + 12% Rubber	1	1140	58.1	968	933
	2	1100	57.8	930	
	3	1062	57.9	900	
Dynamic Shear Rheometer at 64°C (147° F)					
Binder Type	Replicate	G^* (kPa)	δ	$G^*/\text{Sin}(\delta)$	Average
Virgin AC-30	1	1.93	86.2	1.93	1.99
	2	2.01	86.1	2.02	
	3	2.01	86.2	2.02	
AC-30 + 12% Rubber	1	5.94	78.3	6.06	5.85
	2	5.79	78.5	5.91	
	3	5.46	78.3	5.57	

4.3 Mixture Testing

The rutting and cracking resistance of the mixtures were tested by compacting samples on the Servopac gyratory compactor (at gyratory angles of 1.25° and 2.5°, respectively), and also by performing resilient modulus, creep compliance and indirect tensile tests. Details and analysis of the results are presented below.

4.3.1 Servopac Gyrotory Compactor (SGC)

Results of the Servopac gyrotory compaction are presented in Figures 4.1 to 4.6. The samples were compacted at gyrotory angles of 1.25° and 2.5°, respectively. The Servopac samples were produced from two types of aggregates: a) one with a medium-high LA abrasion resistance and very high direct shear test (DST) values; and b) one with a low LA abrasion resistance and low DST values.

The SGC results were looked at and analyzed from three different perspectives to evaluate the measured gyrotory shear resistance of the mixes which could give an insight into their resistance to permanent plastic deformation. The three perspectives are,

1. a direct comparison of the gyrotory shear measurements;
2. evaluation of the areas under the gyrotory shear vs. volumetric strain curve; and
3. the rate of change of gyrotory shear resistance with respect to air voids, dG_s/dAV .

Compacting at an angle of 2.5° produced more sensitive results, which brought out the differences in the various mixes that gave better insight into the effects of rubber (or the absence of any) on the samples. As a result only details of the compaction at 2.5° is discussed.

4.3.1.1 Direct comparison of gyrotory shear resistance (G_s)

- a) Aggregate type: Medium LA abrasion and high DST

Coarse Aggregate

A summary of the SGC test is presented in Table 4.3. Also plots of the gyrotory shear results for the coarse aggregates are presented in Figures 4.1 - 4.3. It is noted from the plots that after the peak value, the gyrotory shear of the samples drop as the number of gyrations increase. This post-peak drop, represented by D_{p-f} (as illustrated in Table 4.3) is greater for the unmodified mixes than the modified ones. All the modified samples (with 6.1%, 6.6%, and

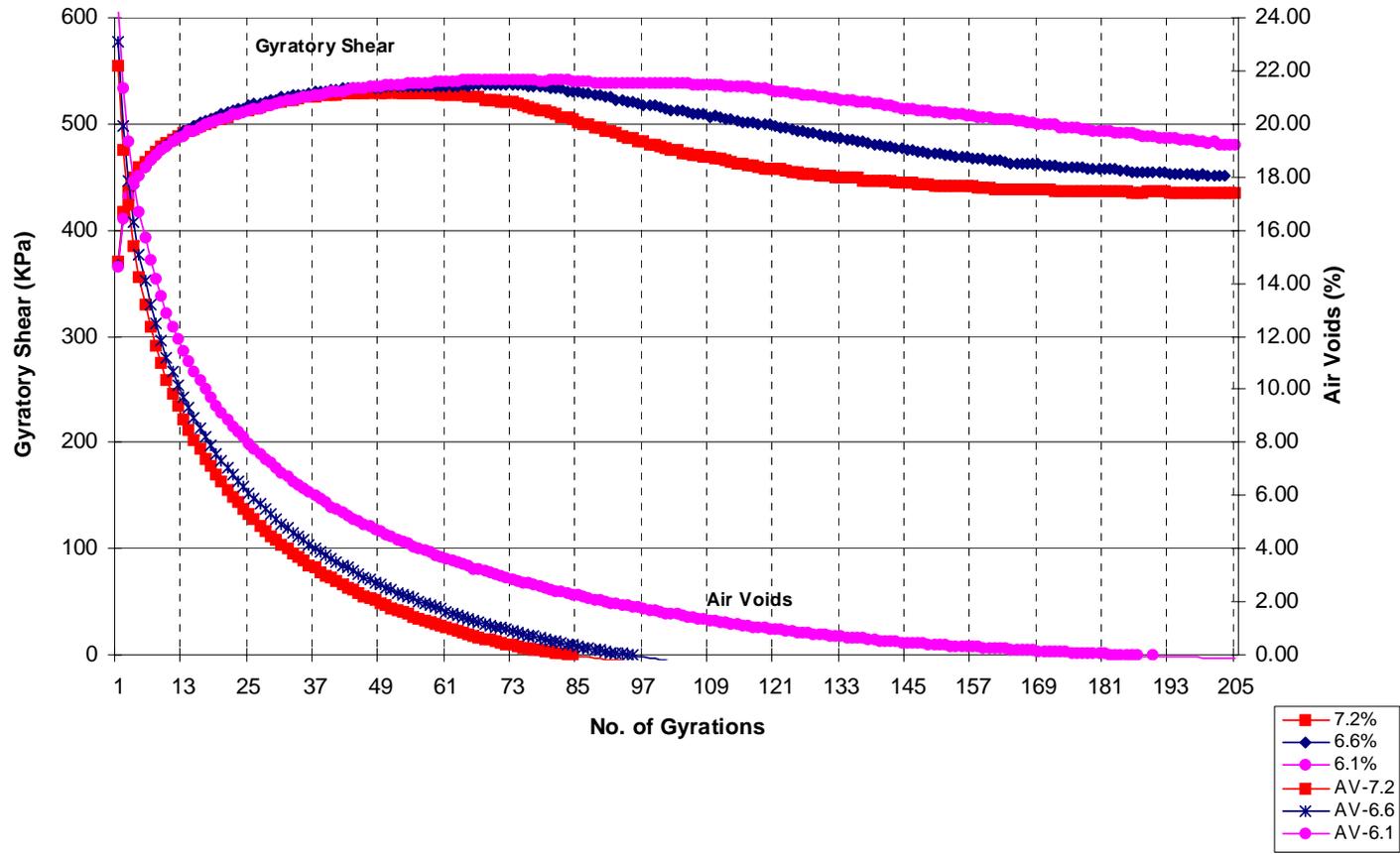


Figure 4.1 Gyratory shear vs. gyrations and air voids vs. gyrations (coarse mixes – straight).

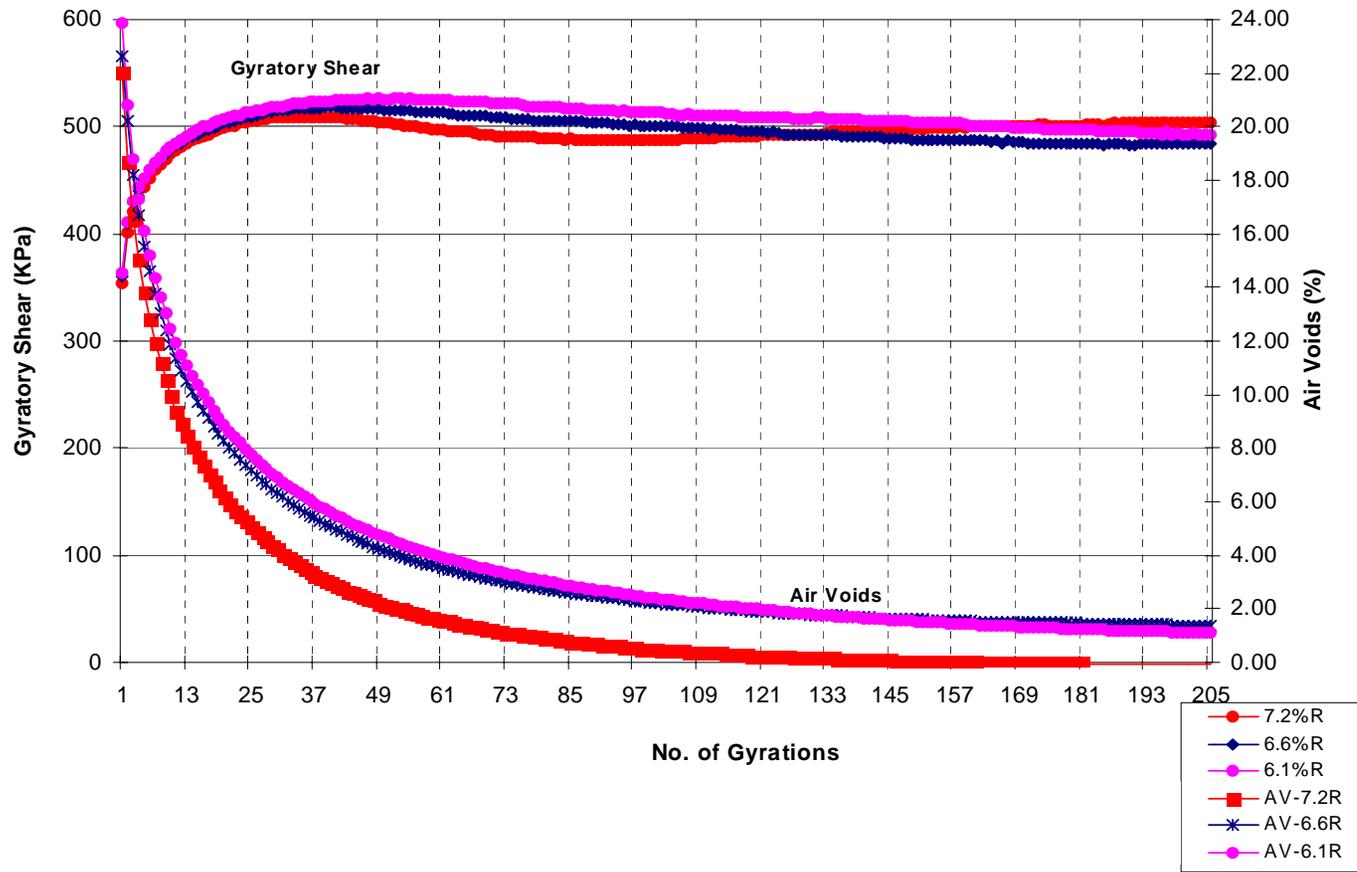


Figure 4.2 Gyrotory shear vs. gyrations and air voids vs. gyrations (coarse mixes – rubber).

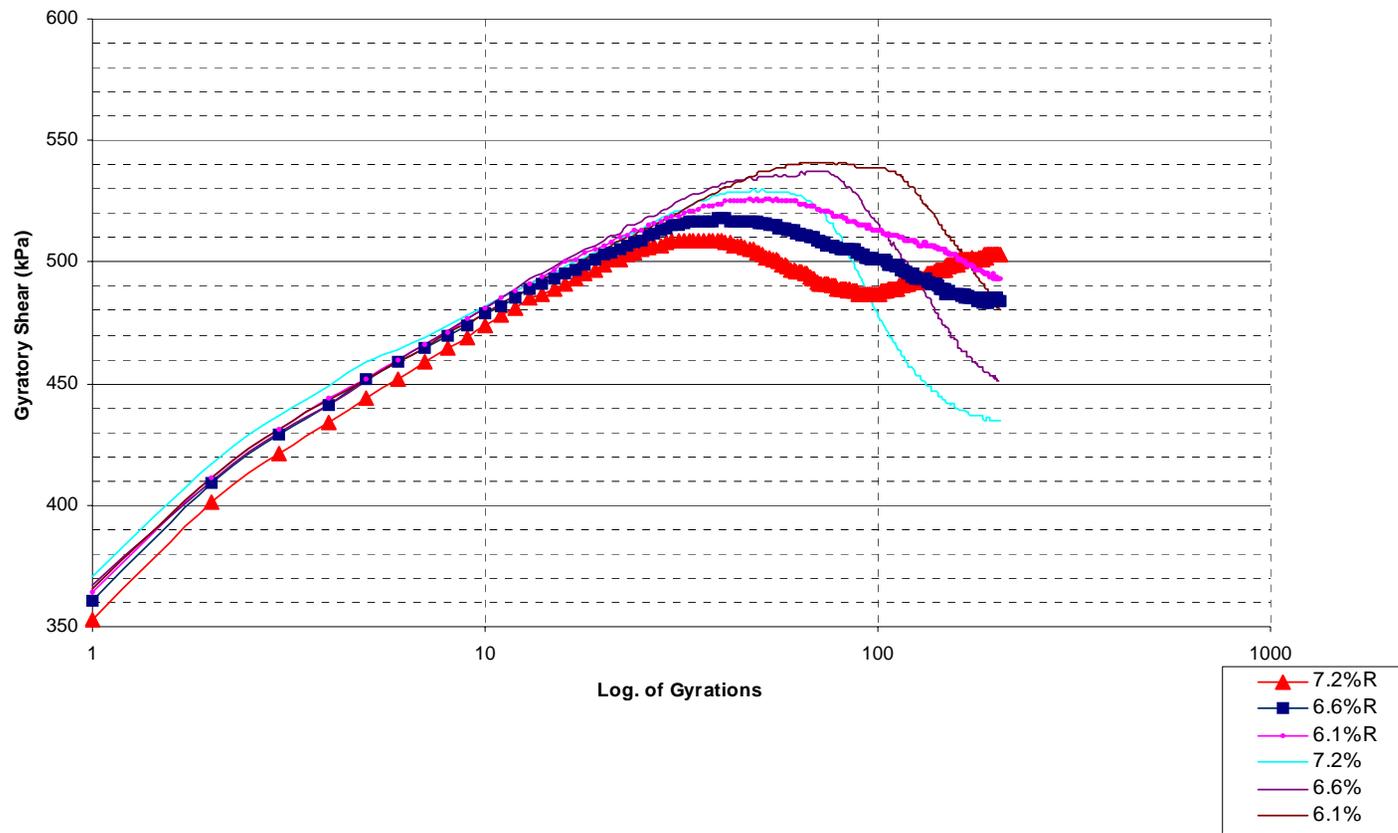


Figure 4.3 Gyrotory shear vs. log of gyrations (coarse mixes).

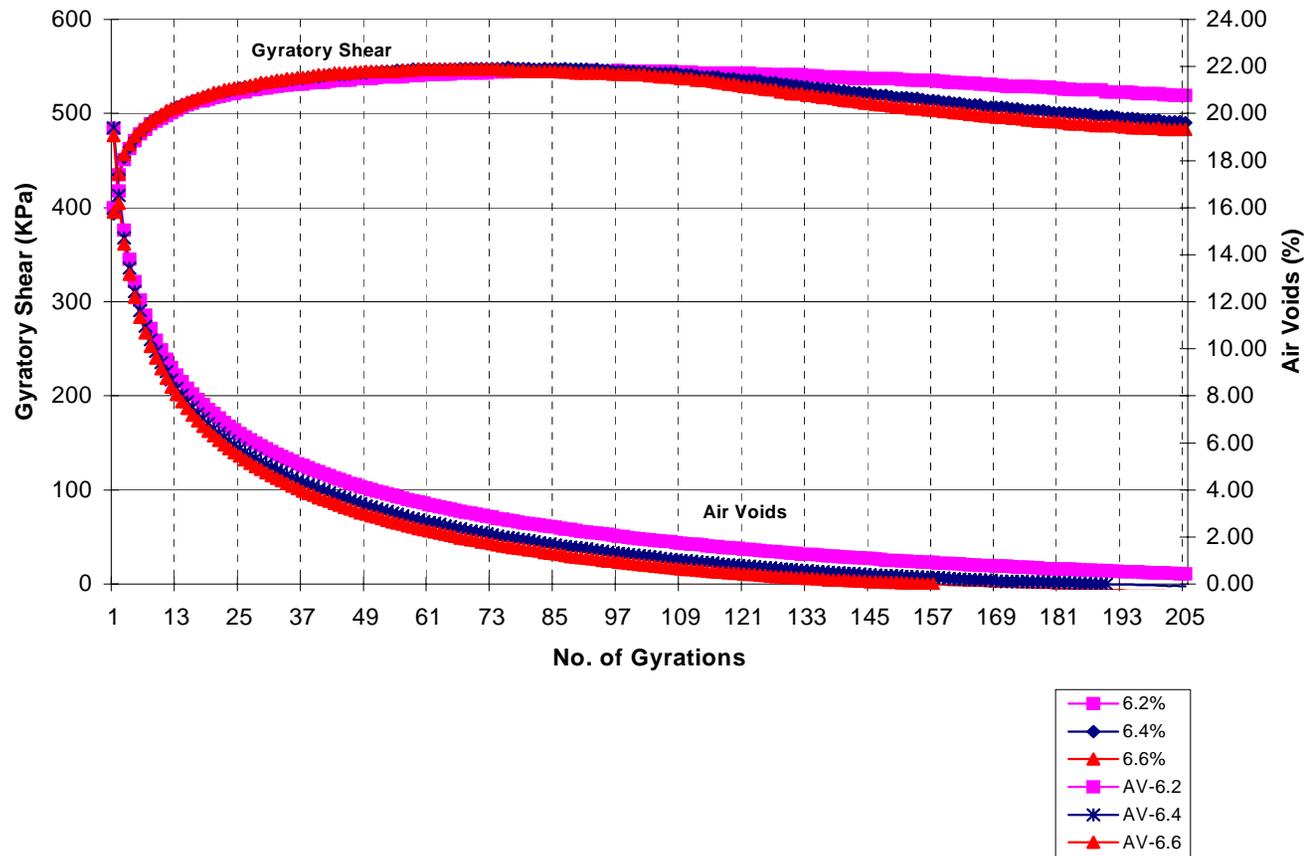


Figure 4.4 Gyrotory shear and air voids vs. gyrations (fine mixes – straight).

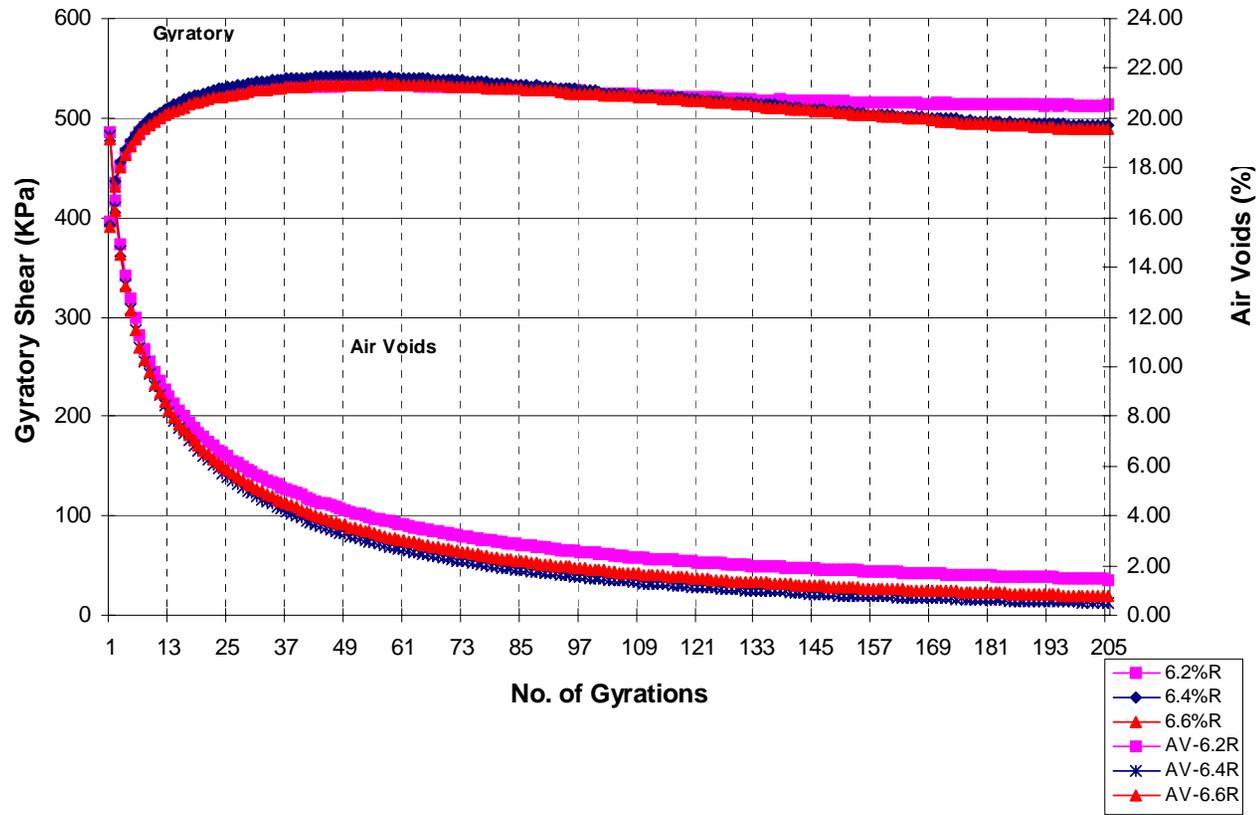


Figure 4.5 Gyrotory shear and air voids vs. gyrations (fine mixes – rubber).

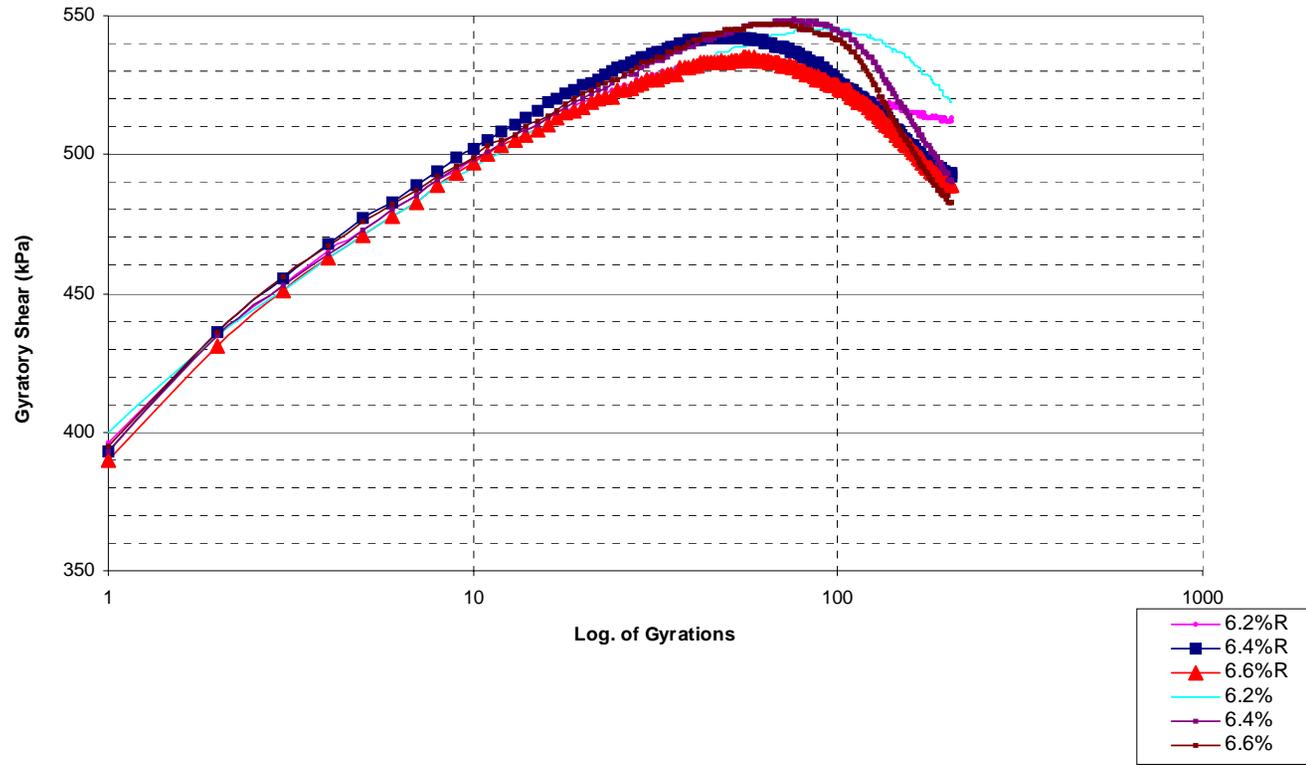


Figure 4.6 Gyrotory shear vs. log number of gyrations (fine mixes).

Table 4.3 Summary of SGC Compaction Data–Gyratory Shear Resistance

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			
		6.1	541	487	10.0			
		7.2r	509	489	3.9	5.3	96.1	
		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			
		6.2	546	519	4.9			
		6.6r	535	489	8.6	5.4	91.4	
		6.4r	542	492	9.2		90.8	
		6.2r	533	542	-1.7		101.7	
White rock + Cabbage Grove fines	coarse	7.4	543	483	11.0	9.5	89.0	90.5
		6.9	555	493	11.2			
		6.4	570	535	6.1			
		7.4r	539	504	6.5	4.8	93.5	
		6.9r	553	524	5.2		94.8	
		6.4r	550	535	2.7		97.3	

7.2% total binder contents) have a lower percentage drop in the gyratory shear value. Also the final value of the gyratory shear of the modified mixtures at the end of the compaction process is higher than that of the unmodified or straight mixes.

In other words, the rubber-modified mixtures sustained a higher percentage of the maximum gyratory shear after the peak value (even though the maximum shear value was found to be lower than that of the straight mixes). This is also illustrated in Table 4.3 by the parameter

R_{fp} (the ratio of the final gyratory to the peak gyratory shear), which is generally higher for the modified samples. Also at the higher binder content (7.2%), it was noted that the unmodified mixes showed signs of binder drain-down. This was not observed in the rubber-modified mixes at the same asphalt content. 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. The above observations lead to the following possible deductions for the coarse aggregate mixes:

1. GTR does not seem to increase the shear resistance of Superpave mixes/pavements. It sometimes slightly reduces the maximum shear strength attainable.
2. However, GTR does seem to sustain the maximum shear resistance, once attained, of an asphalt mix/pavement.
3. GTR may permit the addition of more binder (for coarse aggregates) without substantial loss in the shear resistance of the mix, and without encountering problems of drain-down.

The behavior observed may be explained by the fact that as the mixture approaches “saturation point” during compaction (i.e., approaches zero air voids) the internal stresses within the mixture are transferred from the aggregates to the binder. An asphalt mix or pavement would become unstable when the air voids reaches a critical level. At this critical level the mixes’ resistance to shear is significantly reduced because the binder begins to carry more stresses within the binder-stone matrix. This phenomenon can be likened to, or is analogous to the behavior of a saturated soil. The straight binder with less resistance to flow compared to the rubber binder, flows more easily within the asphalt/stone matrix, which causes (or facilitates) the aggregates to slip or roll over each other as the air voids approaches zero. This results in the reduction of the shear stress of the sample. The rubber-modified binder on the other hand, being

stiffer due to the presence of rubber has more resistance to flow, and maintains a higher shear stress. Thus, at the critical air void level, the rubber-modified binder adds to the stiffness of the mixture matrix, thereby sustaining the resistance of the mixture and making it more stable.

Fine Aggregate

Plots of the gyratory shear strength versus number of gyrations for the fine aggregates showed similar behavior as the coarse aggregate but to a lesser degree (Figures 4.4 - 4.6). The drop from the peak to final gyratory shear strength is almost the same for both the modified and unmodified mixes. This could probably be attributed to the fact that the fine aggregates have considerable amounts of fine particles that increases the apparent viscosity of the binder thereby causing it to behave as a stiff binder. This seems to indicate that the mixture properties of fine aggregates are not affected much by the modified binder. Since the results suggested that the modified binder did not have any significant effect on the fine mixes, testing efforts in subsequent tests were concentrated on the coarse mixes.

Differences in Aggregate/Mixture Structure

Another important aspect of the behavior of the mixes is observed when the gyratory shear of the modified and straight mixes is compared at 1-8% air void levels. Plots of the gyratory shear stress of the mixes between 1-8% air void content are presented in Figures 4.7 and 4.8 for the coarse and fine mixes respectively. From the plots it is noticed that apart from the fact that the peak gyratory shear for the modified mixes is less than the straight mixes, they occur at a relatively higher air void (or VMA).

Content

The resistance to compaction (as measured by the gyratory shear strength, G_s) is related to the degree to which the aggregates interlock and interact with each other in the HMA mix.

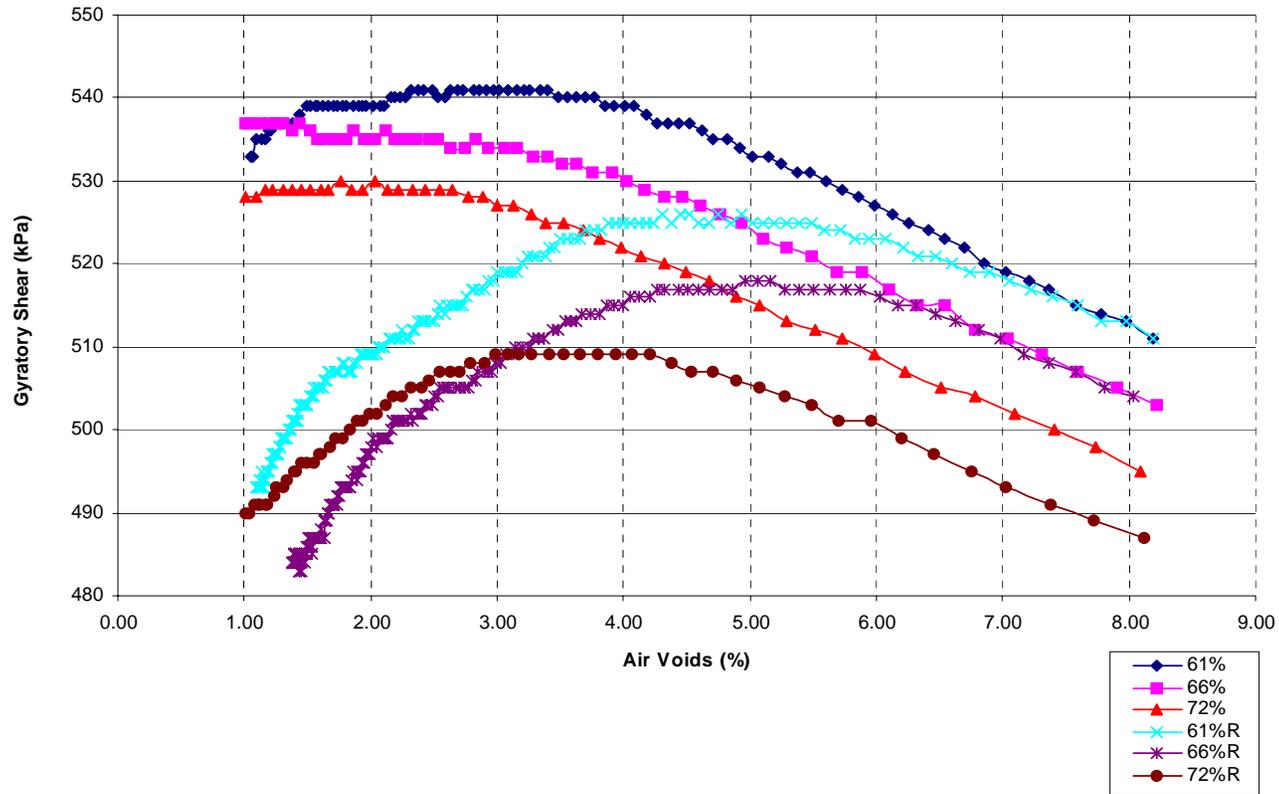


Figure 4.7 Gyrotory shear strength between 1-8% air voids (coarse mixes).

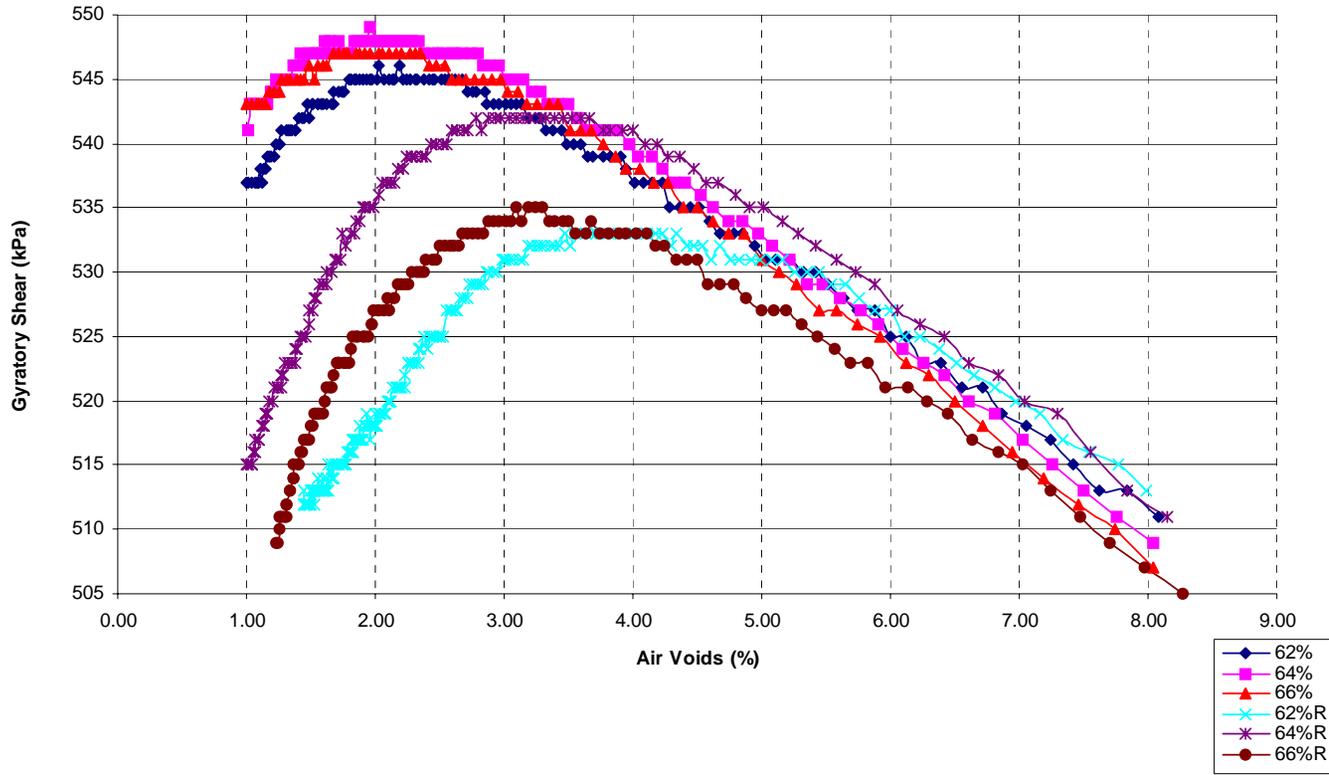


Figure 4.8 Gyrotory shear strength between 1-8% air voids (fine mixes).

Thus the lower shear values at higher air void content seems to suggest that the GTR somehow interferes in the process of aggregate interlocking or interaction which subsequently yields lower gyratory shear values and also at higher air void contents. This observation may not be observed or captured by the binder tests alone. Thus even though GTR-modified binder seems to be stiffer, the overall effect is a reduction in the G_s values as observed in the compaction history of the samples.

4.3.1.2 Areas under the gyratory shear vs. volumetric strain curve

The gyratory shear strength of the mixes was plotted against the volumetric strain of the samples during compaction. The areas under the various sections of these curves are empirical parameters that can be said to correlate with the energy required to compact the samples during the various stages of the compaction process. A schematic diagram with details of the various sections of a typical plot of the gyratory stress-volumetric strain curve is presented in Figure 4.9. Plots of the shear-strain graphs mentioned earlier are also presented in Figures 4.10 and 4.11, respectively. As illustrated in Figure 4.9, the gyratory shear vs. volumetric strain curve can be categorized into four sections. 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 process. The area under the portion of the curve represents the energy required to compact the sample up to the locking point. The locking point has been referred to as 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.

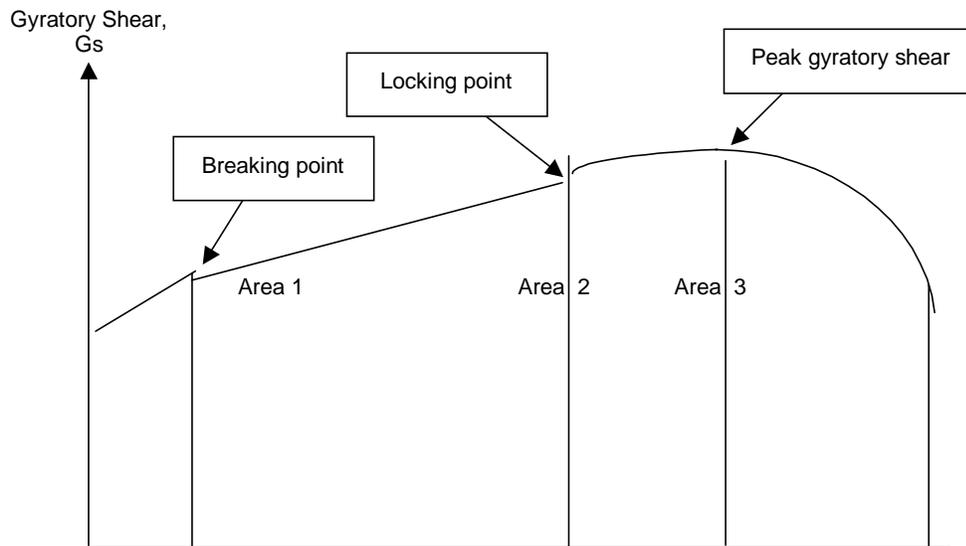


Figure 4.9 Schematic diagram of gyratory shear vs. volumetric strain.

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. A summary of the gyratory shear, volumetric strain, and the locking point (i.e., the number of gyrations at which it occurs) is presented in Table 4.4. To investigate the effects of GTR on the mixtures, the areas under the gyratory shear-volumetric strain curve as described above (Areas 1, 2, and 3) were estimated. A summary of the estimated areas under the various curves of the modified and unmodified samples is provided in Table 4.5.

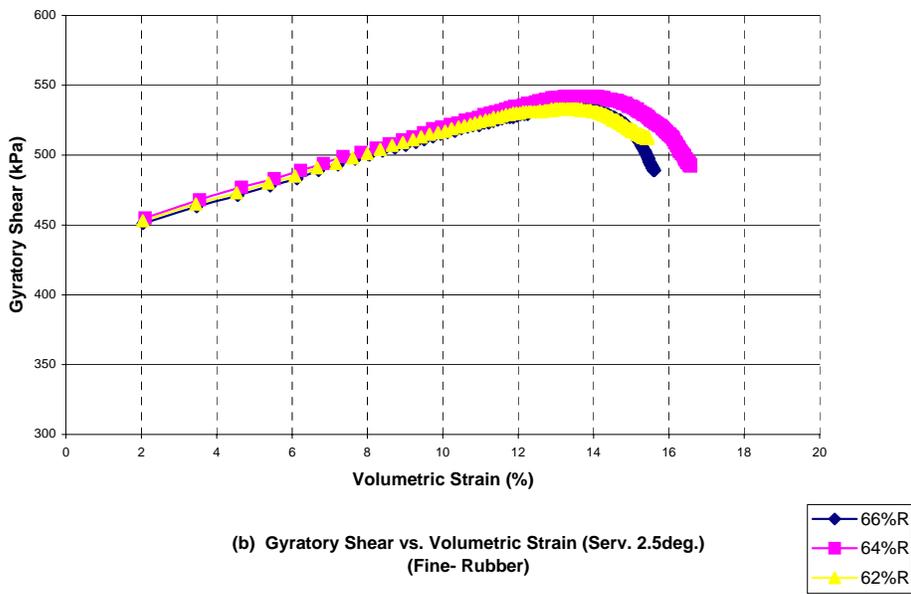
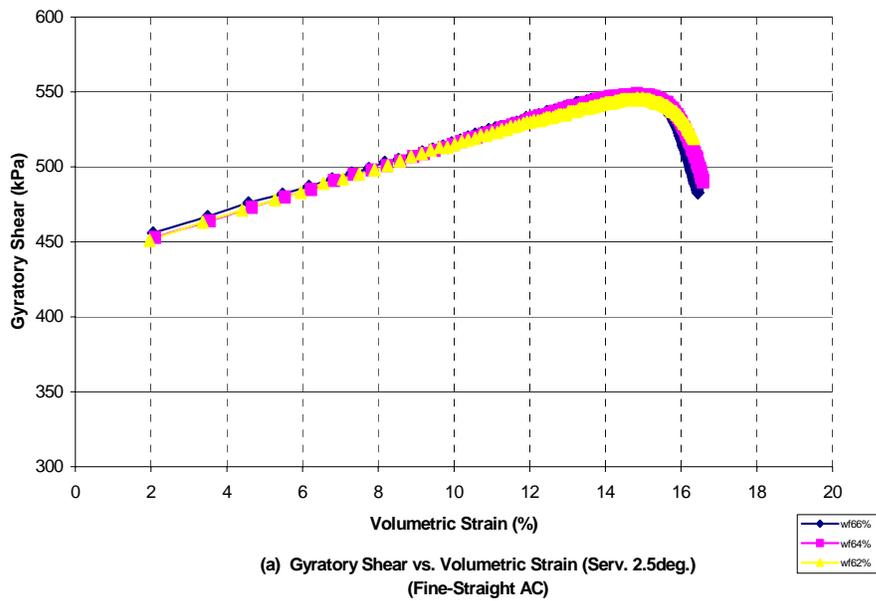
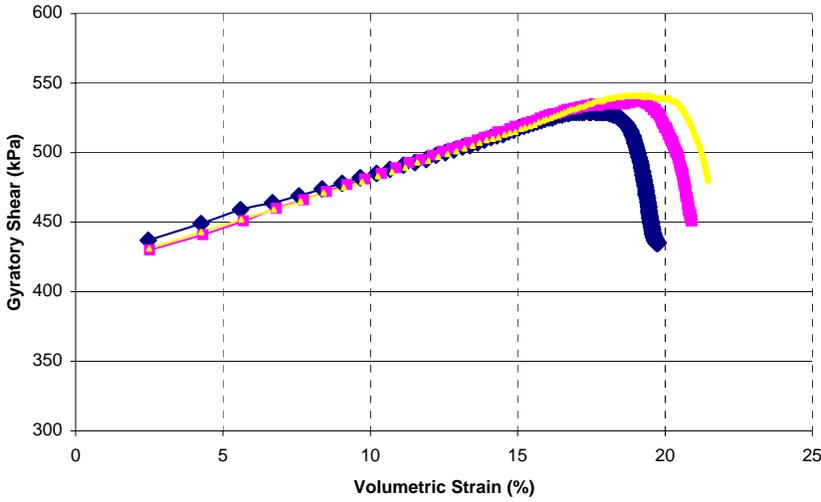
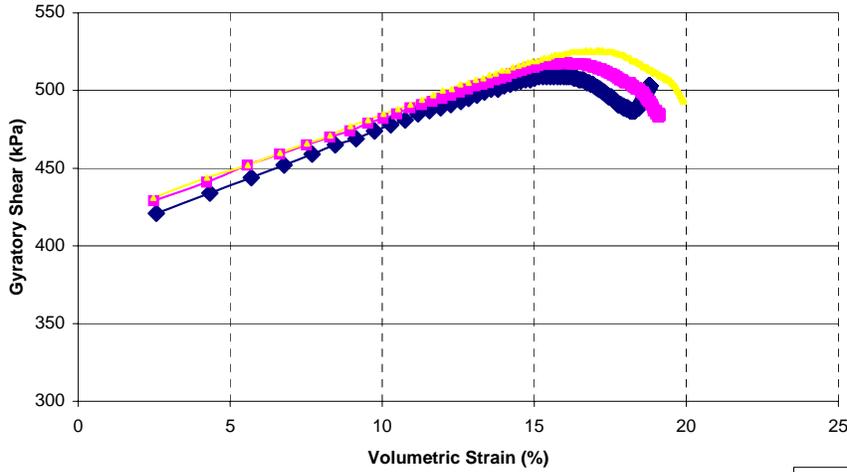
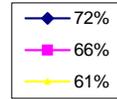


Figure 4.10 Gyrotory shear vs. volumetric strain curves - fine mixes.



(a) Gyratory Shear vs. Volumetric Strain, Serv. 2.5deg.
(Coarse-Straight)



(b) Gyratory Shear vs. Volumetric Strain, Serv. 2.5deg.
(Coarse- Rubber)

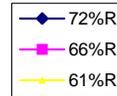


Figure 4.11 Gyratory shear vs. volumetric strain curves - coarse mixes.

Table 4.4 Summary of Areas Under Gyrotory Shear vs. Volumetric Strain Curves

Sample		Area 1	Area 2	Area 3	Areas 1+2	Areas 2+3
W.R. coarse	7.2	1548.14	8740.98	357.37	10289.12	9098.35
	6.6	1515.81	9184.13	499.96	10699.94	9684.09
	6.1	1531.22	9297.15	681.49	10828.37	9978.64
	7.2r	1517.83	7910.77	495.60	9428.60	8406.37
	6.6r	1519.23	8116.09	586.98	9635.32	8703.07
	6.1r	1510.46	8405.81	762.26	9916.27	9168.07
W.R. fine	6.6	1339.59	7117.72	392.63	8457.31	7510.35
	6.4	1450.46	7068.67	605.33	8519.13	7674.00
	6.2	1326.22	6825.97	562.62	8152.19	7388.59
	6.6r	1398.46	6637.61	1166.14	8036.07	7803.75
	6.4r	1432.27	6910.93	1088.24	8343.20	7999.17
	6.2r	1353.91	6555.36	1207.68	7909.27	7763.04

Resistance to Plastic Deformation (Shear failure)

Plastic deformation is primarily due to the lateral displacement of materials of an asphalt pavement. Looking at the areas beneath the shear-strain curve as described above Area 3 seems to be more related to resistance to permanent plastic deformation (or shear failure). The higher the area, the higher the energy required to work on the sample or distort the sample, which seems to suggest a higher or increased resistance to plastic deformation or rutting. The results in Table 4.5 indicate that for Area 2, i.e., from the breaking point to the peak, the unmodified samples have higher areas than the modified samples. However, for Area 3, i.e., from the peak to the end of the compaction process, the modified samples have higher area indicating higher amounts of energy needed to distort the sample within that region. This is in line with the initial idea that the post-peak drop in the gyrotory shear is less for the modified samples than the unmodified samples (indicated in Table 4.1). Nonetheless, a look at the total areas in Table 4.5 (i.e., Areas 2

+ 3) tends to suggest that the rubber-modified samples have lower total areas and most probably lower total energies or overall resistance to shear failure.

Table 4.5 Summary of IDT Test Results

A) Short-term Oven Aging (STOA)									
Sample (% AC)	Property								
	Average Resilient Modulus (Gpa)	Average Creep compliance (1/Gpa)	Average Strength (Mpa)	Fracture Energy (kJ/m ³)	Failure Strain μ -strain)	m-value	ϵ_o ¹	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
B) Long-term Oven Aging (LTOA)									
Straight AC:									
6.1	10.76	4.29	2.13	2.30	1579.00	0.62	1381.04	0.21	2.09
7.2	8.75	9.70	1.66	5.10	3449.09	0.58	3259.38	0.16	4.94
Rubber:									
6.1r	11.64	1.44	1.63	0.90	764.61	0.50	624.58	0.11	0.79
7.2r	9.72	2.74	1.65	1.40	1182.94	0.59	1013.19	0.14	1.26

1. $\epsilon_o = (Mr \times \text{failure strain} - \text{IDT strength})/Mr$
2. Elastic energy = $0.5 \times Mr \times (\text{failure strain} - \epsilon_o)$
3. DE_{cs} (dissipated creep strain energy) = fracture energy - elastic energy

4.3.1.3 Rate of change of gyratory shear with respect to air voids (dG_s/dAV)

One parameter of HMA mixes being investigated is the rate of change of gyratory shear per unit change in air voids (dG_s/dAV , hereafter denoted by dG_s). 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. From the data the parameter dG_s was determined for the modified mixtures and straight mixtures (presented in Figures 4.12 and 4.13). The plots indicate that the values of dG_s 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. (2001), the parameter dG_s was ranked with the known field performance of some mixes and it was noticed that mixes with higher dG_s did not perform well compared to those with low dG_s values. In addition these mixes had low strain tolerances (measured by their volumetric strain). This is also confirmed by the volumetric strains at the peak gyratory shear and locking point mentioned earlier on in Table 4.6. Thus mixes with good field performance (low dG_s) seem to gain in strength slowly over a large range of volumetric strain where as mixes with inadequate field performance seem to gain shear resistance quickly over a lower range of volumetric strain. This seems to suggest that the GTR does not improve the field performance of HMA mixes.

4.3.1.4 Servopac gyratory compaction with 2nd aggregate type: Low LA abrasion and very low DST

Mixtures produced with the second aggregate type were compacted with the Servopac at a gyratory angle of 2.5°. The samples were mixed and compacted at 300°F and 275° F, respectively. The results (presented in Figure 4.15 - 4.16) indicate a similar effect of GTR on the maximum gyratory shear strength of the samples. This can be seen from the shear values for both the modified and unmodified mixtures. Also a look at the compaction data indicates that

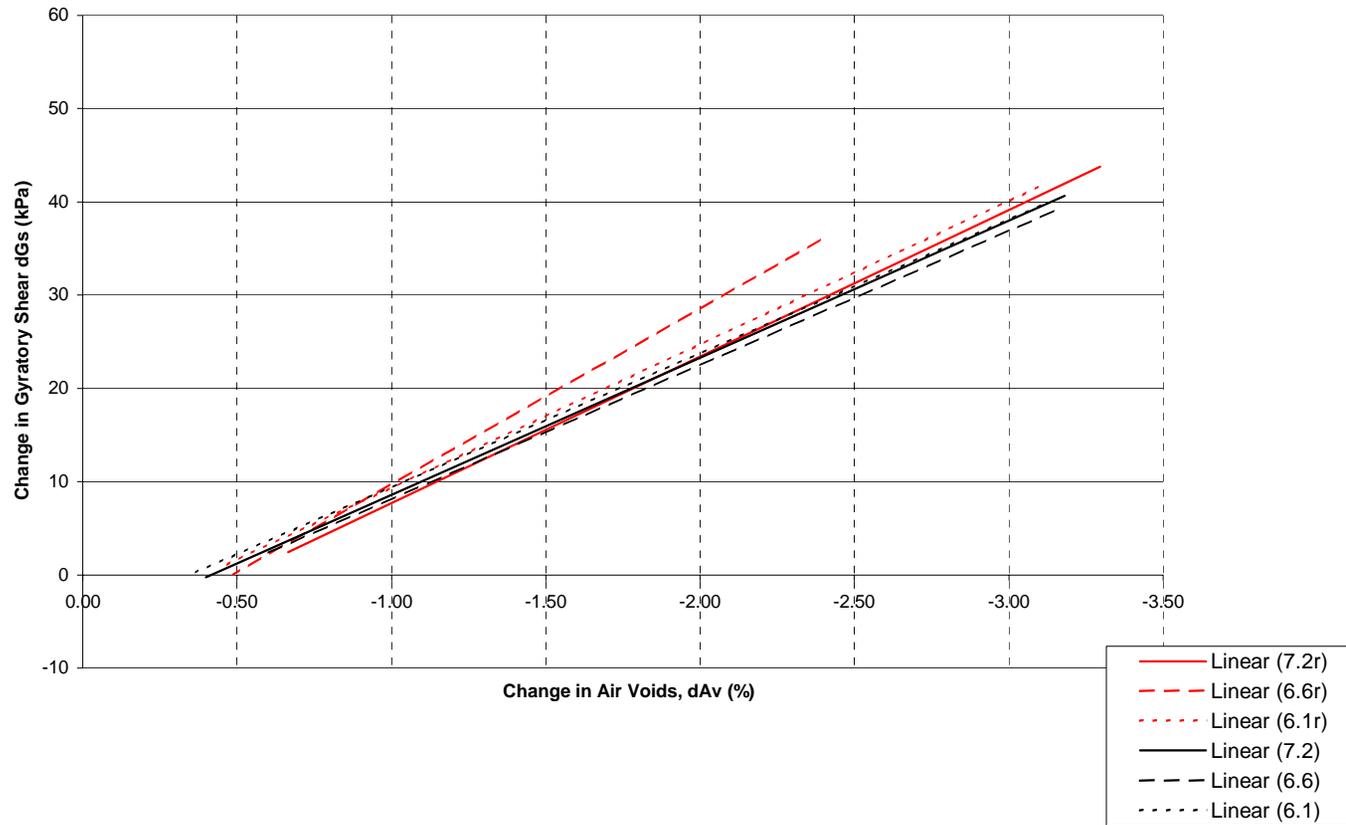


Figure 4.12 Change in gyrotory shear vs. change in air voids – coarse mixes.

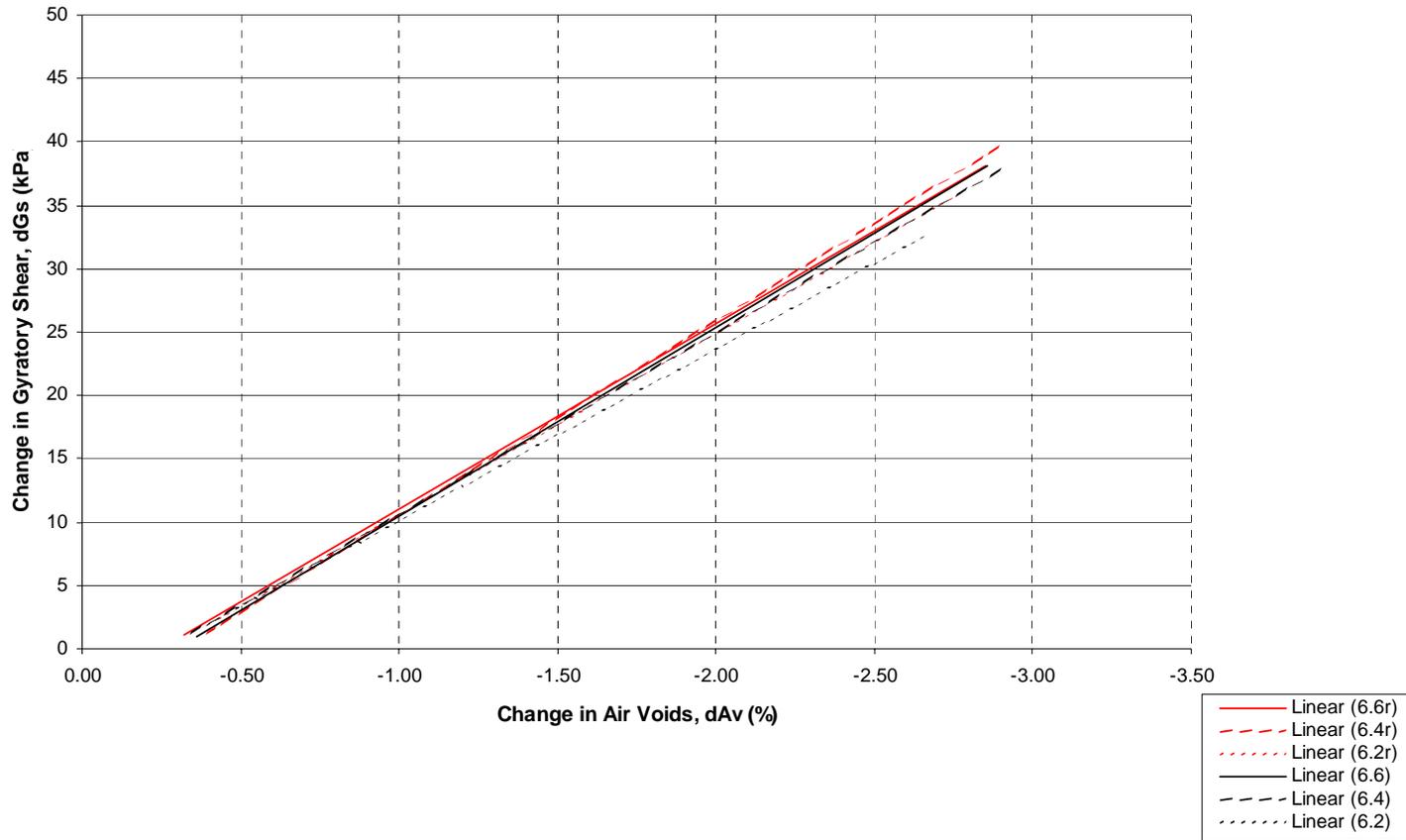


Figure 4.13 Change in gyrotory shear vs. change in air voids – fine mixes.

Table 4.6 Summary of Compaction Data at Peak Gyratory Shear and Locking Point (LP)

a) Data at Locking Point

Sample		No. of Gyration at LP	Gyratory Shear at LP (kPa)	Volumetric Strain at LP	VMA at LP
W.R. coarse	7.2	42	529	16.82	15.73
	6.6	42	533	17.11	15.71
	6.1	46	534	17.30	15.28
	7.2r	34	509	15.76	16.67
	6.6r	33	518	15.26	16.14
	6.1r	41	525	16.20	15.18
W.R. fine	6.6	47	544	13.50	15.15
	6.4	42	541	13.14	15.34
	6.2	38	532	12.37	15.92
	6.6r	40	532	12.56	15.63
	6.4r	35	539	12.56	15.27
	6.2r	36	531	12.13	15.55

b) Data at Peak Gyratory Shear

Sample		No. of Gyration at Peak	Gyratory Shear at Peak (kPa)	Volumetric Strain at Peak	VMA at Peak
W.R. coarse	7.2	56	530	17.56	14.93
	6.6	70	537	19.07	13.63
	6.1	71	541	18.96	13.54
	7.2r	35	509	15.86	16.57
	6.6r	40	518	15.99	15.41
	6.1r	52	526	17.05	14.31
W.R. fine	6.6	67	547	14.52	14.13
	6.4	74	549	14.83	13.66
	6.2	97	546	14.98	13.66
	6.6r	56	535	13.46	14.75
	6.4r	50	542	13.65	14.24
	6.2r	56	533	13.31	14.39

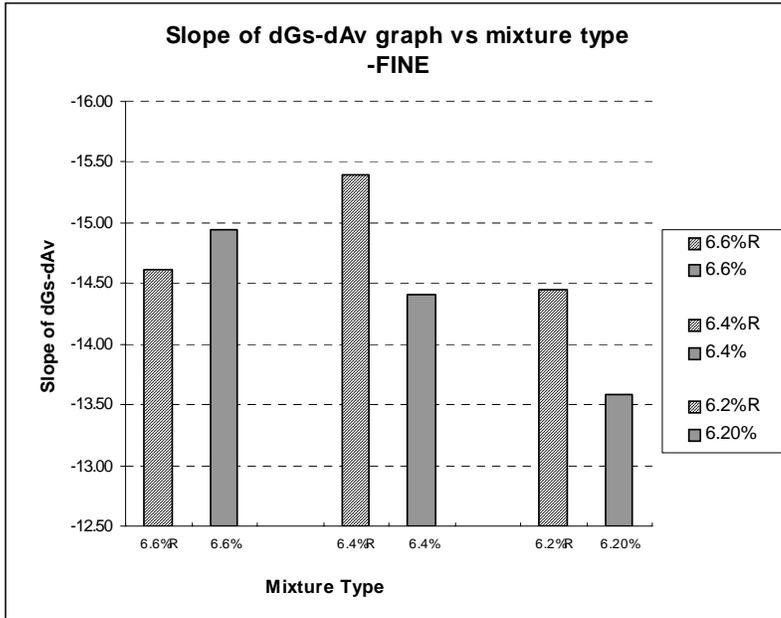
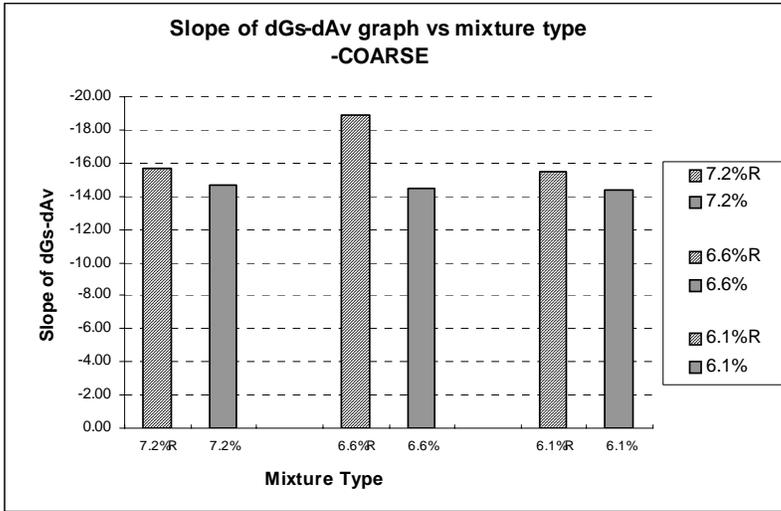


Figure 4.14 Bar chart showing differences in dG_s / dA_v .

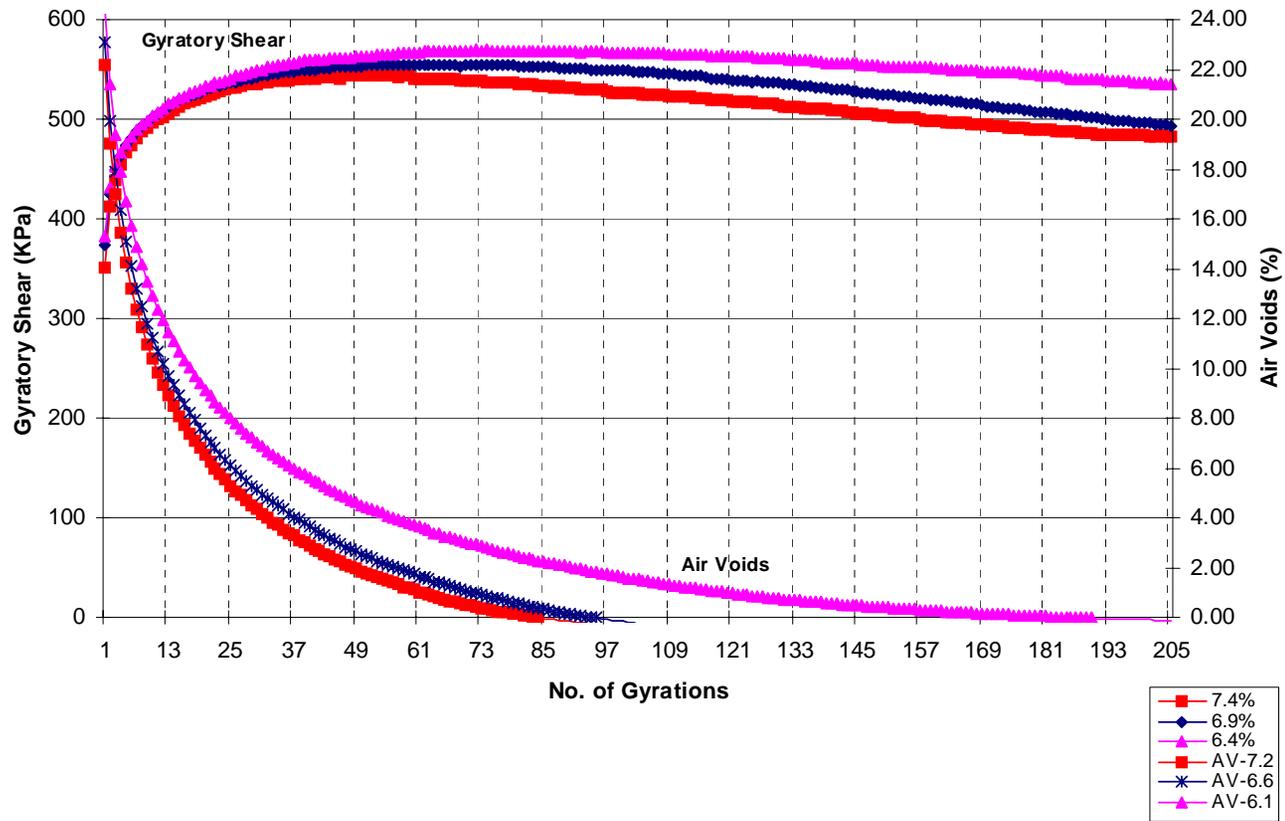


Figure 4.15 Gyratory shear and air voids vs. gyrations (white rock + Cabbage Grove fines – straight mixes).

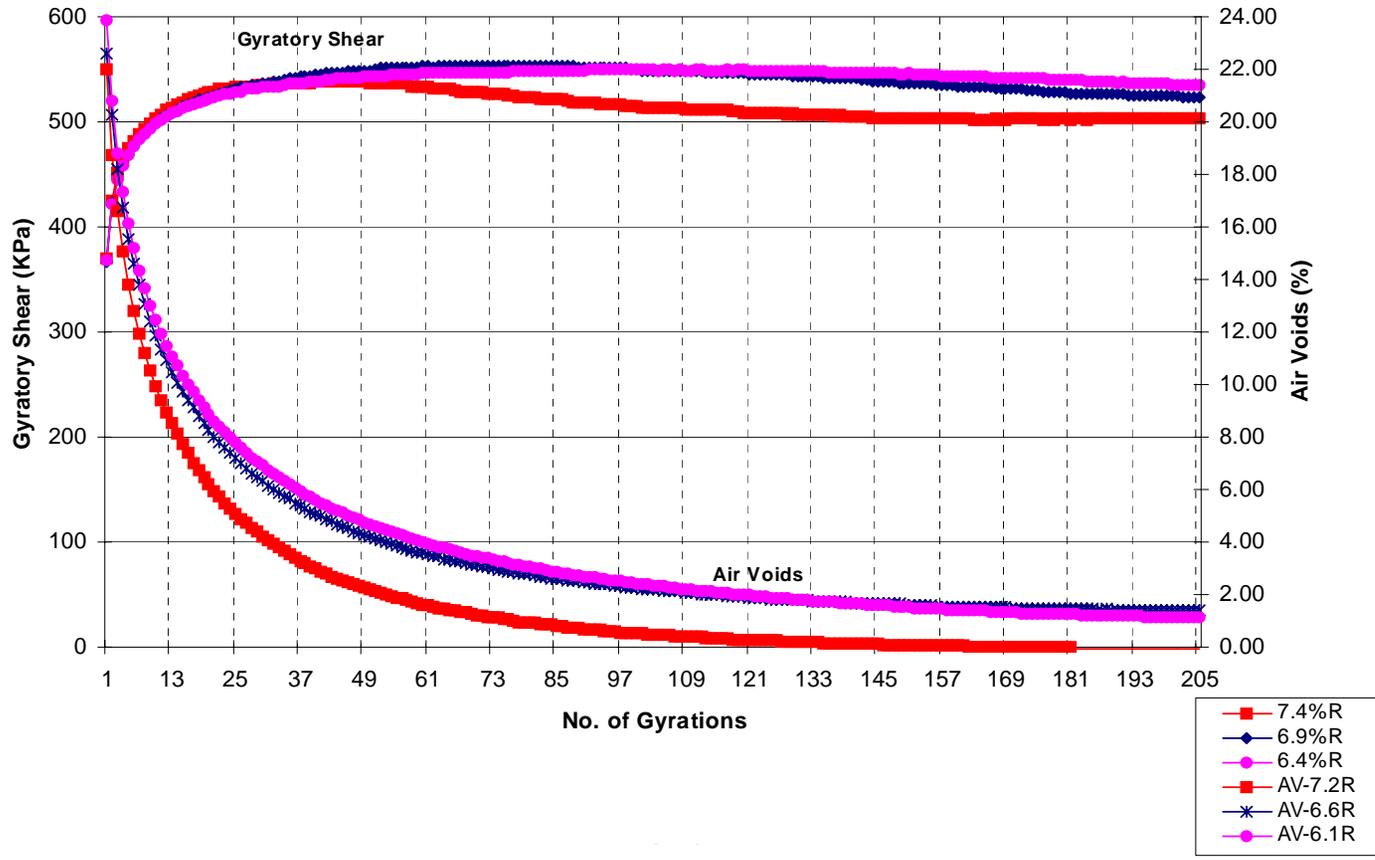


Figure 4.16 Gyrotory shear and air voids vs. gyrotations (white rock + Cabbage Grove fines – rubber).

for the modified mixes a higher percentage of the peak gyratory shear is maintained to the end of the compaction process. Also the air voids during the compaction process is higher for the modified samples as compared to the straight samples, which was found to be true for the first aggregate type. From the above, similar conclusions made for the stronger aggregates can also be made for the aggregates with high LA abrasion values with regard to the effect of GTR on the mixtures. The GTR essentially enables one to increase the binder content for the mixes while maintaining approximately the same level of shear strength of the sample.

4.4 IDT Tests - Resilient Modulus, Creep Compliance, and Indirect Tensile Strength

Rubber-modified samples were produced for tests on the Superpave IDT Test set-up to further evaluate the effects of GTR on crack resistance of the mixes. The samples were tested according to procedures developed by Roque et al. (1997). Both LTOA and STOA samples were produced. The following properties were measured for both aged and un-aged samples: resilient modulus, creep compliance, indirect tensile strength, fracture energy density, failure strain and m-value. Results of the test are presented in Table 4.5 and Figures 4.17 through 4.19.

4.4.1 Short-term Oven Aged Samples (STOA): Resilient Modulus (Mr)

The samples were tested at 10° C. The results of the test indicate that the rubber increases the resilient modulus of the STOA mixes, even though slightly, at the test temperature of 10° C. The Mr of the WR1 (6.1% rubber) was almost the same as the W1 sample. Also the Mr for the WR3 (7.2% rubber) sample was slightly higher than the W3 samples. The increase in the resilient modulus of the mixes most probably can be attributed to the increased stiffness of the rubber-modified binder.

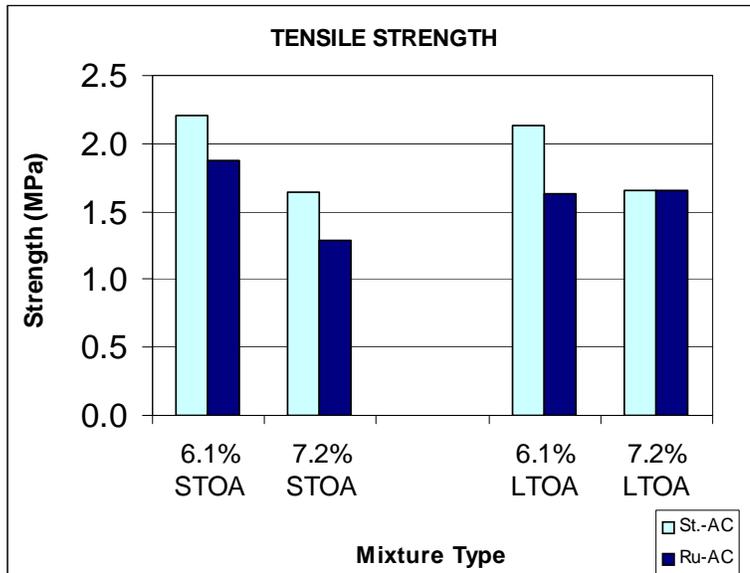
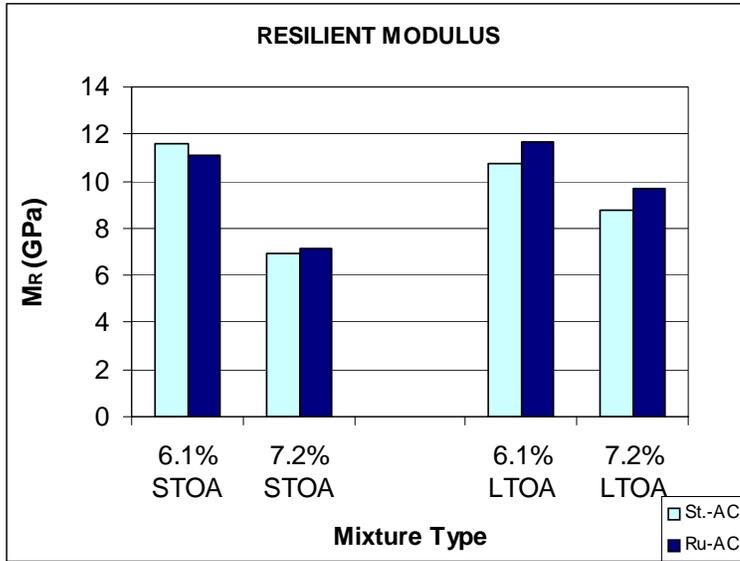


Figure 4.17 IDT test results: Resilient modulus (M_R) and indirect tensile strength.

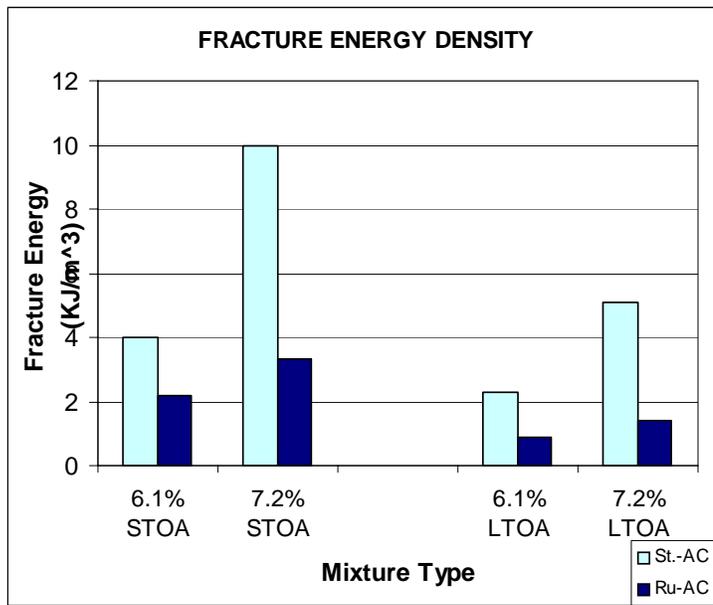
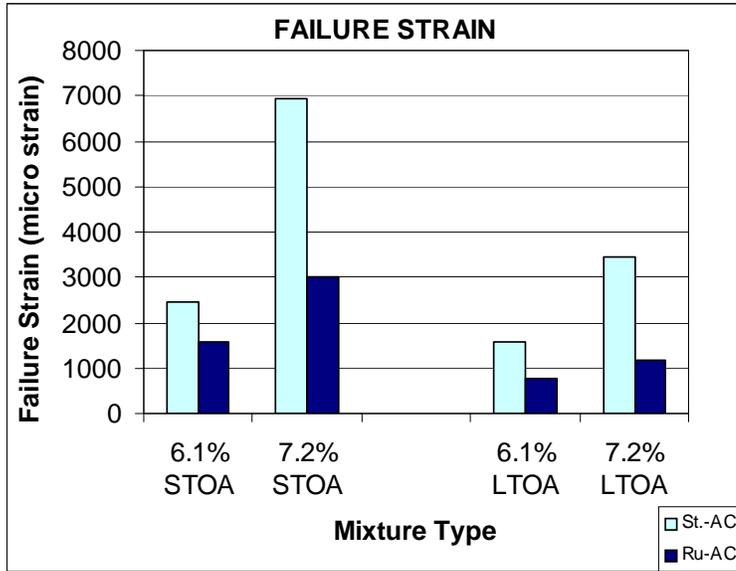


Figure 4.18 IDT test results: Failure strain and fracture energy density.

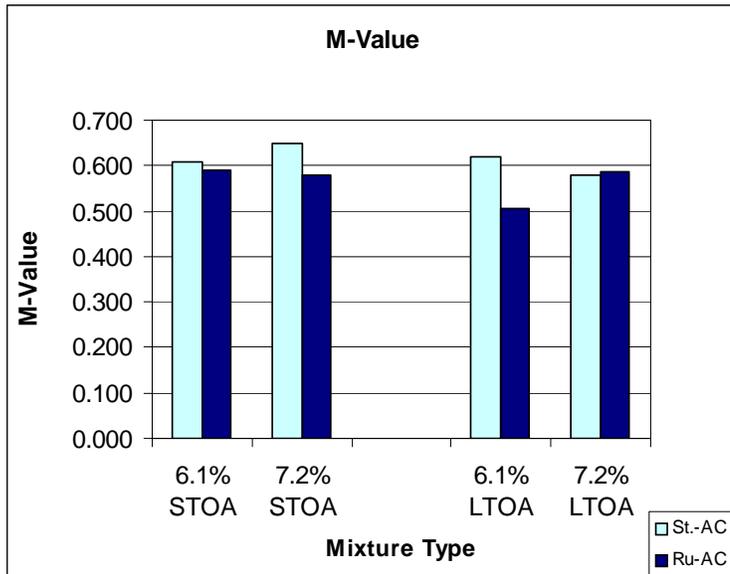
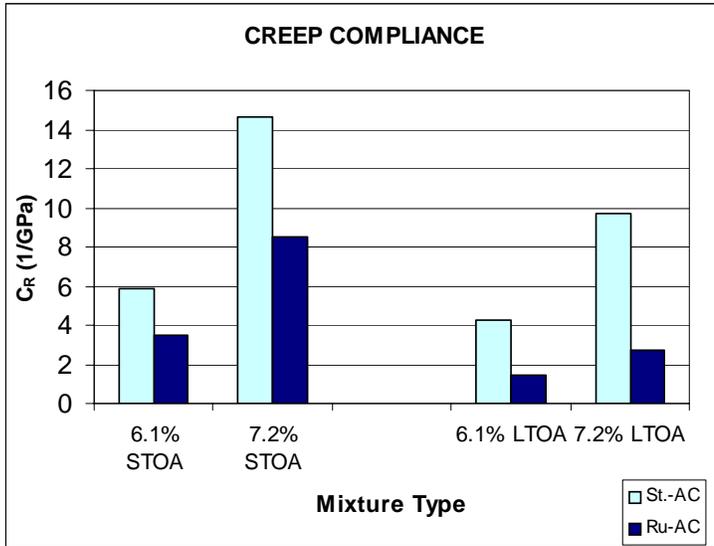


Figure 4.19 IDT test results: Total creep compliance and m-value.

Creep Compliance

The results of the creep compliance of the rubber samples at 1000 seconds indicate that after STOA there is a decrease in the creep compliance of the rubber mixes (measured by the creep at 1000 seconds), as compared to the straight samples. The modified mixes, having binders with higher viscosities, decreased the creep compliance, which seems to correlate well with the observed binder properties. This however seems to indicate a decrease in the low-temperature cracking resistance of the modified mixtures due to stiffening of the mixture as a whole as a result of higher stiffness of the rubber-modified binder. At 10° C (50° F, considered low pavement temperature in Florida), a desirable property of HMA would be the ability to relieve itself of potential stress build-up within the pavement leading to a more crack resistant pavement at low temperatures.

Thus a reduction in the creep compliance at 50° F does not seem to improve the overall cracking performance of the mixture or pavement since it indicates a reduction of the mixtures ability to dissipate (or relieve) potential stresses built up within the system which reduces its low-temperature cracking resistance.

Indirect Tensile Strength

The tensile strength of a HMA sample is the maximum tensile stress it can accommodate before it fractures. A look at the results indicates a general decrease in the indirect tensile strength of the rubber-modified samples as compared to the straight samples. Even though the viscosity of the modified binder is higher than the straight asphalt, it does not translate into an increase in the tensile strength of the asphalt mixture. One possible explanation of the strength results could be attributed to the presence of the ground tire rubber. The results of the modified samples seem to suggest that the GTR behave like minute “discrete” grain particles instead of

being “dissolved” in the AC-30 to form a homogeneous modified binder. This assumption would mean that the minute rubber particles within the mixture mastic (which have a high surface area) decreases the amount of asphalt needed 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 modified samples.

Fracture Energy and Failure Strain

The fracture energy of an asphalt mixture is the energy per unit volume required to cause fracture of an asphalt mixture. The failure strain is the maximum tensile strain of the material immediately prior to failure. The higher the fracture energy and failure strain the higher the crack resistance of the mixture. A look at the IDT test results indicates that there is a decrease in the fracture energy and failure strain of the modified samples compared to the straight samples. As suggested above, one possible explanation of the behavior of the samples may be due to the presence to the rubber particles “dispersed” within the mixture matrix. The GTR particles, which pass sieve size #80, seem to behave as discrete solid particles dispersed within the asphalt aggregate matrix, instead of being “dissolved” in the asphalt binder. As a result, the rubber particles, which have a high surface area, seem to reduce the effective amount of asphalt that would bond with the aggregates. Also the presence of the dispersed rubber particles seems to facilitate the growth of micro-cracks within the binder mastic thereby reducing the fracture energy and failure strain, respectively.

m-Value

The m-value is the slope of the log creep compliance versus log time graph. Recent research at the University of Florida indicates that the m-value is an important parameter that is

related to the crack growth rate of HMA mixes. Field data seems to indicate that higher m -values are associated with good cracking performance. One possible explanation of the above-mentioned observation is that mixtures with high creep response or m -value have the ability to relieve itself of stress build-up upon the application of the said stress, and subsequently exhibit higher strain tolerances. A look at the m -values from the IDT test results (presented in Figure 4.19) indicates that the modified rubber samples exhibited a decrease in the m -value as compared to the straight samples.

The lower m -value could be attributed to the stiffer binder of the rubber-modified mixes. A low m -value implies a slow rate of stress dissipation within the asphalt mix when a stress is applied to the asphalt mixture. The stress build up would eventually reach a limiting point where it exceeds the tensile strength of the mixture, which eventually causes the sample to fail or fracture. This seems to suggest that the rubber samples may have lower resistance to low temperature cracking compared to the straight samples. This position is further reinforced by a reduction in the crack-related parameters, namely fracture energy density, failure strain and indirect tensile strength. Thus the low m -values of the modified samples suggest that the rubber does not improve the fracture resistance of the mixtures.

4.4.2 Long-term Oven Aged Samples (LTOA)

Aged samples were produced to evaluate the effects of aging on the aforementioned mixture properties. The gyratory-compacted samples were aged at a temperature of 185° F for five consecutive days to simulate LTOA according to AASHTO PP2-94. Long-term aging causes the binder and consequently the mixture to become stiffer through the process of oxidation and the loss of more volatile components of the binder over a period of time. The mixture therefore becomes stiffer thereby affecting its response to the application of stress or strain.

Resilient Modulus, Mr

Reference to the summary of results for the IDT tests in Figure 4.16 indicates a general increase in the Mr for both the W1 and W2 samples after LTOA. The WR1 and WR2 samples also recorded a 10-15% increase in the Mr. There is thus a consistent increase in the Mr of the samples after LTOA.

Creep Compliance

The creep resilience for the LTOA samples follows the same trend as the STOA mixes except to a much greater degree. It is indicated in the figure that after LTOA the decrease in the creep compliance of the rubber mixes is as much as about 60% compared to the unmodified mixes. This can be most probably attributed to a much stiffer binder after LTOA.

Fracture Energy and Failure Strain

The general effect of LTOA on the fracture energy and failure strain of the straight and modified mixes is presented in Figure 4.16. It can be seen that a similar trend is observed in the STOA samples. Both the fracture energy and failure strain of the modified binder are reduced by as much as 30-50%. This reduction in the fracture energy and the failure strain of the modified samples suggest that the rubber does not improve the fracture resistance of the modified samples after LTOA. Rather it contributes to further reduce the fracture resistance of the samples at the test temperature.

CHAPTER 5 CLOSURE

5.1 Summary of Findings

Hot Mix Asphalt samples modified with ground tire rubber, GTR (#80) were produced in the laboratory with the Superpave mix design procedure, and tested to determine their mixture characteristics in relation to rutting and cracking resistance. The produced samples were tested with the Australian Servopac gyratory compactor at gyratory angles of 1.25° and 2.5°. Also they were tested by the indirect tensile test (IDT) using the system developed by Roque et al. (1997). The fracture energy density, failure strain, resilient modulus, tensile strength and creep compliance were determined from the IDT test results at 10 ° C. The mixtures were also aged in an attempt to determine the effects of the hardening of the modified binders on mixture properties.

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

1. The Servopac results seem to indicate that the GTR affects the aggregate structure of the mixes in such a way that prevents it (i.e., the aggregates) from achieving its optimum orientation, which would provide the maximum shear resistance of the HMA mixture. This observation relating to the structure of the mixes would not have been captured by the differences in the binder properties.
2. Also the GTR seems to sustain the post-peak or maximum shear strength of the mix during or throughout the compaction process of the mix. This seems to suggest that the GTR increases the stability of the mix at low air void contents, even at higher asphalt contents.

3. With reference to the Indirect Tensile Tests, the rubber-modified mixes had lower creep compliance which seems to buttress the point that the GTR probably could increase the shear resistance and hence the rutting resistance of the mixes.
4. The rubber-modified samples also indicated a general increase in the resilient modulus (or stiffness) of the mixes, even after age hardening.
5. In spite of the above observations, the rubber-modified samples seem to have reduced indirect tensile strengths and lowered fracture energy densities at 10° C, which tend to suggest that the modified samples have reduced cracking resistance at intermediate temperatures. However, a comparison between the straight and modified samples did not show a significant difference in the failure strain of the samples.
6. The rubber-modified samples showed a significant reduction in the failure strain and fracture energy densities of the mixes after age hardening. Compared to the straight samples after LTOA, the results indicate a higher decrease in the fracture and failure strain for the modified samples as compared to the straight samples. This suggests that the GTR modifier enhances the effects of age hardening and thus does not sustain or improve the crack resistance of the mixes after age hardening.

5.2 Conclusions

The effects of GTR on OGFC had been researched and compared with field sections constructed within Florida by Otoo (2000) at the University of Florida. Results from that study suggested that the addition of rubber to asphalt reduces the temperature susceptibility of the binders, provides a higher resilient modulus at higher binder contents, and maintains a low compliance. However, based on the above findings, the following conclusions were made: about the effects of GTR on HMA mixes:

- The possible benefit of the addition of GTR may be realized through the increase in the rutting resistance of dense graded structural layers and also the reduction temperature susceptibility of the mixes at intermediate temperatures.
- It may also be beneficial by allowing the introduction of higher binder contents without binder drain-down during construction, while substantially maintaining the shear resistance of the mix.
- However, rubber may not seem to help the cracking performance of densely graded structural layers due to a considerable reduction in the fracture energy densities and failure strains.

5.3 Recommendations

The performance of the modified and straight HMA mixes was evaluated by maintaining the same design procedure (and parameters) for both the modified and straight mixes. However, cognizance is given to the fact that some of these parameters, such as temperature, could have an effect on the measured properties of the mixes.

As a result it is recommended that further studies of the effect of temperature on the cracking and rutting performance of HMA mixes be investigated.

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