INTERACTION BETWEEN RECYCLED TIRE RUBBER, POLYMERS AND HIGH AMOUNT OF RECLAIMED ASPHALT PAVEMENTS

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ABSTRACT

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Use of recycled materials has been gaining popularity all over the world to provide a sustainable future for next generations. Crumb Rubber (CR) and Reclaimed Asphalt Pavements (RAP) have been the mostly recycled products in the asphalt industry. Federal and state governments have been searching for alternative methods of either disposing or recycling the large number of waste tires. The use of shredded tire rubber (i.e., crumb rubber – CR) in asphalt pavements is one of the sustainable recycling approaches. Increasing the amount of CR integrated into asphalt pavements reduces various risks involved with stockpiling, burning, damping into landfills and other undesirable disposal methods of scrap tires.

Another benefit of using CR is being an alternative for polymer modification. Polymer modification of asphalt binders has gained quite a large popularity in many transportation agencies, primarily due to the superior crack and rut-resistant performance. However, added cost of polymer modification results in an appreciable increase in the initial cost of an asphalt pavement. Use of Recycled Tire Rubber (RTR) is a more economical and sustainable alternative to polymers.

Yet, another benefit of CR is the binder softening effect. Addition of RAP in new Hot Mix Asphalt (HMA) construction is a useful practice since it reduces the amount of naturally derived aggregates and virgin binder in the mixture. Increased RAP usage also provides economical savings (e.g., reduced freight cost of the virgin materials) and environmental benefits (e.g., conserves energy that is used while extracting natural aggregates and asphalt cement). Due to the

continuous increase in the asphalt binder costs (which is directly related to gasoline costs), utilizing high percentages of RAP (up to 40-50%) is essential for lowering the production costs of asphalt pavements. However, increasing the percentage of RAP in a new asphalt pavement may hinder the economic benefits due to the necessity of expensive softer binders used to compensate for the RAP stiffening effect. Alternatively, this problem may potentially be overcome by use of another recycled material; recycled tire rubber. RTR is known to soften the binder at low/intermediate temperatures, eliminating the need to use soft-expensive binder. Three major contributions of this dissertation can be listed as follows:

- Interaction between RTR and polymer modified binders have been investigated in terms of performance grading (stiffness), fatigue cracking and rutting.
- Interaction between RTR and high volume RAP mixtures has been evaluated in order to eliminate costly soft binders.
- Interaction between dry crumb rubber and crumb rubber terminal blend technologies have been studied in order to increase the crumb rubber content without needing expensive plant equipment for crumb rubber wet technology.

TO MY FAMILY
(Biricik Aileme)
Thank you for always believing in me
(Bana her zaman inandığınız için teşekkür ederim)

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CHAPTER 1 INTRODUCTION

1.1 BACKGROUND

Scrap tire handling has been a common problem all over the world. There are more than 300 million scrap tires buried in the landfills of the United States each year. Federal and state governments have been searching for alternative methods of either disposing or recycling the large number of waste tires. The use of shredded tire rubber (i.e., crumb rubber or ground tire rubber) in asphalt pavements is one of the sustainable recycling approaches. Crumb Rubber (CR) has been used as an additive in asphalt pavements since 1950s (Heitzman 1992; Huang et al. 2002; Sonti et al. 2003). Benefits of CR Modified Asphalt Pavements (CRMAPs) have been acknowledged by numerous researchers (Way 1999; Carlson et al. 1999; Hicks et al. 1999). Most CRMAPs are made with the following three major methods: wet process (CRWet), terminal blend (CRTB) process, and dry process (CRDry).

In CRWet process, CR is added to liquid asphalt at temperatures around 325-400 °F (163-205 ºC) and about 15% - 22% CR by weight of the binder is utilized (1-1.5% by total weight of the mix) (Caltrans 2005; Caltrans 2005). One of the recently re-invented crumb rubber asphalt modified technologies is the so-called “De-Vulcanized Rubber (DVR)”. It uses the similar process as in the CRWet process. The primary advantage of DVR is that, when mixed with asphalt binder, the rubber particles completely dissolve within the binder. The final product CRDev, i.e., the DVR modified binder, is a complete fluid not a suspension. Both the CR and DVR are manufactured from recycled tire rubber (RTR).

The terminal blend (CRTB) process is similar to the wet process, except that less amount of CR is used (~10-12%) and a polymeric additive is used to keep the CR particles suspended in the binder. The characteristics of CRWet and CRTB modified asphalt binders depend on the type of rubber, binder type, size of the CR particles, duration and temperature of the reaction and the modification method (Carlson et al., 1999; King et al., 1999).

CRDry process is the method where the CR particles are added to the mix as a partial replacement of fine aggregates. In addition, there are several treated dry rubber technologies where the crumb rubber particles are pre-mixed with low viscosity petroleum-based products or aromatic oils compatible with the lighter fractions of asphalt binder. These treated rubber technologies are used as a CRDry process where they are added as partial fine aggregate replacement to the mixture. Use of CR in asphalt pavement provides more advantages than just merely dealing with the scrap tire handling problems. One of the benefits of using engineered CR modified binders is to provide an economic alternative to costly polymer modified binders. Asphalt pavements have been experiencing heavier traffic volumes as the population of the world keeps growing in recent years (Nahas et al. 1990). Conversely, the funding shortages for maintenance and higher costs of pavements have been leading to thinner pavements, hence a decrease in the service life (Lewandowski 1994). The use of modified binders has been seen as one of the alternatives to address these problems. Polymer modified binders typically provide more crack- and rut-resistant asphalt mixtures. While they increase the initial cost of the asphalt mixture by 15% to 20% (per ton of mixture) (APAM 2014), the superior performance of polymer modified binders may lead to decrease in pavement thickness, and in turn, a possible decrease in overall life cycle cost. Similar to polymer-modified binders, CR modified asphalt binders have been used by many roadway agencies primarily due to their crack-resistant performance. The unit cost of RTR modified binder is less than the typical polymer modified one, therefore, if they can be used to partially replace the polymer, more economical and sustainable pavements can be constructed.

Another benefit of using CR modifier is the ability to eliminate the necessity of using softer binders in HMAs, which include a high amount of recycled asphalt pavement (RAP). To provide a sustainable future for the next generations, re-use of old materials has been gaining popularity all over the world. Recycling old asphalt pavements is one of the major practices in today's asphalt industry in this effort. Recycled/reclaimed asphalt pavement is obtained by crushing and screening distressed asphalt pavements via milling or full depth removal (Hassan 2009, Copeland 2011). Recycling old asphalt pavement materials optimizes the use of natural resources in the production of hot-mix asphalt (HMA). Use of RAP in new hot mix asphalt (HMA) construction is a useful practice since it reduces the amount of naturally derived aggregates and virgin binder in the mixture. Increased RAP usage also provides economical savings (e.g., reduced freight cost of the virgin materials) and environmental benefits (e.g., conserves energy that is used while extracting natural aggregates and asphalt). Due to the continuous increase in the asphalt binder costs (which is directly related to gasoline costs), utilizing high percentages of RAP (up to 40-50%) is essential for lowering the production costs of asphalt pavements. Even though RAP is inexpensive to the contractors, they are reluctant to use high percentages of RAP since it leads to use of softer and expensive binders. Moreover, the requirement for blending charts analysis (where low and high temperature grade of the binder is determined based on the amount of RAP used and the grade of the binder in the RAP) for high RAP mixtures is another disadvantage for contractors because of the extensive time consuming process and use of special equipment. Blending chart analysis and necessity of expensive softer binder to compensate the RAP stiffening effect may hinder the economic benefits of increasing the percentage of RAP. Alternatively, this problem could potentially be overcome by use of another recycled material; crumb rubber.

1.2 PROBLEM STATEMENT

Crumb rubber modification processes have different advantages relative to each other. According to literature, CRMAPs using CRWet process have provided better performance than the other CR modifications. However, because of the necessity of expensive special mixing and reacting equipment at the asphalt plant to produce CRWet, CRTB has been used widely by the asphalt manufacturers in certain states.

Although CRTB has production privilege for asphalt contractors, the fact that it can only accommodate restricted amount of crumb rubber because of suspension and mixing limitations at the binder plant and comparatively higher price makes it unprofitable for most of the state DOTs. For instance, the States of Arizona and California limit the minimum CR modification to 15% for CRMAPs. These deficiencies reveal the need of a new method which can be achieved economically without the requirement of special equipment, be easily acquired in the plant by contractors and integrate more CR into asphalt mixtures.

The new method suggested in this dissertation work achieves integration of more crumb rubber as in CRWet process and provides ease of construction. In practice, it would be preferable by both State DOTs and asphalt contractors. This new method combines two CR modification process CRTB and CRDry and called as CR-Hybrid (denoted as CRHY).

Contractors can adjust the amount of CR economically and readily in the asphalt plant with the CRHY method. For example, by adding about 0.5% dry CR (by weight of the mix), the amount of the rubber in the asphalt mixture can be doubled as compared to the conventional CRTB mix.

1.3 OBJECTIVES

There are several objectives of this study. The main objective is to engineer asphalt binder and mixtures by integrating recycled tire rubber (RTR) to construct more sustainable and durable hot mix asphalt pavements. In order to successfully achieve the main objective, intensive investigation of RTR binder and mixture properties has been performed with the following objectives:

- To provide a novel CR modification method which can increase the amount of rubber in typical CRTB modified asphalt pavements without adversely affecting the pavement performance.
- To determine the feasibility of using CR and DVR modified binders with high percentage RAP mixtures in lieu of expensive soft binders and blending chart analysis.
- To examine the relative performances of the Styrene-Butadiene-Styrene (SBS) polymer and CR/DVR modification in an asphalt binder and mixtures.

1.4 RESEARCH PLAN

To successfully achieve the objectives of this research, a detailed study on the effect of RTR modification in various binders and mixtures is performed. Both binder and mixture performances are investigated.

The first phase of this research work includes an intensive literature review which covers the main subject in the problem statement with methods used to achieve the ultimate goal. This doctoral work was done through the review of relevant publications from international conferences and proceedings, journals, and technical meetings along with scientific documents.

The second phase establishes the fundamentals and performance tests of CRHY method.

Within this phase, the main focus is to determine the most suitable CR replacement method. Mixture performance testing follows the weight and volumetric-based aggregate CR replacement methods. Performance tests focus on rutting and moisture susceptibility, and fatigue and thermal cracking.

Phase 3 investigates the binder softening effect of the RTR modifiers in high RAP asphalt pavements. In this phase hot mix asphalt mixture performances are studied. Low temperature and fatigue cracking of the mixtures are performed by using indirect tensile (IDT) strength and uniaxial push pull (PP) tests, respectively. Moreover, the linear viscoelastic characterization of mixtures is studied. Mixture fatigue tests are examined by using viscoelastic continuum damage (VECD) models at different temperature and strain levels. AASHTOWare Pavement ME Design software is used to predict the field fatigue performances of the mixtures.

Phase 4 encompasses the effect of polymer and rubber modification of various asphalt binders. In this phase, different polymer and rubber types are studied. In addition to regular crumb rubbers (CR), de-vulcanized rubber (DVR) is investigated. Individual and combined performances of polymer and/or recycled tire rubber modifications are examined. Firstly, the improvements on Superpave Performance Grade (PG) of the modified binders are observed. For this purpose, high, low and intermediate continuous PGs of the modified binders are studied. Subsequently, binder performance tests including linear amplitude sweep (LAS) and multiple stress creep recovery (MSCR) are performed.

Lastly, phase 5 is the culmination of the previous phases. It is accomplished by reinstating the achievement acquired within the research as well as recommending new subjects and topics for future research that is important to better develop the recycled tire rubber (in the form of CR or DVR) modification in asphalt pavements.

The research presented in this doctoral dissertation has been disseminated through publications and presentations at international conferences.

1.5 DISSERTATION ORGANIZATION

This doctoral dissertation presents the methods, procedures, tests, results, analysis and discussions obtained from the detailed examination of recycled tire rubber modified hot mix asphalt pavements. The dissertation is divided into six chapters. First two chapters include the introduction and literature review. Chapters three through five contain research articles submitted to or published or getting prepared to be submitted international journals or conferences. Each of these chapters discusses the various aspects of tire rubber modification to improve certain properties of asphalt binders and/or hot mix asphalt mixtures and satisfy at least one of the objectives of this doctoral study. Chapter six is a summary chapter and includes the ultimate conclusions and recommendations for future research on tire rubber and polymer modified asphalt binders and pavements.

Introduction chapter provides the background information, terminology and common abbreviations used throughout the dissertation. Background information is followed by the problem statement which states the reasons why this research is needed. Subsequently, the objectives are stated to successfully fulfill the research needs. Research plan and dissertation organization are the last two parts of chapter, explaining how the research evolved and this doctoral dissertation was written.

Literature review chapter covers the broad range of information about recycled tire rubber, production technologies, ingredients, modification processes, history of crumb rubber and performance of crumb rubber modified asphalt pavements.

Chapter three consists of articles which introduces a novel method for crumb rubber modified asphalt methods. Chapter four is another article that focuses on the use of high RAP in HMA. It includes the results of investigation of the binder softening effect of recycled tire rubber. Chapter five is also another article and focuses on individual and combined effect of polymer and recycled tire rubber (RTR) (in the form of either crumb rubber/ground tire rubber or de-vulcanized rubber) modifications on the performance grade, fatigue cracking resistance and rutting potential of modified asphalt binders.

Overall, this dissertation included results of comprehensive experimental analyses whose goals were to investigate the question of how various crumb rubber modified asphalt technologies interact with (i) each other, (ii) the other sustainable construction techniques (e.g., high recycled asphalt pavements) and (iii) polymer modified asphalt binders.

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CHAPTER 2 LITERATURE REVIEW

2.1 CRUMB RUBBER BACKGROUND

Each year approximately 300 million scrap tires have been discarded solely in the United States. One third of these tires have been retreaded, resold or diverted into alternative uses. The remaining 200 million tires have been stockpiled, dumped illegally or buried into landfills. Stockpiling of scrap tires has brought up some environmental problems as well. Publicized stockpiles have been a favorable place for mosquitos to reproduce and also they have been fire hazards. Search for an alternative scrap tire market has been an on-going process for years. There have been benefited two potential use of scrap tire on this process. The first one is the fuel for combustion and the second one is crumb rubber modifier (CRM) for the asphalt paving industry (S.T.M.C. 1990). CRM is derived from vehicular tires by mechanical cutting and grinding of the recycled tire rubber into small particles (Palit 2004). Crumb rubber (CR) modified hot mix asphalt pavements are formed by introducing ground tire rubber into the asphalt mixture using different methods. Although the asphalt modification with CR seems to emerge from environmental concerns, in reality it came out as a combined effort of chemists and engineers to enrich the properties of the asphalt binder in early 1840s (RW 1967). They have performed research on blends of natural (latex) and synthetic rubbers (polymers) which are the ingredients of raw tire rubber. Schnormeier (1992) stated that an average passenger car tire contains synthetic rubber (10 types), natural rubber (4 types), carbon black (4 types), chemicals (40 types), steel cord, bead wire, oils, pigment and waxes. Table 1 provides the rubber content of an average tire (FHWA1998).

Table 1 Rubber content of average tires manufactured (FHWA 1998)

Tire Type	Tire Construction Type	Natural Rubber Amount	Synthetic Rubber Amount
Passenger	Radial	35%	65%
	Bias	15%	85%
Truck	Radial	65%	35%
	Bias	30%	70%

2.2 HISTORY OF CRUMB RUBBER IN THE UNITED STATES

The first trace of using CR as an asphalt paving material could go back to the1940s. U.S. Rubber Reclaiming Company began processing a devulcanized recycled rubber product, called Ramflex, as a dry particle additive to asphalt paving mixtures. In the mid-1960s, Charles McDonald developed a crumb rubber modified binder, called Overflex, to improve pavements in Phoenix, Arizona (Cao 2007). McDonald was a material engineer for the City of Phoenix and worked in conjunction with a local asphalt company, Sahuaro Petroleum, to produce highly elastic CR modified surface patch. The Arizona Department of Transportation (ADOT) started using CRM as a thin maintenance patch and stress absorbing membrane (SAM) during the late 1960s. ADOT paved the first stress absorbing membrane interlayer (SAMI) in 1972 and CR modified open graded friction course (OGFC) hot mix asphalt (HMA) in 1975. In the mid-1970s ADOT had over 700 miles of SAM and SAMI paved (ADOT 1989).

Arizona Refinery Company (ARCO) further improved the wet process (a.k.a McDonald process) technology by using CRM and devulcanized CRM in the same decade. However, the benefit of devulcanized rubber was insignificant compared to regular CRM. Moreover, the additional processing cost of rubber devulcanization resulted in the project to come a halt (Heitzman 1992).

Sweden worked on CRM during the same decade as ADOT as well. However, their main focus was the dry process. In Europe, the dry process was developed under trade name Rubit.

U.S. investors patented this technology late 1970s under the PlusRide trade name and refined the mix design to come up with a gap graded mix (Heitzman 1992).

2.3 EFFECT OF CRUMB RUBBER ON ASPHALT BINDERS

CR-binder interaction is defined as the diffusion of the lighter binder fractions into the CR particles. The amount of aromatic fraction, temperature and viscosity of binder and the grinding method (ambient and cryogenic), particle size, percentage, specific surface area and chemical composition (i.e., amount of natural rubber) of crumb rubber majorly affect the properties of crumb rubber modified asphalt binders.

The specific surface area was indicated to be the most important physical property of CRM binders (LaGrone 1980, Heitzman 1992, West et al. 1998, Lee et al. 2007). Since the particle size impacted the surface area, finer particles (~40 mesh (0.425 mm)) have been generally selected to modify the binders. However, some states such as Arizona and California have been using coarser CR particles (~ 14 meshes) and their pavements have been performing successfully, indicating that CR surface area may not be the most important factor. In addition, the CR content affects the pavement performance. Some states such as Arizona and California utilize at least 15 % by weight of the binder. On the other hand, states like Florida limit CR usage to maximum 10% range (Hicks et al. 1995).

Bahia et al. (1994) studied the effect of CRM on performance related properties of asphalt binders. One of their ultimate goals was to conclude if Strategic Highway Research Program (SHRP) finding for original/unmodified binders could be applied to CR modified asphalt binders. In their study, the test matrix had three CR grinding methods (ambient, cryogen and extrusion process with the use of some additives) and four binders. The pumping, storage and construction temperatures were determined by rotational viscometer. Three rubbers were selected in the same gradation with a maximum particle size of approximately 1mm. The selected binders covered a wide range of compositional properties. Additionally, the chemical and physical properties of binders were measured including the asphaltene and aromatic content, molecular weight, rheological and failure properties. Maximum and intermediate temperature performances of the binders were observed with dynamic shear rheometer. Low temperature properties of the binders were evaluated with bending beam rheometer and the direct tension device. Thin film oven test and pressure aging vessel were used to age the binders to measure the aging characteristics and high-temperature stability. This study indicated that the SHRP testing procedures could be utilized to the CR modified binders with minor changes.

Tayebali et al. (1997) studied the use of dynamic shear rheometer (DSR) to performance grade the CR modified binders. The authors used different gap settings in parallel plate geometry. They achieved one PG grade jump for 7% CR and two grade-jump for 14% CR inclusion. The test matrix also included #40 and #80 mesh sizes to study the effect of CR gradation. Their conclusion was that while the high PG of the different size CR modified binders showed a similar behavior, intermediate and low PGs of them were not quite as similar.

Gopal et al. (2002) studied the low temperature rheological properties of CR modified binders. The test matrix included three particle sizes, two CR contents and four binder types. Results showed that creep stiffness (S) decreased with increasing CR content. However, the logarithmic creep rate (m-value) was inconsistent. The size of CR particles was insignificant on low temperature properties.

To evaluate CR modified binders, some researchers focused on the penetration, elastic recovery, softening point and ductility. While the penetration is a measure of hardness of a binder, Elastic Recovery (ER) is the tendency of recovery of the binder after applied stress or strain is released. ER is used as a fatigue and rutting resistance parameter. Mashaan et al. (2011) stated that the penetration of binder decreases as the CR amount increases. Moreover, aging further decreased the penetration value. Huang et al (2007) studied the effect of CR particle size on elastic recovery and he concluded that as the CR particle size decreased, ER of the binder increased. Ductility was another parameter that was used by researchers to differentiate the CR modified binders. It is the distinct strength of asphalt binder, tolerating it to endure notable deformation or elongation. Finer size CR particles resulted in higher elongation ductility. Toughness showed an increasing behavior as the amount of CR was increased. Hence, CR modified binders performed higher ductility and rutting resistance (Mashaan et al. 2011). Another binder property Mashaan et al. (2011) studied was the softening point resilience of the binders. Compared to unmodified binders, they showed that CR modified binders had higher softening points and lower resilience. Becker et al (2001) researched the effect of viscosity, resilience, softening point and penetration of rubberized binders. Their findings matched with that of previous researchers. Wypych (2000) revealed that viscosity measurements were not enough to determine the properties of CR modified binders because of their non-Newtonian behavior.

During the same decade, while some researchers focused on the effect of CR particle gradation other than amount by itself, some others investigated the dimensional changes on CR particles (e.g. size and morphology). Neto et al. (2006) studied viscosity, resilience and softening points properties of the binders modified with various CR gradation, digestion time and temperature. They concluded that as the specific surface increased, all measured properties tended to increase as well.

Lee at el. (2007) studied the effect of the CR processing methods. Ambient temperature grinding process provided better rutting and thermal cracking behaviors.

Putman et al (2006) developed two equations related with CR modified binders. While Interaction Effect (IE) focused on absorption of aromatic oils from binder by CR particles, Particle Effect (PE) measured the effect of CR as filler in the binder. Their study revealed that binder source and CR content has a significant impact on IE. Moreover, PE was most significantly affected by CR content than CR particle size.

Cong et al. (2013) investigated the morphology and rheological properties of CR modified binders. A fluorescent microscope was used to analyze the morphology of the binders. They concluded that ambient and cryogenic rubber particles have different swelling properties.

2.4 PERFORMANCE OF CRUMB RUBBER MODIFIED ASPHALT PAVEMENTS

Most early researches focused on the properties of CR modified asphalt binders with the CRWet process. After familiarizing with CR binders, researchers investigated the mixture properties. Asphalt pavements were tested and analyzed to address the distresses that may develop during the life cycle of the pavement. Performance tests conducted on asphalt mixtures include mainly resistance to permanent deformation, thermal and fatigue cracking and moisture susceptibility. Most of the researches and studies on crumb rubber modified asphalt pavements (CRMAP) focused on these performance tests. There are different methodologies and analyses used for each performance test in the laboratory conditions.

Heizman (1992) synthesized both CR and CRMAP studies in his publication. His work showed that the performance of asphalt mixtures was enhanced by CR modification. This was mainly attributed to the reduction in viscosity of CRM binders. Since the viscosity decreases, aggregates are coated with a thicker binder film which provides more resistance to oxidative aging. It is a well-known fact that asphalt binders that are less affected from oxidative aging are more durable to thermal and fatigue cracking. Moreover, Palit et al. (2004) showed that oxidative aging was more pronounced for base (control) binders than CR modified binders. Many researchers concluded that the use of CR enhances the fatigue life of pavements (Raad et al. 1998).

The permanent deformation at high temperatures (60^{o}C) and thermal cracking at low temperatures (-10^{o}C) of CR modified asphalt mixtures using the dry process were investigated by Cao (2006). These CRDry mixes had the potential to use more CR than CRWet and CRTB mixes. Mixtures with dense gap graded gradation and CR contents changing from 1% to 3% were prepared. The results revealed that both permanent deformation resistance and low temperature cracking properties were improved with an increasing amount of CR.

The main problem with the CRDry method is the lack of interaction between CR particles and the binder itself. This deficiency may result in the lowered moisture resistance (Moreno et al. 2011).

2.5 METHODS OF PROCESSING SCRAP TIRE INTO CRUMB RUBBER

The method of processing of scrap tire into crumb rubber is determined according to the project specifications. Surface area, shape, size and type of crumb rubber particles (granulated or ground) are the key factors for determination of the process. Other than the micro-mill process, each process uses a series of fiber and steel separators which remove the fiber reinforcement and steel belting from the processed crumb rubber. Talc or some other inert mineral powder is introduced to the CR to minimize the tendency of rubber particles to stick together and form a lump (Heitzman 1992).

2.5.1 Cracker-mill Process

It is the most common and productive crumb rubber production process. It is performed at ambient temperature. In order to use the scrap tire with this method, tires need to be shredded. In this process, scrap tires pass between rotating corrugated steel drums for size reduction. The spacing and the differential speed of the drum-pairs control the tearing of the scrap tire. Ground tire rubber produced with the crackermill process has irregular shapes with larger surface areas. Particle sizes over a range of No 4 to No 40 sieve openings (4.75 mm to 425 micron) can be achieved with this method (Heitzman 1992, West et al. 1998, Willis et al. 2012).

2.5.2 Granulator Process

The granulator process is also performed at ambient temperature. The form of the scrap tire does not have an effect in this process. It can even be performed on whole, unprocessed tires. In this method, the scrap tire is cut with revolving steel plates that pass at close tolerance. The resultant granulated crumb rubber particles have uniform cubical shapes with a low surface area. This method can produce a range of sizes from 3/8-inch to no 10 sieve openings (9.5 mm to 2.0 mm) (Heitzman 1992, West et al. 1998, Willis et al. 2012).

2.5.3 Micro-mill Process

The micro-mill process is used for a further size reduction of crumb rubber particles. It uses the CR obtained from cracker-mill or granulator process. First, the crumb rubber is mixed with water to form a rubber-slurry. Subsequently, this slurry is forced between rotating abrasive discs to reduce particle size. The final product is retrieved and dried. Sizes of the crumb rubber particles acquired with micro-mill process range from No 40 to No 200 sieve openings (425 micron to 75 micron) (Heitzman 1992, West et al. 1998, Willis et al. 2012).

2.5.4 Cryogenic Grinding

The cryogenic process is performed at lower temperatures than the other grinding processes. The temperature of the scrap tire is lowered to obtain frozen scrap tire by submerging into liquid nitrogen and making it brittle. The desired particle size of the brittle scrap tire is achieved by crushing. This method of grinding is too costly to be practical (Heitzman 1992). Moreover, the particles grinded with the cryogenic process have more angular and relatively smooth surfaces as a result of fracturing rubber particles at low temperatures (-87 °C to-198 °C) (Blumenthal 1994). Cryogenically processed rubber may not be suitable for asphalt pavement modification because of the manufactured surface texture (West et al. 1998, Willis et al. 2012).

2.6 CRUMB RUBBER MODIFICATION TECHNOLOGIES

In broad terms, crumb rubber modification technology can be divided into three categories; the wet, the terminal blend and the dry processes.

2.6.1 Wet Process

The wet process is the CR modification method that incorporates the CR particles to the asphalt cement prior to constructing asphalt mixtures. The outcome is crumb rubber modified asphalt binder (CRWet). In the case of devulcanized rubber (DVR), the product is devulcanized rubber binder (herein CRDev). In the CRWet process, the CR is added to hot liquid asphalt in special mixers at the asphalt plant. The blending mixer applies low shear mechanical energy to produce the CRWet. Following the blending, the rubber-binder mixture is immediately transferred into the reaction tank where uniform blending and constant temperature are maintained. While the uniformity of the mixture is provided by a mechanical agitating system, the constant temperature is satisfied with the circulation of the reaction tank. The wet process is performed at elevated temperatures. The heating system has to be capable of keeping the temperature between 175^{o}C and 200^{o}C. The reaction time is typically 45 to 60 minutes. All system can be hauled by a tractor and easily transferred to any job site.

A typical amount of CR used during CRWet process is approximately 15% to 25% by weight of binder. The wet process can be applied to crack sealant, surface treatments and hot mix asphalt mixtures. In laboratory conditions, there are different combinations of temperature, mixing duration and shear rate (Heitzman 1992).

2.6.2 Terminal Blend Process

Crumb rubber terminal blend (CRTB) is a special form of CRWet process. While CRTB is produced in asphalt binder terminals, CRWet is manufactured in the asphalt plant and added to the aggregate blend to produce the CRMAP. CRTB binders can typically accommodate 10%-12% CR. Since CRTB binders are produced at binder terminals and transported to asphalt plants, the storage stability is maintained by addition of a polymeric additive during the mixing process.

2.6.3 Dry Process

In the dry process, the CR particles partially replace the fine aggregate portion of the mixture and they are introduced to the mix at the asphalt plant. It is a common practice to pre-blend the CR particles with heated aggregate prior to incorporating to the liquid binder. There is no special equipment required. The only concern is to adjust the temperature and mixing time to blend the mixture uniformly. In the asphalt plants with a RAP feeder, CR is introduced into the system using this feeder (Heitzman 1992).

The dry process (CRDry) is limited to hot mix asphalt applications and cannot be used other paving methods, such as surface treatments.

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CHAPTER 3 INTERACTION BETWEEN TERMINAL BLEND AND DRY CR TECHNOLOGIES- A NOVEL CRUMB RUBBER ASPHALT MODIFICATION TECHNOLOGY: A HYBRID TERMINAL BLEND

3.1 INTRODUCTION

Scrap tire handling has been a common problem all over the world. There are more than 300 million scrap tires buried in the landfills of the states each year. Federal and state governments have been searching for alternative methods of either disposing or recycling the large number of waste tires. The use of shredded tire rubber (i.e., crumb rubber – CR) in asphalt pavements is one of the sustainable recycling approaches. Crumb Rubber (CR) has been used as an additive in asphalt pavements since the 1950s (Heitzman 1992, Huang et al. 2002, Sonti et al. 2003). The benefits of CR Modified Asphalt Pavements (CRMAPs) have been acknowledged by numerous researchers (Way 1999, Carlson et al. 1999, Hicks et al. 1999). Most CRMAPs are made with the following three major methods: wet process (CRWet), dry process (CRDry), and terminal blend (CRTB) process. In the CRWet process, CR particles are added to the liquid asphalt at temperatures around 325-400 °F (163-205 ºC) and approximately 15% - 22% of CR by weight of the binder is utilized (1-1.5% by total weight of the mix) (Caltrans 2005; Caltrans 2005). The terminal blend (CRTB) process is similar to the wet process, except that a lower amount of CR is used (~10-12%) and a polymeric additive is used to keep the CR particles suspended in the binder. The characteristics of CRWet and CRTB modified asphalt binders depend on the type of rubber, binder type, size of the CR particles, duration and temperature of the reaction and the modification method (Carlson et al. 1999; King et al. 1999).

The CR particles react with asphalt binder in two different ways during the modification process: swelling and/or degradation (Abdelrahman et al. 1999). Depending on the reaction time and temperature, CR particles swell approximately two to three times of their original size (Jamrah et al. 2015). The CRDry process is the method where the CR particles are added to the mix as a partial replacement of fine aggregates. In addition, there are several treated dry rubber technologies where the crumb rubber particles are pre-mixed with low viscosity petroleum-based products or aromatic oils compatible with the lighter fractions of asphalt binder. These treated rubber technologies are used as a CRDry process where they are added as partial fine aggregate replacement to the mixture.

The objective of this study is to investigate if the amount of rubber in typical CRTB modified asphalt pavements can be increased without negatively affecting the pavement performance. One of the advantages of CRHY over the Wet Process is that it can be significantly less expensive in states where plant mixing and reacting equipment for CR is not readily available (e.g., Michigan). By adding about 0.5% dry CR (by weight of the mix), the amount of the rubber in the mixture can be doubled as compared to the conventional CRTB mix. Increasing the amount of CR is absolutely important since some of states require a minimum threshold for integrated CR in CRMAPs. The States of California and Arizona require at least 15% CR by weight of binder (Hicks et al. 1995). The relative performances of CRTB and CRHY were investigated in terms of their linear viscoelastic properties, rutting susceptibility, moisture damage, resistance to fatigue and low temperature cracking. The effect of volumetric and weight-based CR replacement method on rutting performance was also investigated. Rutting susceptibility was evaluated by using flow number (FN) test whereas tensile strength ratio (TSR) was performed to investigate the potential for moisture damage. Fatigue performance was evaluated using viscoelastic continuum damage (VECD) modeling approach with the data obtained from uniaxial push-pull (tension-compression) test. Lastly, indirect tensile strength (IDT) test was conducted to assess the thermal cracking behavior of the mixtures.

3.2 MATERIALS AND METHODS

The materials (aggregates, binder, and RAP) used in this study were obtained from local asphalt plants near Lansing, MI. The aggregates and reclaimed asphalt pavement (RAP) were sampled from the most commonly used stockpiles to provide a better representation of the materials used in the State of Michigan. RAP used in this study had 4.53% binder content. CRTB binder was supplied by a company located in the State of Illinois. CRTB was modified with 12% CR by weight of binder and had performance grade (PG) of 70-28. The base binder PG prior to modification was PG58-28. Three different types of crumb rubber were used in the preparation of performance samples: (i) untreated crumb rubber (UCR), (ii) type1 treated rubber (T1TR) and (iii) type2 treated rubber (T2TR). UCR had #20 mesh-size with the gradation shown in Table 2. The first type of treated rubber (T1TR) was manufactured by a company located in Florida, by soaking the CR particles with a proprietary fluid. The second type of treated rubber (T2TR) was manufactured by a company headquartered in the State of New Jersey by using a different type of chemical liquid and coating.

Table 2 Crumb rubber gradation

Sieve Size		UCR (#20 mesh)
Metric (mm)	Standard	
2.36	#8	100
1.18	#16	99.6
0.600	#30	53.6
0.300	#50	11.1
0.150	#100	2.3
0.075	#200	0

3.2.1 Sample Preparation

In this project the control samples were produced by using the mix design prepared with CRTB binder (without any dry CR). Preparation of the CRHY samples was performed by using volumetric replacement method rather than conventional weight based replacement method. In this method, fine aggregate particles inside the control mixture were replaced with the same volume and size of CR particles. This method was found to provide better control of the air voids and was adopted for the rest of the study. The other mixtures were prepared by using treated rubbers T1TR and T2TR instead of conventional dry rubber. CRHY-Field and LVSP-Field samples were obtained from field collected loose mixture during the construction behind the paver. After performing the laboratory investigations on the loose mixtures, performance samples were prepared. Table 3 summarizes the designation and explanation of the mixtures.

Table 3 Mixture designation and explanation

Designation	Sample Preparation Technique
CRTB	Control (base) mixture by using CRTB binder
CRHY-WB	Weight- based replacement of fine aggregates (WRFA) with CR
CRHY	Volumetric replacement of fine aggregates (VRFA) with CR
CRHY_T1TR	VRFA with FL based company’s treated CR
CRHY_T2TR	VRFA with NJ based company’s treated CR
CRHY-Field	Field loose mixture, plant produced according to CRHY
LVSP-Field	LVSP Mix used by ICRC

Low Volume Superpave (LVSP) mixture is a commonly used mixture type by Ingham County Road Commission (ICRC) in the State of Michigan. It is designed according to Superpave mix design requirements for low volume local roads (<300 commercial ADT) (APA-MI, 2012).

The volumetric samples prepared during mix design were conditioned at compaction temperature for two hours, whereas performance test samples were conditioned at 135^{o}C for four hours. Cylindrical samples were compacted by using Superpave Gyratory Compactor (SGC) either to required gyrations numbers (for mix-design samples) or to a specific height to get 7% ± 0.5% air voids (V_a) (for performance test samples). To minimize the end-effects, performance samples were compacted to 150 mm diameter and 180 mm height and cored, then cut into 100 mm or 76 mm diameters and 150 mm height depending upon the type of testing. Subsequently, each individual specimen physical properties (i.e., dimensions, air voids, end flatness) were measured and recorded.

3.2.2 Mix Design

The Michigan Department of Transportation (MDOT) Superpave mix design guidelines were adopted during the mixture design process. The type of MDOT mix designed was 4E03, meaning that this is 4^{th} layer (which can be used either as a top or leveling course), with a design traffic level of 300,000 ESALs (Equivalent Single Axle Loads) (Defoe 1985). The requirements for MDOT 4E03 Superpave mix design are given in Table 4.

Table 4 MDOT 4E03 Superpave mix design requirements (Defoe 1985)

Maximum % G_{mm} at Ninitial		91.5%
Number of Gyrations	$N_{initial}$	7
	N_{design}	50
	N_{max}	75
VFA (%) at N_{design}		70-80
Percent of Maximum Specific Gravity (%G_{mm}) at the Design Number of Gyrations (N_{design})		96%
% G_{mm} at the Maximum Number of Gyrations, (N_{max})		98%
Minimum VMA % at N_{design} (based on aggregate bulk specific gravity, (G_{sb})		14
Fines to effective asphalt binder ratio (P#200/P_{be})		0.6-1.2

Table 5 provides design aggregate gradations of mix designs, Superpave specification limits, binder and RAP amounts and combined bulk specific gravity of the aggregate batches.

Table 5 Laboratory and field design gradation and specification requirements

Sieve Size	CRHY & CRTB Gradation	LVSP-Field Gradation	CRHY-Field Gradation	12.5 mm Superpave Specification
19 mm (3/4")	100	100	100	100 min
12.5 mm (1/2")	91.5	94.9	90	90-100
9.5 mm (3/8")	83.6	87.7	78.9	90 max
4.75 mm (#4)	39.1	76.4	38.2	-
2.36 mm (#8)	29.4	58.6	21.6	28-58
1.18 mm (#16)	22	45.7	17.2	-
0.600 mm (#30)	14.9	34.3	13.8	-
0.300 mm (#50)	8.4	19.5	9.2	-
0.150 mm (#100)	5.5	8.5	5.6	-
0.075 mm (#200)	4.3	5.6	4.3	2-10
Binder Content (%)	5.30%	4.10%	4.76%	-
RAP Content (%)	10.00%	29.00%	20.00%	-
Bulk Specific Gravity, (Gsb)	2.624	2.654	2.643	-

There were two different CR-fine aggregate replacement methods. The goal was to find a replacement method that would be convenient for asphalt contractors and would perform better in the field. The first method used was a weight-based (WB) replacement method. The second one was a volumetric replacement method (VRM). Both methods are discussed in detail.

3.2.2.1 Weight-Based replacement method

The first replacement method investigated in the research was weight-based (WB). In this method, the same gradation and amount of (weight based) fine aggregates were replaced with CR particles. The amount of replacement was 0.5% by weight of total mixture. The compaction effort was maintained constant for both the control and the WB replacement mixtures. It was observed that the air void reduction was so high on CRHY- WB samples. Moreover, it was almost impossible to stay within the limits of Superpave volumetric requirements. These shortcomings yielded the modification of the mix design which would result in a tremendous amount of work for asphalt contractors. However, to analyze the performance of CRHY-WB samples, the first set of performance samples were prepared and tested for investigation of permanent deformation.

3.2.2.2 Volumetric replacement method (VRM)

The second replacement method was performed by replacing the same volume of fine aggregates with CR particles. Low percent air voids obtained with the previous replacement method established the fundamentals for VRM. Replacement of the fine aggregate and CR particles was done according to equal volume. The calculation steps can be summarized as follows:

- Calculate the weight of replacement according to 0.5% by weight of mixture.
- Convert the calculated weight to volume by using the specific gravities of fine aggregates.
- Convert the CR volume to weight by using the specific gravities of CR particles.

The equations 3.1 through 3.3 show the computation of percentage of CR by weight of the asphalt binder for the CRHY mixture. As shown, CRHY method almost doubled the amount of CR used in the HMA, as compared to the CRTB mixture (which contained 12% CR by weight of binder). Given that the optimum binder content of the mixture was 5.30%, the following illustrates the CR content relative to the binder in the mixture:

$$\%\text{CR by weight of CRTB mixture}=5.3\%*12\%=0.64\% \tag{3.1}$$

$$\%\text{CR by weight of CRHY Mixture}=0.64\%+0.5\%=1.14\% \tag{3.2}$$

$$\%CR \text{ by weight of binder}= \frac{1.14\%}{5.3\%} = 21.4\% \quad (3.3)$$

It is also noted that this amount of CR (21.4%) is more than most wet process crumb rubber (CRWet) modified asphalt mixtures.

3.3 PERFORMANCE TESTS ON LABORATORY AND FIELD SAMPLES

Performance tests of hybrid and control mixtures were initialized with flow number tests. Since the use of CR makes the mixture softer, permanent deformation tests have been chosen as a starting point. According to the results of the FN tests, the testing matrix for further performance tests was updated. The FN test results showed that treated rubbers were not as good as conventional CR. Moreover, the FN test results of the samples prepared according to weight-based replacement method showed that use of volumetric replacement method was more suitable. As a result of the FN tests findings, it was concluded that further performance tests would only compare CRTB and CRHY produced with conventional CR particles using volumetric replacement method.

Field test sections using CRHY method were constructed and samples were collected for further evaluation. Control section was constructed by using mainly used Low Volume Superpave (LVSP) mix design. FN test results of LVSP-Field and CRHY-Field mixture were provided to give a comparison as well.

3.3.1 Flow Number Testing to Evaluate Permanent Deformation (Rutting) Susceptibility of Mixtures

Rutting is one of the major pavement distress types in asphalt pavements, especially in warm climates. The Flow Number (FN) test is a reliable test method to assess the rutting susceptibility of HMA pavements. FN is a repeated load test typically performed at relatively high temperatures. During the test, cylindrical HMA samples having 100 mm diameter and 150 mm height are subjected to uniaxial repeated haversine pulse load with 0.1 second loading followed by 0.9 second rest period. Depending upon the field conditions, tests can be conducted either in a confined or unconfined configuration. The AMPT data acquisition system records the plastic strain at the end of each loading cycle. The Flow Number is defined as the cycle at which the tertiary flow starts (Witczak 2005). In this research, FN tests were run at 45^{o}C with 483 kPa (70 psi) deviatoric and 69 kPa (10 psi) confining stresses both all samples.

Figure 1 illustrates the results of FN tests where the permanent microstrains of CRHY mixtures seem to be more than those of the CRTB mixture in laboratory prepared mixtures. This illustrates that the CRHY mixtures may be more prone to rutting than the CRTB mixture. However, the curves of CRHY mixtures are not too far from the CRTB (i.e., the control mix). Moreover, field CRHY mixture is as good as the laboratory CRTB mix. The other mixtures made with treated rubber (i.e., T1TR and T2TR) seem to be significantly more rut susceptible than the CRTB mixture. This might be because of the oily components within the treated rubber being squeezed out during compression and causing the overall mixture to deform more than the control mixture. On the other hand, it is worth noting that in treated CRHY mixtures, the initial deformation (within the first 2000 cycles) is quite significant and the rate of change of deformation (i.e., the slope) after 2000 cycles is very similar to the rate of change of deformation of the CRTB mixture. CRHY-Field and LVSP-Field samples were obtained from constructed test sections which were paved next to each other. Figure 1 also demonstrates that CRHY-Field samples perform better than LVSP-Field samples for rutting susceptibility.

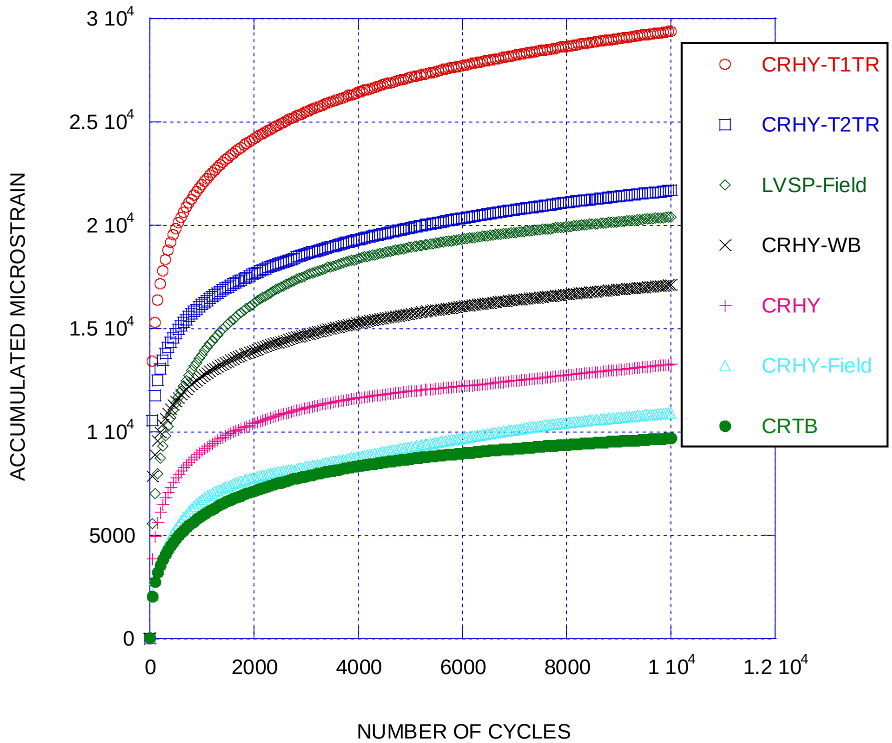


Figure 1 Flow number test results

3.3.2 Dynamic Modulus Testing for Viscoelastic Characterization of Mixtures

|E*|, dynamic modulus, test is hypothetically a non-destructive laboratory test used to determine the linear viscoelastic properties of asphalt mixtures. |E*| and phase shift angle are measured under uniaxial haversine compressive stress at different temperatures and frequencies. The test is performed by using asphalt mixture performance tester in the laboratory conditions. At least three samples were tested and obtained values were averaged. Testing was performed at -10°C, 10°C, 21°C, 37°C, and 54°C. Loading frequencies at each temperature level were chosen as 25Hz, 10Hz, 5Hz, 1Hz, 0.5Hz and 0.1Hz. Once |E*| values were collected at various temperatures (T) and loading frequencies (*f*), the |E*| master curve was constructed using the time-temperature superposition (TTS) principle. Based on the TTS principle, a single |E*| master curve can be developed by shifting the |E*| data obtained at different temperatures horizontally in

a log-log plot of |E*| versus frequency. Once shifted, the parameter in x-axis is called reduced frequency (f_R), which is defined as follows (Kocak et al. 2016):

$$f_R = f \; a_T(T) \tag{3.4}$$

where f is the frequency of the load and (a_T(T)) is the shift factor coefficient for a given temperature T. The shift factor coefficient (a_T(T)), i.e., the amount of horizontal shift for each temperature is different. After the shifting is completed and the shift factor coefficients (a_T(T)) are determined, they are plotted against each temperature (T). Then typically a second order polynomial is fit to the data to obtain the polynomial coefficients a_1 and a_2 in the equation 3.5 (Kocak et al. 2016):

$$a_T(T) = 10^{a_1(T^2 - T_{ref}^2) + a_2(T - T_{ref})} \tag{3.5}$$

where T_{ref} is the reference temperature. During shifting process, the shift factors (a_T(T)) at each temperature are varied until a good sigmoid fit to the |E*| data of all temperatures is obtained. Typically the sigmoid function given in equation 3.6 is used (Kocak et al. 2016):

$$\log(|E^*|) = b_1 + \frac{b_2}{1 + \exp(-b_3 - b_4 \log(f_R))} \tag{3.6}$$

where b_1, b_2, b_3 and b_4 are the sigmoid coefficients, and f_R is the reduced frequency.

Figure 2 illustrates the dynamic modulus master curves of CRTB, CRHY and CRHY-Field mixtures. Dynamic modulus master curve gives a general idea about the mixture performance at high and low temperatures. Stiffer mixtures are desirable at high temperatures-low frequencies to minimize permanent deformation. CRTB and CRHY-Field mixtures demonstrate very similar behavior at low reduced frequencies, which both seems to be better than CRHY mixture for rutting susceptibility. On the contrary, softer mixes will be useful at low temperatures-high frequencies to prevent or minimize the thermal and fatigue cracking. Field and laboratory hybrid mixtures demonstrate very similar behavior at low frequencies, which is marginally softer than CRHY mix.

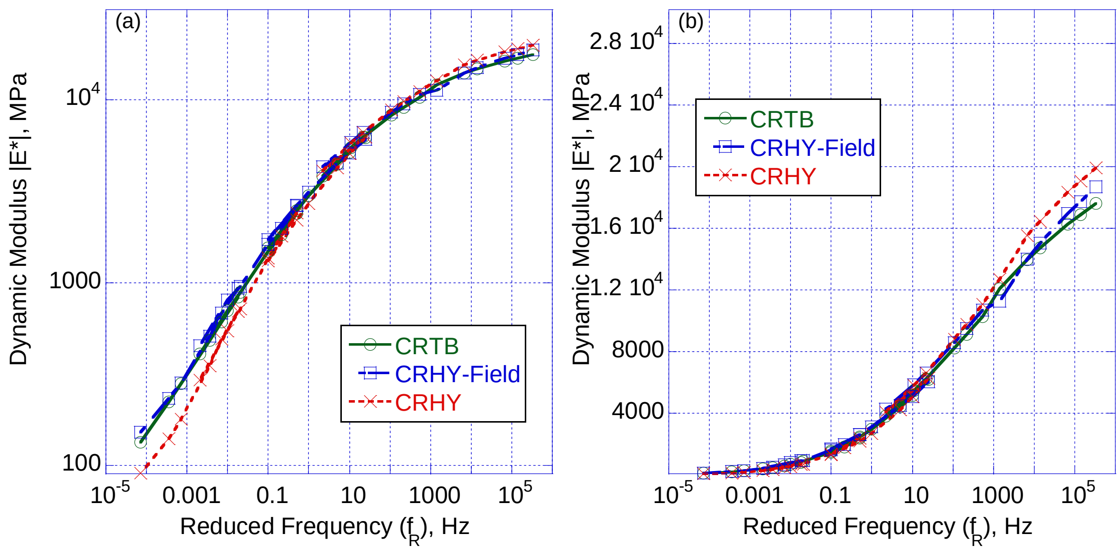


Figure 2 Dynamic modulus master curves of CRTB, CRHY and CRHY-Field (a) Log-Log and (b) Log-Linear scales

The master curves of all mixtures are within very close to each other at high frequencies which show inconsequential differences in the thermal and fatigue cracking behaviors. Since the differences between linear viscoelastic properties of the mixtures are very small, it is necessary to further perform rutting, fatigue and thermal cracking tests on the mixtures.

3.3.3 Push Pull (PP) Testing for Fatigue Cracking Behavior of Mixtures

Laboratory uniaxial cyclic push-pull (tension-compression) test is one of the common methods to assess field fatigue susceptibility of asphalt pavements. PP fatigue test was conducted in lieu of Four Point Bending Beam (FPBB) fatigue tests. The advantages of using PP over FPBB can be summarized as follows:

- Use of less material to prepare the materials
- Use of Superpave gyratory compactor, no need for an expensive slab compactor
- Applicability of viscoelastic continuum damage model (VECD) analysis

Dimensions of PP samples were 76 mm in diameter and 150 mm in height. Initially, Linear Variable Displacement Transducer (LVDT) tabs are mounted with two components high strength epoxy (120° apart from each other). LVDT opening was about 70 mm and the top and bottom tabs are about 37.5 mm away from the top and bottom edges of the samples. Subsequently, the specimens were glued with steel epoxy to aluminum top and bottom plates using a special gluing jig to provide perfectly parallel specimen ends. Finally, the tests were conducted using Asphalt Mixture Performance Tester (AMPT). Extreme care was given in placing samples into the loading frame to eliminate eccentricity which may cause non-uniform stress distribution and localized failure (in general close to one of the end platens).

The push-pull test can be conducted in both stress controlled and strain controlled loading mode. In this study, PP tests were performed at strain-controlled mode at 10 Hz frequency. Testing temperatures were chosen as $10^{o}C$ and $20^{o}C$. Two replicates were tested at each temperature level. The test was setup to apply 200 microstrain at each cycle. The termination criteria were chosen either the failure of sample or 300,000 cycles. Failure criterion was adopted from other fatigue tests and accepted as 50% reduction in the initial modulus of the sample.

The PP fatigue tests were only performed on the CRTB and the CRHY mixture made with dry rubber. The CRHY mixtures made with treated rubber were not included in the testing program since they were eliminated during FN testing due to their high rutting susceptibility.

3.3.3.1 PP fatigue data analysis by using viscoelastic continuum damage model

Results of the PP fatigue tests were analyzed by using VECD model. First the model was calibrated according to mixtures following the procedure described in Kutay et al. (2009) and Kocak et al. (2016). Fatigue lives (i.e., N_f) were computed at different strain levels and temperatures.

Figure 3 shows the number of cycles to failure for both of the mixtures. As shown, at relatively low strain levels the CRTB and CRHY are similar. At higher strain levels, the CRHY seems to be performing slightly better than the CRTB.

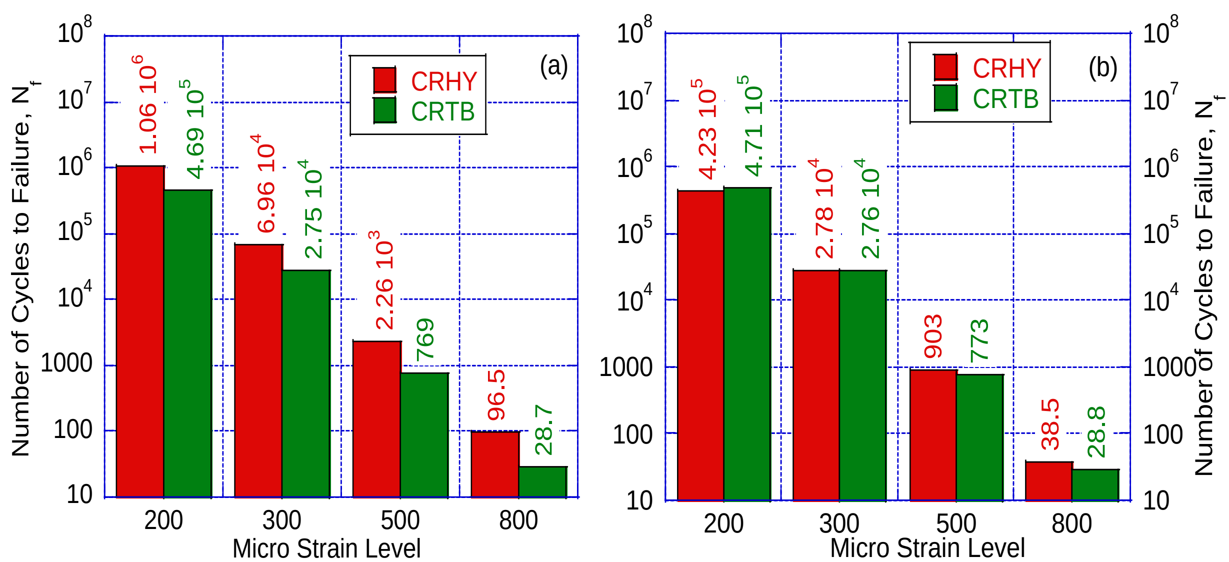


Figure 3 VECD analysis results at (a) 10°C and (b) 25°C

3.3.4 Indirect Tensile Strength (IDT) Test for Thermal Cracking of Mixtures

Low temperature cracking behavior of laboratory and field compacted samples were analyzed by using indirect tensile strength (IDT) test at -10°C. The loading rate was maintained at 12.5mm (0.5inch) per minute according to AASHTO T322. At least three replicates per mixture were tested under axial loading in materials testing system (MTS).

Figure 4 demonstrates the load versus displacement curves of three replicates for each mixture type. The difference ductile behavior of CRHY samples and comparatively brittle behavior of CRTB samples can obviously be seen.

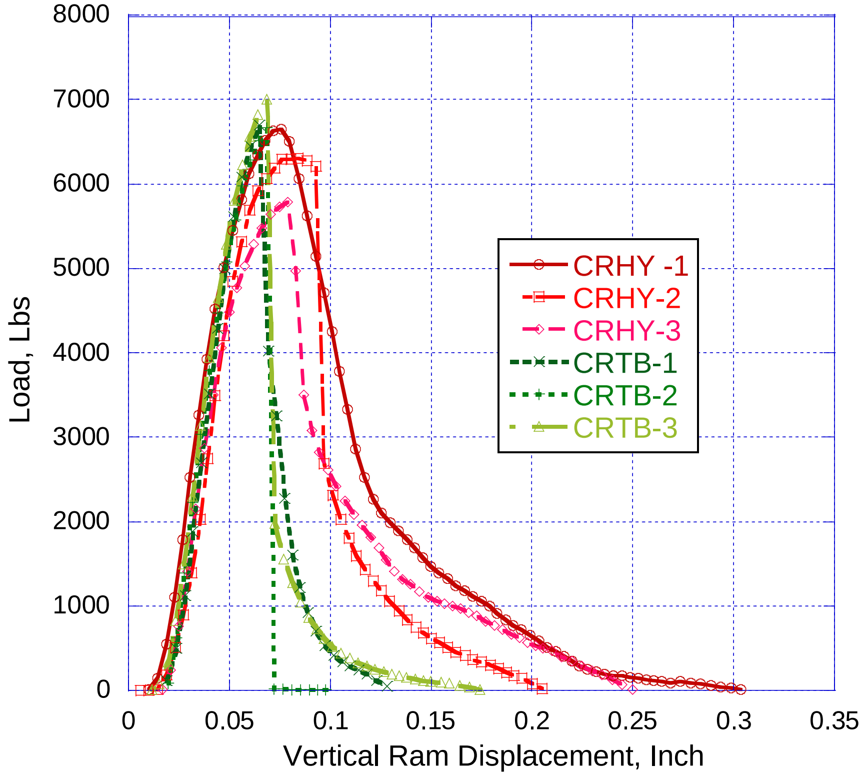
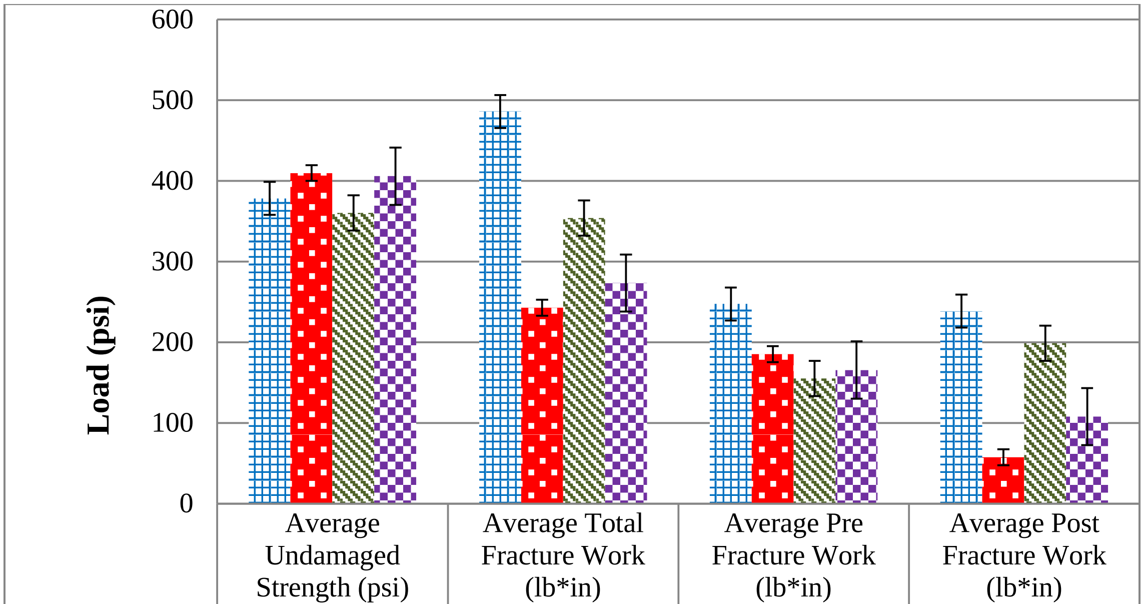


Figure 4 Individual loading curves of laboratory mixtures

This curve provides some other useful information as well. First information that can be retrieved is the load carrying capacity of the samples, in other terms, the strength of the samples. Second, pre-, post- and total work done on sample during the loading can be obtained. This information provides the crack initiation and propagation characteristics during the test. The total enclosed area by the curve is a showing of total fracture work done on the sample. As the total fracture work increases, the mixture shows more ductile behavior. The area enclosed between peak and the first portion of the curve is equal to the amount of work done on sample till crack initiation, which is pre-fracture work. Moreover, the area enclosed between second portion of the curve and the peak is equal to the amount of work done on sample during crack propagation, which is post fracture work.



	Average Undamaged Strength (psi)	Average Total Fracture Work (lb*in)	Average Pre Fracture Work (lb*in)	Average Post Fracture Work (lb*in)
CRHY	378	486	247	239
CRTB	410	243	185	57
CRHY-Field	360	354	155	199
LVSP-Field	406	273	166	108

Figure 5 IDT strength and fracture work at -10^{o}C

Figure 5 demonstrates IDT strength test results for both laboratory and field compacted mixtures. Although the undamaged strength values of laboratory and field CRHY and CRTB samples are close, all fracture works (total, pre and post) show considerable differences. These differences between CRHY and CRHY-Field can be attributed to the better control of CR addition on the lab prepared mixtures. The strength of CRTB mixtures is slightly higher than that of CRHY mix. On the other hand, the fracture work done on CRHY samples is significantly higher than fracture works on CRTB mixtures. This indicates that CRHY mixtures exhibit more ductile behavior. Figure 4 also support this finding since the area under the curve for CRHY is visibly larger than the CRTB.

3.3.5 Tensile Strength Ratio (TSR) Test for Moisture Susceptibility of Mixtures

Moisture susceptibility is one of the most common distress types that occur in the HMAs. Moisture within the asphalt pavements can result in the loss of cohesion within the asphalt binder and breakage of the adhesive bonds between aggregate particles and binder.

Although there are various test methods available for the assessment of asphalt mixtures moisture susceptibility, the Tensile Strength Ratio (TSR) has been adopted in this study due to the use of same equipment setup as in IDT strength test for thermal cracking. Specimen preparation and testing procedure are performed according to AASHTO T283, Standard Method of Test for Resistance of Compacted Hot Mix Asphalt (HMA) to Moisture Induced Damage.

Sample preparation for the TSR test differs significantly from other performance tests. At least six samples per mixtures, having a size of 150 mm diameter and 95±5 mm height with 7%±0.5% air voids need to be prepared. Loose mixture is maintained in the room temperature for 2±0.5 hours to cool down after mixing the aggregate batch with hot liquid binder. Following the cooling period, the same lose mixture is transferred into a forced air draft oven at $60^{o}C \pm 3^{o}C$ for another 16 hours. Subsequently, it is placed in a preheated oven at compaction temperature for 120±10 minutes prior to the compaction. The air void check is performed after an overnight cooling period at room temperature and samples are grouped into two subsets, each consisting 3 samples. One of the subsets is assigned as unconditioned set while the other subset is used for saturation and called as conditioning set. The unconditioned set is stored at room temperature two hours prior to the testing and temperature is maintained at $25^{o}C \pm 0.5^{o}C$ for 120±10 minutes before testing. The conditioned set is saturated to a level anywhere in between 70% to 80% under vacuum container. Saturated samples are wrapped with a plastic film and sealed in another bag containing 10±0.5 ml water. Covered conditioned set is stored in an environmental chamber for a minimum of 16 hours at -18^{o}C±3^{o}C. Following the freezing cycle, thawing cycle is performed by directly transferring the unsealed and unwrapped samples into a water bath at 60^{o}C±1^{o}C for another 24±1 hours. As a last conditioning step prior to testing, conditioned set samples are placed in a water bath having a temperature of 25^{o}C±0.5^{o}C for 120±10 minutes. Both conditioned and unconditioned tests are performed on MTS with a strain-controlled mode at 50 mm/min loading rate.

The tensile strength of each sample is calculated by using the dimensions and the maximum applied load during testing. The tensile strength can be calculated as;

$$S_{ten} = \frac{2 * P_{max}}{D * t * \pi} \tag{3.7}$$

where

S_{ten}: indirect tensile strength (psi)

P_{max}: maximum load applied (lbs)

D: diameter and

t: thickness of the sample (inch)

TSR is defined as the ratio of the indirect tensile strengths of conditioned set to unconditioned set. It can be calculated as follows;

$$TSR = \frac{S_1}{S_2} * 100\% \tag{3.8}$$

where

TSR: tensile strength ratio

S_1: average indirect tensile strength of conditioned set (psi)

S_2: average indirect tensile strength of unconditioned set (psi)

TSR ratio has to be greater than 80% so that the mixture will not be considered as susceptible to moisture damage.

It is a common practice to incorporate antistripping agents into the mixtures in a case of low TSR ratio is obtained. Most commonly used antistripping agent is hydrated lime. There are liquid additives available in the market as well.

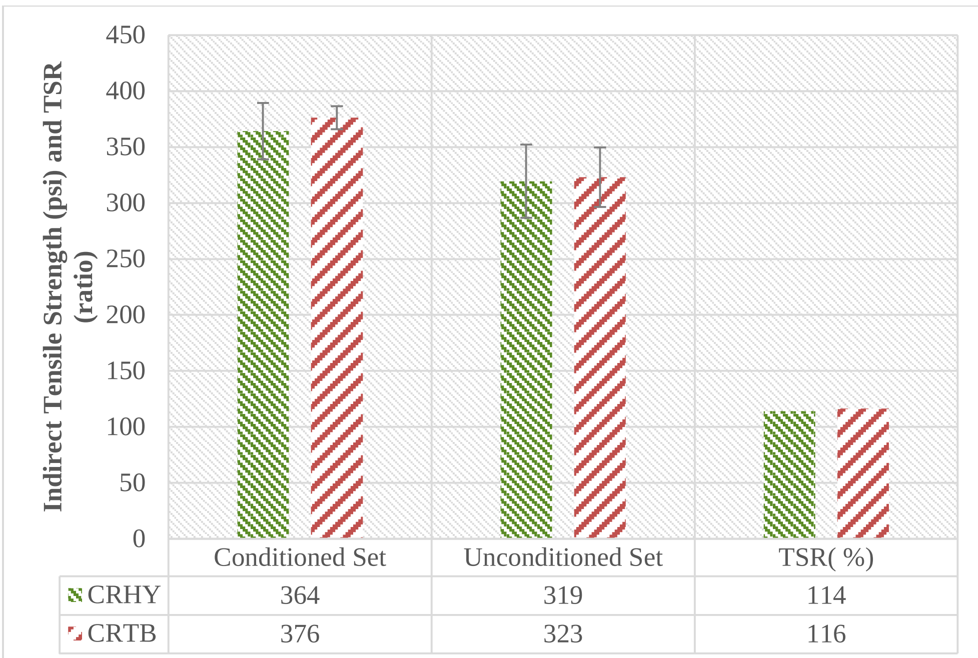


Figure 6 TSR test results

TSR tests were conducted on both CRTB and CRHY mixtures. The conditioned and unconditioned average indirect tensile strength of the mixtures along with TSR values were given in Figure 6. TSR of both mixtures were found to be greater than 80%. Therefore, there was no need to use of antistripping agent.

3.4 CONCLUSIONS

This project introduced a new crumb rubber modified asphalt method called CRHY, which is a combination of terminal blend (CRTB) and dry process (CRDry). The goal of such combination was to increase the CR content of CRTB modified asphalt mixtures by adding about 0.5% CR (by weight of the mix) via dry process. The CRHY method approximately doubled the rubber content in CRTB method by adding dry crumb rubber particles. The relative performances of CRTB and CRHY were investigated in terms of their susceptibility to rutting and moisture damage, and fatigue and thermal cracking. Rutting resistance was evaluated by using flow number (FN) tests and fatigue resistance was evaluated using the Push-Pull tests and the Viscoelastic Continuum Damage Theory (VECD). Low temperature cracking and moisture damage susceptibility were assessed by using indirect tensile (IDT) strength test. TSR for moisture damage susceptibility test results revealed that there was no need to add an antistripping agent to the mixtures. The project involved development of laboratory mix designs and performance tests (linear viscoelastic characterization, fatigue, and rutting, thermal cracking and moisture damage) to evaluate relative performance of CRHY as compared to CRTB. Laboratory tests run on the lab-designed mixtures revealed that the CRHY mixture made with dry crumb rubber is as good or better than CRTB mixtures in fatigue cracking, thermal cracking and moisture damage and slightly worse in rutting. As a result, the following conclusions can be drawn from performance test results:

- Although there was not any significant difference between dynamic modulus master curves of the mixtures, CRTB was the softest at low temperatures-high frequencies and CRHY-Field was the stiffest at high temperatures-low frequencies.
- The CRHY mixtures made with treated rubber technologies exhibited significant rutting susceptibility.
- Rutting susceptibility of hybrid mixtures (CRHY and CRHY-Field) was slightly worse than CRTB mix.

- Thermal cracking tests showed that the hybrid mixtures are more ductile than the CRTB. Even though the undamaged strength of CRTB mix was slightly higher than that of hybrid mixes, the total, pre and post fracture work values were significantly lower.
- Fatigue cracking behaviour of CRHY mixture showed superior performance over CRTB mixture at all temperature and micro strain combinations other than 25^{o}C-200ms.
- TSR test results revealed that there was no need for an antistripping agent in any of the mixtures.
- Overall, hybrid mixtures were as good or better than terminal blend mixture in fatigue and thermal cracking and slightly worse in rutting. Both mix types showed almost equal resistance against moisture induced damage.

The long term performance of the test section is currently being monitored and will be evaluated in future publications.

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REFERENCES

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CHAPTER 4 BINDER SOFTENING EFFECT OF RECYCLED TIRE RUBBER MODIFIERS FOR HIGH PERCENT RECLAIMED ASPHALT PAVEMENT MIXTURES

4.1 INTRODUCTION

Recycling old asphalt pavements is one of the major practices in asphalt industry today. Recycled/reclaimed asphalt pavement (RAP) is obtained by crushing and screening distressed asphalt pavements via milling or full depth removal (Hassan 2009, Copeland 2011). Recycling old asphalt pavement materials optimizes the use of natural resources in the production of hot-mix asphalt (HMA). Use of RAP in new hot mix asphalt (HMA) construction is a useful practice since it reduces the amount of naturally derived aggregates and virgin binder in the mixture. Increased RAP usage also provides economical savings (e.g., reduced freight cost of the virgin materials) and environmental benefits (e.g., conserves energy that is used while extracting natural aggregates and asphalt). Because of the continuous increase in the asphalt binder costs (which is directly related to gasoline costs), utilizing high percentages of RAP (up to 40-50%) is essential for lowering the production costs of asphalt pavements.

Each State has its own specification for incorporating RAP in HMA. Currently, Michigan Department of Transportation's (MDOT's) specification allows up to 17% RAP in HMA without any modification to the asphalt binder (Tier 1). In order to add between 17 and 27% RAP into HMA (Tier 2), low temperature grade of the asphalt binder is required to be lowered by one grade in high volume roads (i.e., roads more than 1 million Equivalent Single Axle Load (ESAL)). For using more than 28% RAP (Tier 3), blending chart analysis is required, where low and high temperature grade of the binder is determined based on amount of RAP used and the grade of the binder in the RAP. Since the binder in RAP is usually stiff and brittle, blending chart analysis leads to the requirement of soft virgin binder, which is typically more expensive (Daniel et al. 2010, APAM 2014). According to Asphalt Pavement Association of Michigan (APAM), in 2014, the price of asphalt binders PG 58-28 and PG 58-34 binders were $585/ton and $705/ton, respectively (APAM 2014). As a result, even though RAP is inexpensive to the contractors, they are reluctant to use high percentages of RAP since it leads to soft and expensive binder. An evidence of this is given in Figure 7 which shows the percentage of asphalt pavements constructed by MDOT in 2011 and 2012 with different RAP tiers (Kutay et al. 2013).

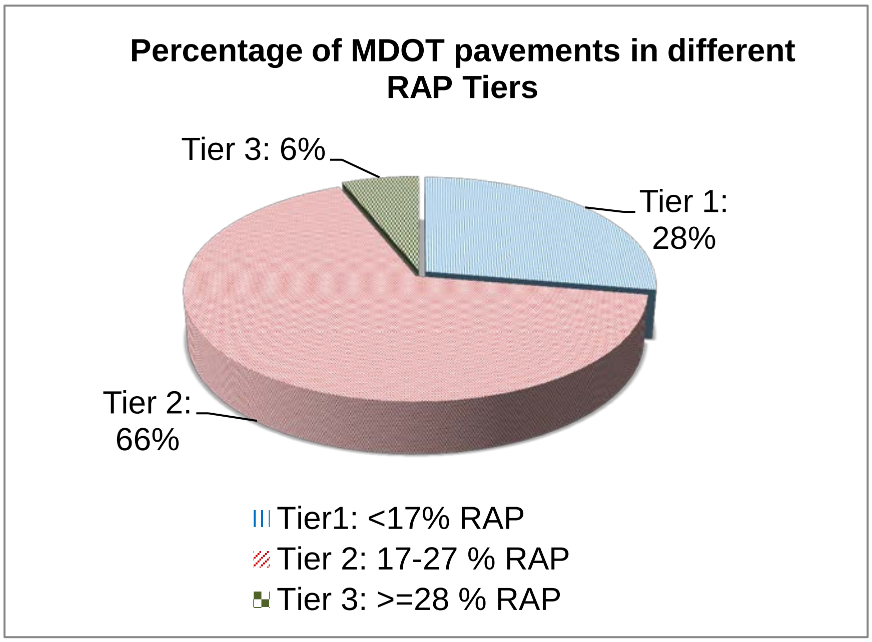


Figure 7 Percentage of pavements in different tiers constructed by MDOT in 2011 and 2012

As shown in Figure 7, 66% of the pavements constructed (in 2011 and 2012) were in Tier 2. This shows that the contractors prefer to use relatively high percentage of RAP at the expense of lowering the low temperature grade of asphalt binder. However, only 6% of the pavements were in Tier 3 because the binder becomes too expensive, and economical benefit of using high

RAP does not compensate the cost of the binder. As a matter of fact, among all the 65 pavements, the maximum RAP used was 30%, which indicates that the contractors did not use larger than 30% even though there is no upper limit.

The objective of this study was to determine the feasibility of using crumb rubber (CR) modified binders with high percentage RAP mixtures in lieu of expensive soft binders and blending chart analysis.

4.2 MATERIALS AND METHODS

Experimental plan of the study is shown in Figure 8.

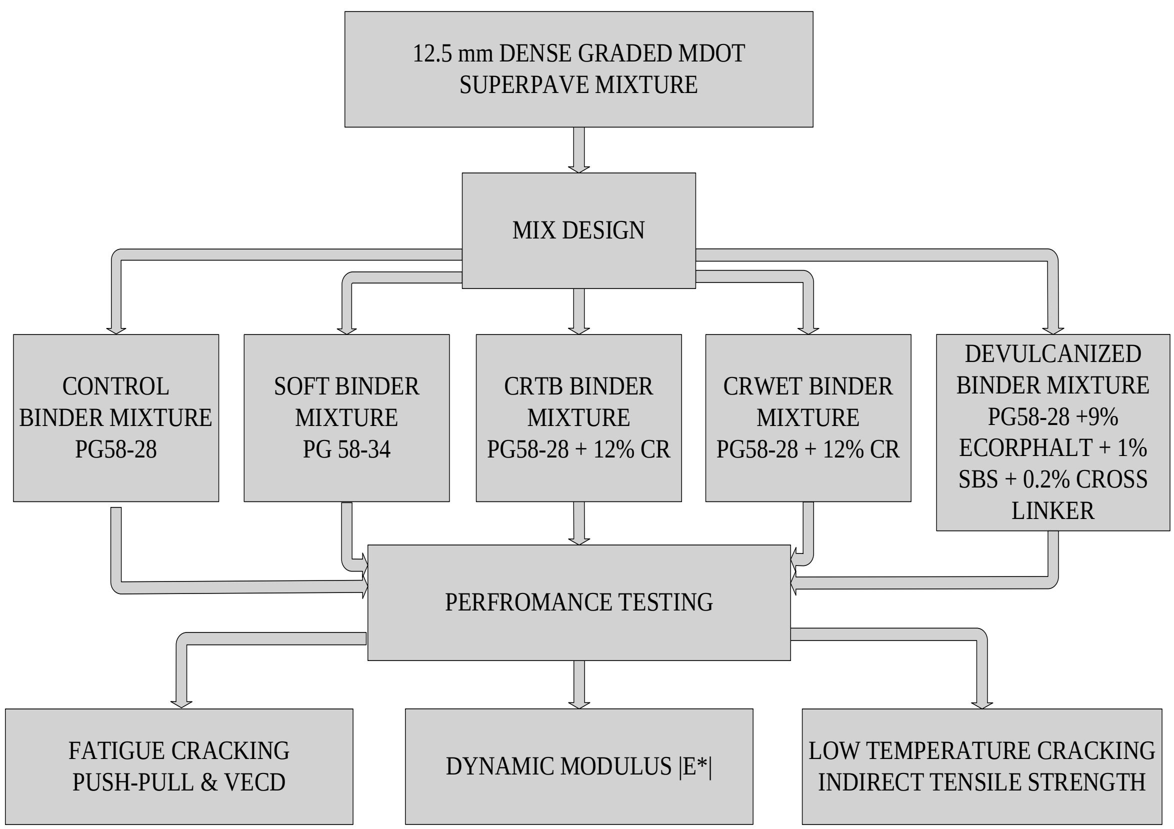


Figure 8 Experimental plan

4.2.1 Preparation of Binders

Figure 8 shows the experimental plan of this study, which included the following binders; (i) PG58-28 unmodified binder (control), (ii) PG58-34 grade bumped (soft) binder, (iii) crumb rubber terminally blend (CRTB) binder, (iv) crumb rubber modified binder made via wet process (CRWET) and (v) devulcanized crumb rubber (CRDEV) binder. The CRTB had a crumb rubber content of 12% by weight of binder and was produced at an asphalt terminal located in Chicago, IL. CRWET and CRDEV binders were prepared by using a high shear mixer in Advanced Asphalt Characterization Laboratories (AACL) of Michigan State University (MSU). To prepare the CRWET, 12% (by weight of binder) #20 mesh size CR was blended with base binder at 190^{o}C approximately at 700 revolutions per minute (rpm) for 35 minutes. The CRDEV binder was produced by first soaking the devulcanized rubber particles (9% by weight of the binder) in the asphalt binder at mixing temperature for two hours, and then mixed for 15 minutes at 3000 rpm. After mixing, the temperature was raised to 190^{o}C and 1% styrene-butadiene-styrene (SBS) was introduced into the blend. This blend was allowed to react for around 35 to 45 minutes at 3000 rpm till homogeneity was achieved. As the last step, 0.2% cross linker (by weight of the binder) was added and stirred for 15 more minutes.

4.2.2 Mixture Design

The nominal maximum aggregate size (NMAS) of all asphalt mixtures were 12.5 mm and designed in accordance with the MDOT Superpave specification. As it is illustrated in Figure 8, the same mix design was used to produce mixtures with different virgin and modified asphalt binders. Design aggregate gradation, specification limits and combined bulk specific gravity of the aggregates are provided in Table 6.

Table 6 Mixture design gradation and specification limits

Sieve Size	12.5 mm Superpave Dense Graded	12.5 mm Superpave Specification
19 mm (3/4")	100.0	100 min
12.5 mm (1/2")	90.6	90-100
9.5 mm (3/8")	81.3	90 max
4.75 mm (#4)	63.5	-
2.36 mm (#8)	48.6	28-58
1.18 mm (#16)	34.6	-
0.600 mm (#30)	22.4	-
0.300 mm (#50)	10.9	-
0.150 mm (#100)	5.8	-
0.075 mm (#200)	3.9	2-10
Binder Content (%)	4.80%	-
RAP Content (%)	40.0%	-
Bulk Specific Gravity, (Gsb)	2.668	-

Since RAP stockpiles exhibited more variations than the virgin aggregates, the RAP was separated into fine (material passing #4 sieve) and coarse portions to better control the mixture gradation and binder content. The binder content of the RAP was 4.53%. RAP also had an NMAS of 12.5 mm and it was mainly obtained from trimming the surface course of interstate roads in the State of Michigan. Mixing and compaction temperatures of PG58-28 and PG58-34 binders were based on the viscosity measurements. The mixing and compaction for CRTB mixture was based on the manufacturer's recommendation, which was around 175-180^{o}C. For CRWET and CRDEV, the mixing temperature varied from 160^{o}C to 173^{o}C, and the compaction temperature ranged from 147^{o}C to 163^{o}C. Table 7 shows the MDOT Superpave mix design requirements. These values are for an MDOT 4E1 Superpave mix design. The 4E1 mixture can be used in top (wearing) or levelling (intermediate) course (4th layer from the bottom) and expected to withstand 1 million ESAL traffic load (MDOT 2008). Mix design was performed by using the base binder to obtain the optimum binder content, which was 4.8%. Verification of the mix designs with the other binder types was performed using the same gradation and binder content. Since the volumetrics were within the specification limits, there was no need for varying aggregate gradation and binder content for the other binders.

Table 7 12.5mm MDOT Superpave mix design specification limits

% G_{mm} at (N_i) (maximum)		90.50%
Number of Gyrations	$N_{initial}$	7
	N_{design}	76
	N_{max}	117
Specific VFA (%) at N_d		65-78
Percent of Maximum Specific Gravity (%G_{mm}) at the Design Number of Gyrations (N_d)		96.0%
%G_{mm} at the Maximum Number of Gyrations, (N_m)		98.0%
VMA min % at N_d (based on aggregate bulk specific gravity, (G_{sb})		14
Fines to effective asphalt binder ratio (P#$_{200}$/P_{be})		0.6-1.2

4.2.3 Sample Preparation for Mixture Performance Tests

The Superpave gyratory compactor (SGC) was used to compact all the performance test specimens. The after batching and mixing, the loose mixtures were short term aged for 4 hours at 135^{o}C in forced-draft oven. To provide a uniform conditioning of the mixture, loose mixtures were stirred at each hour. After short term aging, mixtures were compacted to produce cylindrical samples having 150 mm diameter and 180 mm height. Compacted samples were allowed to cool down to the room temperature overnight. Depending on the performance test, cylindrical samples were either cut to the desired height or cored and cut to obtain more uniform smaller dimension cylindrical samples with smooth surfaces. The re-sized samples were dried prior to the determination of the bulk specific gravity (G_{mb}). Sample drying was accelerated via the Core-Dry equipment. Dried samples were ready to be tested under water to obtain the G_{mb}.

Only specimens having VTM of 7% ± 0.5% were accepted for performance testing. All procedures of specimen fabrication were conducted according to the relevant AASHTO standards (AASHTO 2013).

4.2.3.1 Dynamic modulus (|E|) tests*

Dynamic modulus (|E*|) tests were performed using the Asphalt Mixture Performance Tester (AMPT). The |E*| test is theoretically a non-destructive test run to determine the linear viscoelastic characteristics of the mixtures. The |E*| and phase angle values at various temperature and loading frequency levels are measured by applying axial haversine compressive stress. All samples were prepared in accordance with AASHTO PP60-14 "Preparation of Cylindrical Performance Test Specimens Using the Superpave Gyratory Compactor (SGC)". The AASHTO T342 "Determining Dynamic Modulus Mastercurve of Hot Mix Asphalt (HMA)" was followed to develop the |E*| master curve (AASHTO 2013). Each specimen was tested at temperatures of -10^{o}C, 10^{o}C, 21^{o}C, 37^{o}C, 54^{o}C. At each temperature, the tests were conducted at loading frequencies of 25Hz, 10Hz, 5Hz, 1Hz, 0.5Hz and 0.1Hz. Once |E*| values were measured at different temperatures (T) and loading frequencies (*f*), the |E*| master curve was obtained using the time-temperature superposition (TTS) principle. Based on the TTS principle, a single |E*| master curve can be obtained by shifting the |E*| data obtained at different temperatures horizontally in a log-log plot of |E*| versus frequency. Once shifted, the parameter in x-axis is called reduced frequency (f_R), which is defined as follows:

$$f_R = f * a_t(T) \tag{4.1}$$

where f is the frequency of the load and $a_T(T)$ is the shift factor coefficient for a given temperature T. The shift factor coefficient *(* a_T *(T)),* i.e., the amount of horizontal shift for each temperature is different. After the shifting is completed and the shift factor coefficients *(* a_T *(T))* are determined, they are plotted against each temperature (T). Then typically a second order polynomial is fit to the data to obtain the polynomial coefficients a1 and a2 in the following equation:

$$a_T(T) = 10^{a_1(T^2 - T_{ref}{}^2) + a_2 * (T - T_{ref})} \tag{4.2}$$

where T_{ref} is the reference temperature. During shifting process, the shift factors *(* a_T *(T))* at each temperature are varied until a good sigmoid fit to the |E*| data of all temperatures is obtained. Typically the following sigmoid function is used:

$$\log(|E^*|) = b_1 + \frac{b_2}{1 + \exp(-b_3 - b_4 \log(f_R))} \tag{4.3}$$

where b_1, b_2, b_3 and b_4 are the sigmoid coefficients, and f_R is the reduced frequency.

4.2.3.2 Push-Pull fatigue tests

Push-Pull (PP) is a uniaxial compression-tension fatigue test run on cylindrical samples having the same (or similar) dimensions as those used in |E*| testing. The samples were equipped with steel tabs where three LVDTs were mounted around the circumference, and aluminum end plates to attach them to testing frame. Figure 9 shows pictures of a PP test specimen on MTS fixture and broken samples after testing.

Figure 9 Pictures of a push-pull test specimen on MTS fixture and after testing

The most important step during the PP sample preparation is to attach the end plates. If they cannot be placed parallel to each other, it will lead to an end-failure of the samples because of the induced eccentricity. Moreover, improper gluing of the plates can cause the detachment of the plate from sample surface. The tests in this study were performed by using a material testing system (MTS) in controlled actuator displacement mode. The displacement level at the actuator was initially selected such that about 900 microstrain was applied from one end of the sample to the other end. However, because of significant machine compliance issues, the on-specimen LVDT measurements showed that the actual strain level the sample experienced was only about 200 microstrains. The frequency of all the PP tests was 5Hz, except one sample which was tested at 1Hz. The sample tested at 1Hz was used as a verification of the Viscoelastic Continuum Damage (VECD) model that is used to compare the mixtures in this study. The test temperatures varied from $10^{o}C$ to $23^{o}C$. As explained later, the VECD model actually needs one PP test at a specified temperature and frequency to be calibrated. Once calibrated, the VECD model can simulate both stress and strain control tests at different temperatures and frequencies.

4.2.3.3 Indirect tensile strength (IDT) tests

Indirect tensile (IDT) strength is typically used for evaluating the low temperature cracking susceptibility of mixtures. IDT strength samples were cut from 150 mm diameter SGC specimens to a height of 44±6 mm. In accordance with the AASHTO T-322, only the samples having 7% ± 0.5% air voids were allowed for testing. IDT strength tests were conducted at -10^{o}C using a loading rate of 12.5 mm/min. MTS with a special loading fixture was utilized to apply the required axial loading at controlled-displacement mode. At least three replicates for each mixture were tested.

One of the improvements to AASHTO T-322 proposed in NCHRP 530 report was IDT strength testing without the use of LVDTs to monitor deformations. LVDTs were initially incorporated into AASHTO T-322 to determine the precise moment of failure during the IDT test. Despite the accuracy and precision of the exact failure moment determination with the LVDTs testing at state agencies, the FHWA, and regional Superpave Centers reported feasibility issues with using the LVDTs. LVDTs were found to be difficult to keep in place and due to the explosive nature of specimen failure, the expensive and delicate LVDTs were at risk to be damaged during testing, potentially jeopardizing overall reliability and accuracy of the IDT strength test. As a result of feedback from testing centers, an empirical relationship was developed between the true (corrected) and uninstrumented (uncorrected) strength values as follows (Christensen et al. 2004):

$$\text{True IDT Strength (psi)} = \left[0.781 \times \text{Uncorrected IDT Strength (psi)}\right] + 38 \qquad (4.4)$$

The IDT strength at maximum load is termed the uncorrected IDT strength in Equation 1. The IDT strength test can also be used to measure a mixture's fracture energy (Kim et al. 2002, Li et al. 2010) and fracture work (Wen 2013). Fracture energy of a mixture is defined as the area under the stress versus strain curve, while the fracture work is the area under the load versus horizontal displacement curve. Fracture work measured by the IDT test has been shown to correlate well with field performance, whereas fracture energy has been shown not to correlate well with field performance (Wen 2013). This is in part due to fracture work's ability to capture the entire post-peak behavior, whereas fracture energy often does not capture the entire behavior due to limits in the range of LVDT measurements. Another advantage of fracture work is the elimination of LVDT instrumentation during testing. Vertical ram movement has been shown to be no different than LVDT measured horizontal displacement (Wen 2013). This enables rapid testing and eliminates LVDT damage. For these reasons the use of fracture work was used in this study as an additional method to characterize thermal cracking susceptibility of the mixtures.

4.3 RESULTS AND DISCUSSION

4.3.1 Linear Viscoelastic Characterization of Asphalt Mixtures

Figure 10(a) and Figure 10(b) show the comparison of master curves of mixtures in log-log and log-linear scale, respectively.

The log-log scale plot shown in Figure 10(a) allows better differentiation of the |E*| values at high temperatures/low frequencies (lower left side of the graph), whereas linear-log scale plot in Figure 10(b) allows better differentiation of the |E*| values at low temperatures/high frequencies (upper right side of the graph). As shown in Figure 10(a), at high temperatures/low frequencies, CRTB had the highest stiffness, followed by the Control and CRWET. The PG58-34 and CRDEV mixtures had very similar stiffness and lower than the rest of the mixtures at high temperatures/low frequencies.

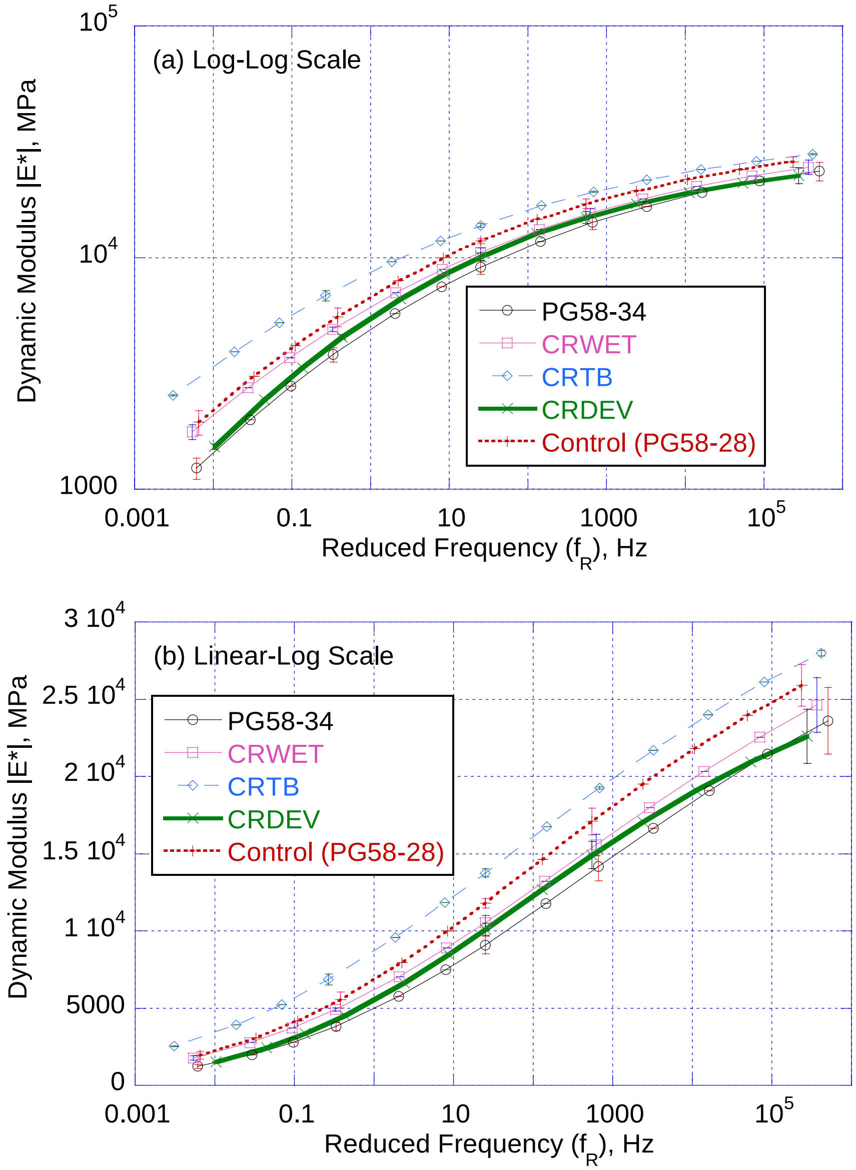


Figure 10 Dynamic modulus (|E*|) master curves of the asphalt mixtures: (a) log-log scale and (b) log-linear scale. The reference temperature is 21°C

As shown in Figure 10(b), at low temperatures/high frequencies (upper right side of the graph), CRTB also had the highest stiffness, followed by the Control. This was somewhat unexpected since the crumb rubber modified mixtures are usually softer at low-temperatures. A possible reason for this behavior may be attributed to the other ingredients that exist within the CRTB binder, which are not disclosed to the authors because of their proprietary nature. The

CRWET, CRDEV and PG58-34 mixtures all had less stiffness than the CRTB and Control. It is noted that at lower frequencies and higher temperatures, stiffer mixtures typically exhibit better rutting resistance. On the other hand, soft mixtures at low/intermediate temperatures are expected to perform better in fatigue and low temperature cracking.

Table 8 shows the dynamic modulus (|E*|) master curve coefficients of the mixtures.

Table 8 The dynamic modulus (|E*|) mastercurve coefficients of the mixtures

Coefficient	**PG58-34**	**CRWET**	**CRTB**	**CRDEV**	**Control (PG58-28)**
$\mathbf{a_1}$	4.61E-04	3.66E-04	2.76E-04	4.35E-04	3.09E-04
$\mathbf{a_2}$	-1.44E-01	-1.39E-01	-1.39E-01	-1.35E-01	-1.32E-01
$\mathbf{T_{ref}}$ **(°C)**	21.0				
$\mathbf{b_1}$	-0.28	0.19	0.90	0.22	0.54
$\mathbf{b_2}$	4.82	4.37	3.67	4.25	4.01
$\mathbf{b_3}$	1.55	1.55	1.57	1.59	1.53
$\mathbf{b_4}$	0.32	0.31	0.32	0.37	0.34

4.3.2 Viscoelastic Continuum Damage (VECD) Analysis of PP Tests

In order to be able to compare the fatigue lives of different mixtures at different strain levels and temperatures, the push-pull fatigue data measured in the laboratory was analyzed using the viscoelastic continuum damage (VECD) model (Kutay 2014). In VECD model, so-called ‘damage characteristics (C versus S) curve’ of a mixture is computed from the cycles-peak stress-peak strain data (Kutay et al. 2008). Once the damage characteristics curve is obtained for a mixture, the VECD model can predict the fatigue life at any temperature and frequency for the desired strain level. In this research, PP-VECD software, which is an implementation of the VECD formulation given in (Kutay et al. 2008), was utilized to perform VECD analysis (Kutay 2014).

As part of the analysis of the uniaxial cyclic push-pull tests using the VECD model, the mixture |E*| master curve is needed. Once the |E*| master curve is known, a single push-pull test (either stress or strain controlled) at a selected frequency and temperature is sufficient to calibrate the VECD model (i.e., C versus S relationship). However, in order to confirm the collapse of C versus S curves at different loading conditions, it is suggested to run the tests for at least two different temperatures (e.g., 10^{o}C and 20^{o}C) at a selected frequency (e.g., f =5Hz). The summary of the steps of the VECD-based analysis of mixtures is listed below:

Step 1. Perform dynamic modulus (|E*|) tests and develop the |E*| master curve.

Step 2. Compute the relaxation modulus (E(t)) master curve through viscoelastic inter-conversion (e.g., using the procedure in (Park et al. 1999) and calculate the damage exponent α= 1/*m*, were *m* is the maximum slope of the relaxation modulus versus time graph drawn in log-log scale, i.e.:

$$m = \max\left\{\frac{\Delta\log(E(t))}{\Delta\log(t)}\right\} \tag{4.5}$$

Step 3. Conduct push-pull (tension-compression) tests at an intermediate temperature (e.g., 20^{o}C) in strain (or stress) control mode. Develop C (pseudostiffness) versus S (damage parameter) curve using the following formulation (Kutay et al. 2008):

$$\varepsilon_N^R = |E^*|_{LVE} * \varepsilon_0^N \tag{4.6}$$

$$C_N = \frac{|E^*|_N}{|E^*|_{LVE}} \tag{4.7}$$

$$S_{N+\Delta N} = S_N + (\Delta N/\mathrm{f})^{\frac{1}{1+\alpha}}\left[-0.5\,\mathrm{I}\,\varepsilon_N^{R2}\left(C_{N+\Delta N} - C_N\right)\right]^{\frac{\alpha}{1+\alpha}} \tag{4.8}$$

where $|E^*|_{LVE}$ is the linear viscoelastic (undamaged) dynamic modulus, ε_0^N is the peak strain, $|E^*|_N$ is the dynamic modulus measured in N^{th} cycle, ε_N^R is the peak pseudostrain at N^{th} cycle, S_N and $S_{N+\Delta N}$ are the damage parameters in N^{th} and $(N+\Delta N)^{th}$ cycles, C_N is the pseudostiffness in N^{th} cycle, ε_N^R is the peak pseudostrain computed in N^{th} cycle, f is the frequency and I is sample-to-sample variability parameter and calculated as $I = |E^*|_{N=1} / |E^*|_{LVE}$ where $|E^*|_{N=1}$ is the dynamic modulus in push-pull test at first cycle and $|E^*|_{LVE}$ is the linear viscoelastic dynamic modulus that is obtained from the dynamic modulus master curve. It is noted that both stress and stress controlled tests should result in the same C (pseudostiffness) versus S (damage parameter) curve. It may be preferable to run two separate tests with stress and strain control modes to verify if the C versus S curves collapse on a single curve. If desired, fit a pre-defined curve to C versus S curve, but this step is not required for N_f formula shown in Equation 5.

Step 4. Select a failure criterion (e.g., C=0.5, 50% reduction in modulus), strain level, frequency and temperature and calculate N_f using the following equation (Kutay et al. 2009):

$$\mathrm{N_f} = \sum_{i=1}^{N_S} \left[-\frac{\varepsilon_0^2 \left|E^*\right|_{LVE}^2}{2} \left.\frac{\mathrm{dC}}{\mathrm{dS}}\right|_{at\,S_i} \right]^{-\alpha} f\,\Delta \mathrm{S_i} \tag{4.9}$$

where N_f = the number of cycles to failure, N_S = the number of discrete intervals of S up to the S_f where S_f = damage parameter at failure (or when C=0.5 value), ε_0= the selected strain level, $|E^*|_{LVE}$ = the magnitude of linear viscoelastic dynamic (complex) modulus, $\left.\frac{dC}{dS}\right|_{at\,S_i}$ = the rate of change of C with respect to S (slope of C vs. S curve) at a given S_i value, f = the reduced frequency, $\Delta S_i = S_{i+1} - S_i$

Steps 1 through 4 are repeated for each HMA mixture and the specimens are ranked based on N_f.

The pseudostiffness (C) versus damage parameter (S) curves of the specimens can indicate differences between different HMA types. However, it should be noted that the mixtures should not be ranked based on the C versus S curve since it is a normalized curve that combines the effects of temperature, frequency and strain (or stress) level. It is important to rank the mixtures with respect to their response at a given temperature, frequency and strain (or stress) level (Kutay et al. 2008). One method, as described by Kutay et al. (2008), is to perform simulation of a (truly) strain controlled tension-compression fatigue test and look at the reduction in |E*| with increasing number of loading cycles. Another method is to select a failure criterion (e.g., 50% reduction in stiffness(C=0.5)) and compute the N_f using the equation 4.9 suggested by Kutay et al. (2009).

In order to verify the validity of the VECD procedure, the PP tests for each mixture were performed at different temperatures and frequencies. This was needed to make sure that the damage characteristic (C vs S) curves collapse in a single curve for different temperatures, frequencies and strain levels. Added value of using VECD model is to minimize the number of testing samples. While each PP test provides N_f values for only one temperature, frequency and microstrain combination, VECD model can provide N_f values at any temperature, frequency and microstrain level once the model is calibrated by using as little as two PP samples.

As shown in Figure 11, C vs S curves of each mixture collapsed on a single curve with a small variation regardless of temperature and frequency, indicating the validity of the VECD approach. Using these C vs S curves, strain controlled test simulations were performed to compute the number of cycles to failure based on 50% reduction in stiffness criterion.

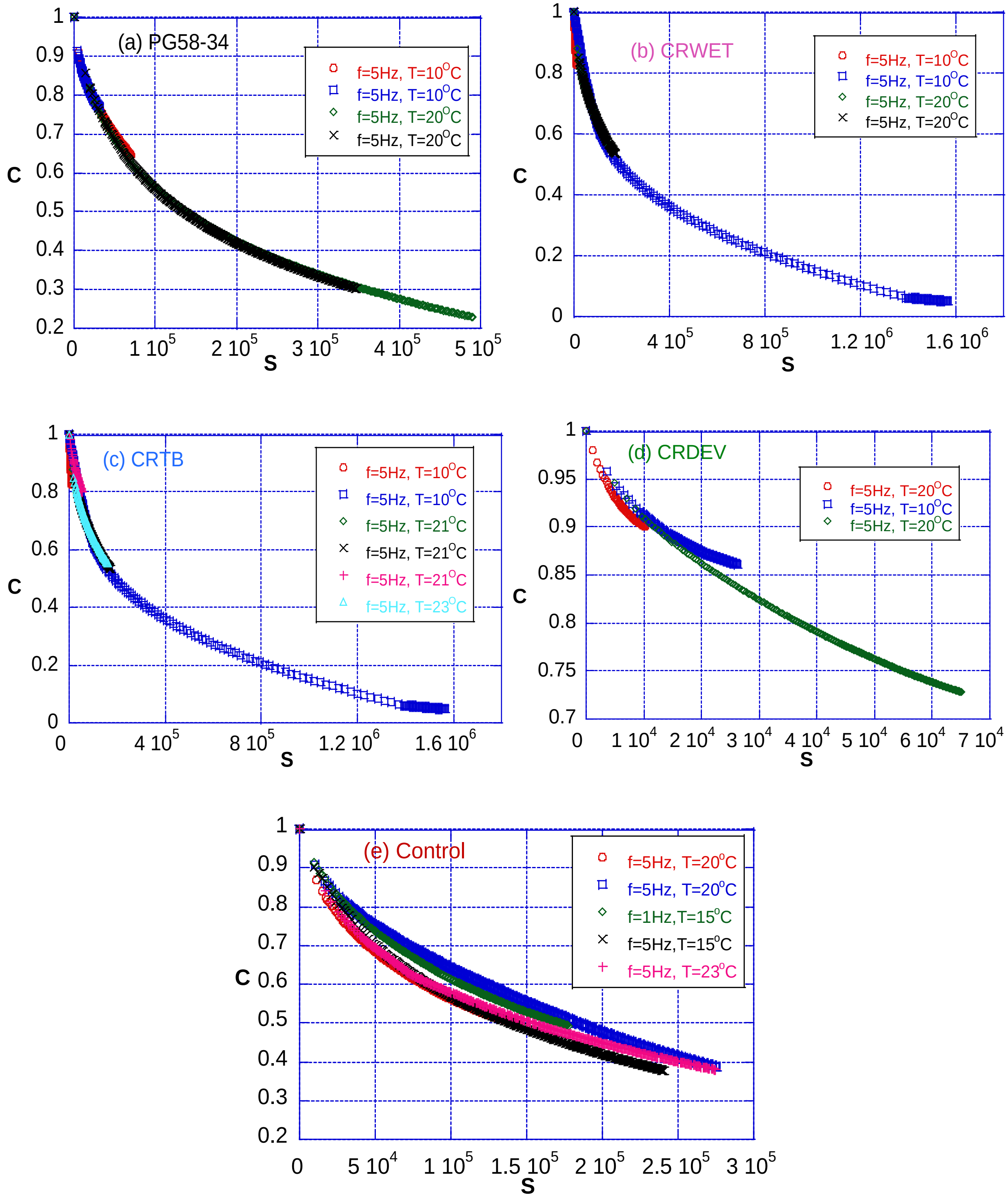
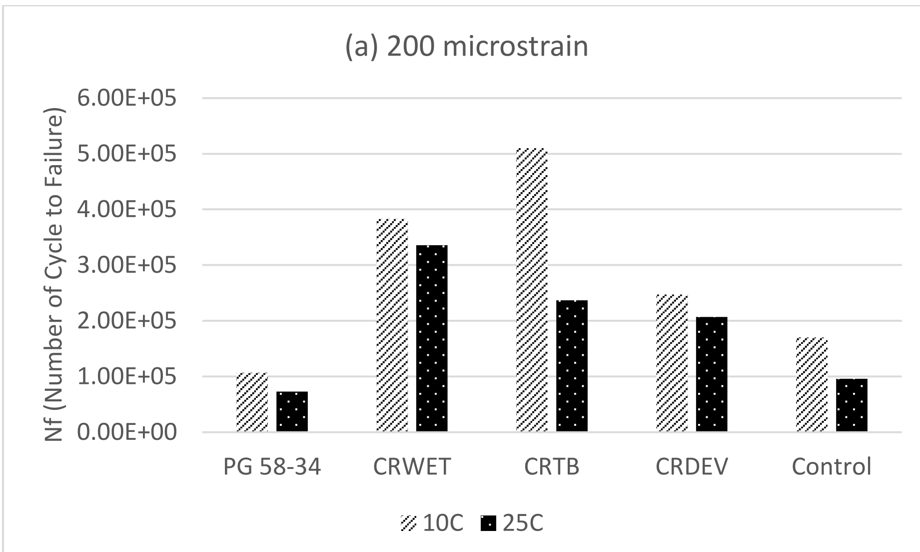


Figure 11 Damage characteristic curves (C vs S curves) of asphalt mixtures

Figure 12(a) and Figure 12(b) illustrate the number of cycles to failure (N_f) computed at 200 and 800 microstrains, respectively. Each graph shows the N_f at two different temperatures, namely 10^oC and 25^oC. As shown, the CRTB performs the best at low strain level (200 microstrain), whereas, it performs very similar to the Control and worse than the rest of the modified mixtures at 800 microstrain. The PG58-34, on the other hand, performed better than the Control in 800 microstrain level, whereas, it performed worse than the Control at 200 microstrain level.



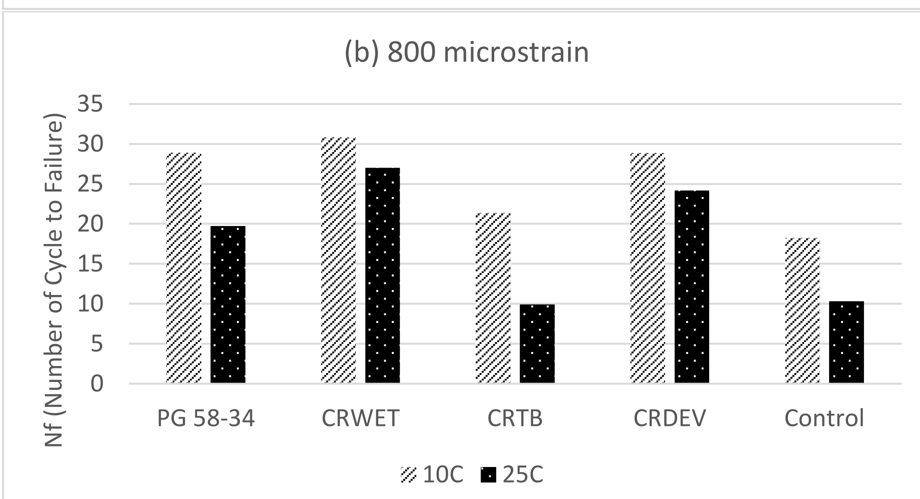


Figure 12 VECD analysis results for (a) 200 microstrain & (b) 800 microstrain

Both CRWET and CRDEV performed better than the Control as well as the mixture made with the PG58-34 binder at 200 and 800 microstrain levels. It can be concluded that the CRWET and CRDEV technologies are as good or better than the PG58-34 mixture; therefore, the PG58-34 binder can be substituted with the CRWET or CRDEV technologies. It appears that the benefit of the CRTB mixture can be observed at relatively low strain levels. At high strain levels, the improvement in the fatigue life due to the use of CRTB technology was minimal.

4.3.3 Thermal Cracking Performance of Asphalt Mixtures by IDT Tests

Figure 13 shows the load versus displacement curves obtained from the low-temperature Indirect Tensile (IDT) strength test run on each mixture.

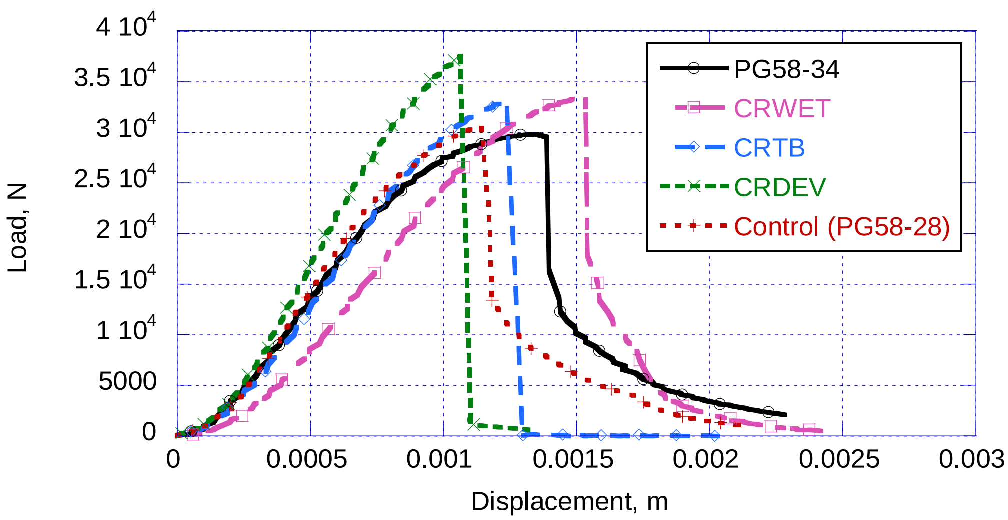


Figure 13 Load versus displacement curves of the indirect tensile (IDT) strength test

The area under the load-displacement curves corresponds to the total fracture work done on the sample during the testing. Total work includes the pre- and post-peak fracture work. Specimens typically start to fail when the peak stress is reached. The area under the curve until the peak of the curve is the pre-peak fracture work, which is thought to be related to the energy required for crack initiation in the sample. On the other hand, the area under the curve after the peak is termed post-peak fracture work, which is associated with crack propagation (Zborowski 2011, Wen 2013).

Figure 14a and Figure 14b show, respectively, the IDT strength and the fracture work (both pre-peak and post-peak) for all the mixtures tested. Based on the data illustrated in Figure 14, the following conclusions can be made:

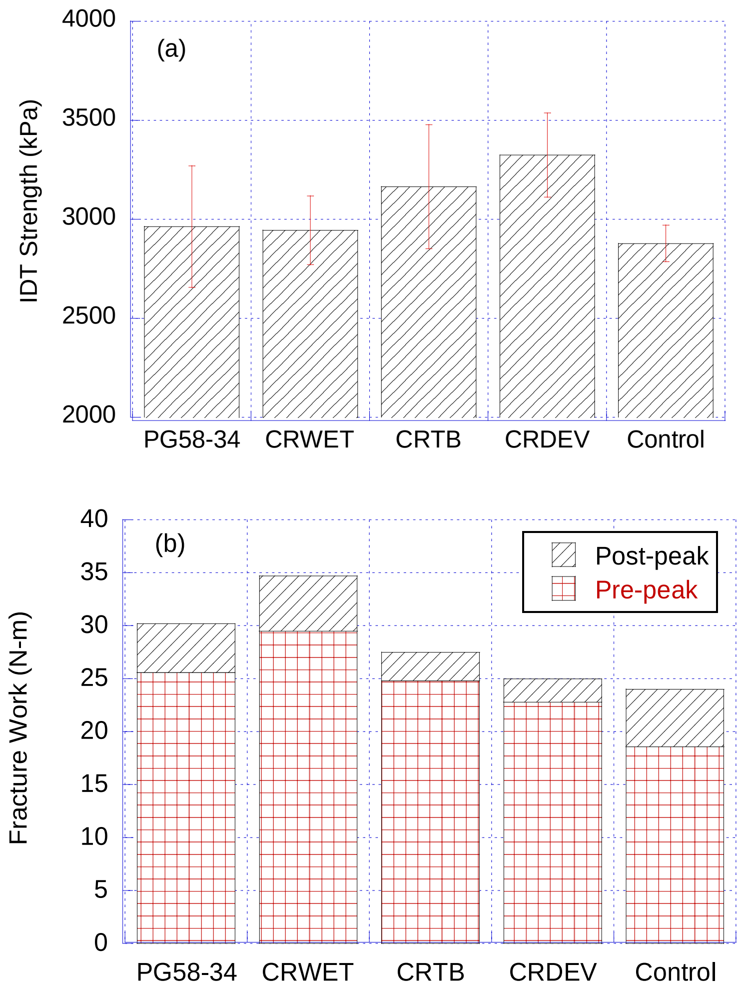


Figure 14 a) IDT strength and b) fracture work at -10^{o}C at 12.5 mm/min loading rate

- Mixture using devulcanized binder (CRDEV) has the highest IDT strength, which was followed by CRTB, PG58-34, CRWET and the Control mixtures.
- CRWET had the highest total fracture work, followed by PG58-34, CRTB, CRDEV and Control. From this data, CRWET is expected to perform as good or better than the grade bumped PG58-34 mixture in terms of low temperature cracking. CRTB and CRDEV are expected to perform slightly worse than the PG58-34, but better than the Control mixture.
- Pre-peak fracture work ranking followed the same trend as the total fracture work. The post peak fracture work values for all mixtures were very low, indicating rapid failure after the crack initiation.
- While devulcanized rubber had the highest IDT strength, it had the lowest fracture work among CR modified mixtures.
- Even though the IDT strengths of the PG 58-28 and PG 58-34 mixtures within variability limits, PG 58-34 mixture exhibited better total fracture work than PG 58-28 control mixture.

4.4 FATIGUE PERFORMANCE EVALUATION USING AASHTOWARE PAVEMENT ME DESIGN

The fatigue performances of the mixtures were analyzed by using AASHTOWare Pavement ME Design software. Since the Pavement ME software uses the equations developed for four-point bending beam (FPBB) laboratory fatigue test, the calibration of parameters between FPBB and viscoelastic continuum damage (VECD) analyzed Push-Pull (PP) are conducted as a first step.

4.4.1 Calibration of Fatigue Model Parameters Between FPBB and VECD Analyzed PP Fatigue Tests

Fatigue model used by the Pavement ME software was originally developed by using data obtained from FPBB tests performed at different temperatures and strain levels. Equation 4.10 shows the original material model:

$$N_f = k_1 \left(\frac{1}{\varepsilon_t} \right)^{k_2} \left(\frac{1}{E} \right)^{k_3} \tag{4.10}$$

where N_f is the number of cycles to failure and k_1, k_2, and k_3 are the empirical constants. The Pavement ME software uses a modified version of Equation 4.10, as listed in Table 9. The modifications were made to better reflect the field conditions and allow calibration of the models through the use of calibration factors β_{f1}, β_{f2} and β_{f3}.

Table 9 Material level fatigue damage model in Pavement ME for AC

Distress	Revised material model in Pavement ME	Calibration factors (1)		Variable definitions
Bottom-up fatigue (AC)	$N_{f-bu} = \beta_{f1} k_{f1} C C_{H-bu} \left(\frac{1}{\varepsilon_{t-bu}} \right)^{\beta_{f1} k_{f2}} \left(\frac{1}{E} \right)^{\beta_{f3} k_{f3}}$	$C_{H\text{-}bu}$	C, β_{f1}, β_{f2} and β_{f3}	N_{f-bu} = N_f for bottom-up cracks N_{f-td} = N_f for top-down cracks ε_{t-bu} = tensile strain at the bottom ε_{t-td} = tensile strain at the surface E = modulus/stiffness
Top-down fatigue (AC)	$N_{f-td} = \beta_{f1} k_{f1} C C_{H-td} \left(\frac{1}{\varepsilon_{t-td}} \right)^{\beta_{f1} k_{f2}} \left(\frac{1}{E} \right)^{\beta_{f3} k_{f3}}$	$C_{H\text{-}td}$		

Notes: (1) $C = 10^{4.84\left(\frac{V_{be}}{V_a + V_{be}} - 0.69\right)}$ where V_{be}=effective asphalt content by volume, $V_{a=}$ percent air voids in the HMA mixture.

$$C_{H-bu} = \left(b_{bu1} + \frac{b_{bu2}}{1 + e^{(b_{bu3} - b_{bu} h_{ac})}} \right)^{-1}$$

where h_{ac}= height of the AC layer and b_{bu1}=0.000398, b_{bu2}=0.003602, b_{bu3}=11.02, b_{bu4}=3.49

$$C_{H-td} = \left(b_{td1} + \frac{b_{td2}}{1 + e^{(b_{td3} - b_{td4} h_{ac})}} \right)^{-1}$$

where h_{ac}= height of the AC layer and b_{td1}=0.01, b_{td2}=12, b_{td3}=15.676, b_{td4}=2.8186

Since the PP fatigue test was used in lieu of FPBB test in this study, first comparison of the N_f values obtained from these two tests were performed. Figure 15 shows a comparison of the N_f values obtained from PP and FPBB data used in Mogawer et al. (2013). As shown, a decent correlation between the two sets of N_f values is visible. From the data shown in Figure 15, the following empirical equation was developed to convert the N_f obtained from PP tests to N_f obtained from FPBB tests:

$$N_f^{FPBB} = 3716.4 \left(N_f^{PP} \right)^{0.612} \tag{4.11}$$

In order to perform the calibration of the Pavement ME software's fatigue model, Equation 4.10 was used to obtain the k_1, k_2 and k_3 values using N_f^{FPBB} obtained from Equation 4.11. Then, the following relations were used to calculate β_{f1}, β_{f2} and β_{f3}: $k_1 = \beta_{f1} k_{f1}$, $k_2 = \beta_{f2} k_{f2}$ and $k_3 = \beta_{f3} k_{f3}$, where k_{f1}=0.007566, k_{f2}=3.9492 and k_{f3}= 1.281 represent the k_1, k_2 and k_3 values of a 'baseline mixture'. A list of β_{f1}, β_{f2} and β_{f3} values are shown in Table 10.

After the N_f values are calculated using equations listed in Table 9, the Pavement ME Design software uses damage accumulation models and transfer functions to calculate the actual top-down fatigue and bottom-up fatigue distresses. Table 11 illustrates the damage accumulation models and the transfer functions used in the software.

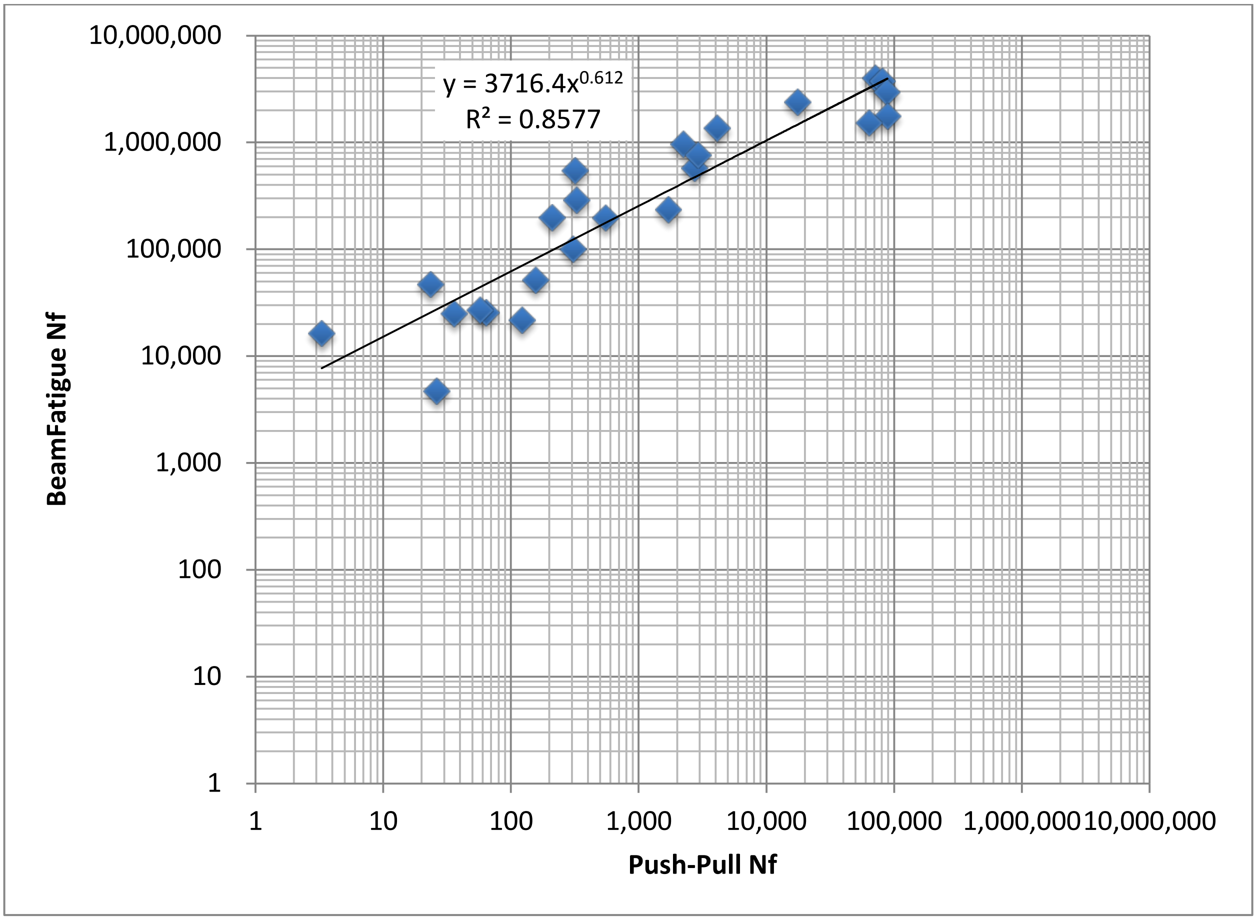


Figure 15 Comparison of the number of cycles to failure (N_f) obtained from Push-Pull tests and Four Point Bending Beam (FPBB) tests.

Table 10 shows the β_{f1}, β_{f2} and β_{f3} values obtained for the mixtures used in this study.

Table 10 Material calibration factors for tested mixtures

	PG 58-28	PG 58-34	CRTB	CRWET	CRDEV
β_{f1}	0.16442	1.16565	0.16801	0.17329	0.16601
β_{f2}	0.95251	0.87578	1.00212	1.00541	0.93223
β_{f3}	0.86021	0.83272	0.94728	0.95776	0.78334

Table 11 Fatigue transfer functions and distress equations in Pavement ME

Distress	Damage Accumulation Model	Transfer Function	Field Calibration Factors
Bottom-up fatigue (AC)	$D_{bu} = \sum_{i=1}^{TP} \frac{n_i}{N_{f-bu-i}}$	$FC_{bu} = \left(\frac{1}{60}\right)\left(\frac{C_{4-bu} c_4^*}{1+e^{C_{1-bu} C_1^* + C_{2-bu} C_2^* \log D_{bu}}}\right)$	$C_{1\text{-bu}}$, $C_{2\text{-bu}}$, $C_{4\text{-bu}}$
Top-down fatigue (AC)	$D_{td} = \sum_{i=1}^{TP} \frac{n_i}{N_{f-td-i}}$	$FC_{td} = \left(\frac{1}{60}\right)\left(\frac{C_{4-td} c_4^*}{1+e^{C_{1-td} C_1^* + C_{2-td} C_2^* \log D_{td}}}\right)$	$C_{1\text{-td}}$, $C_{2\text{-td}}$, $C_{4\text{-td}}$

D_{bu} = Bottom-up crack damage, N_{f-bu-i} = N_f for bottom-up cracks for period I,

D_{td} = Top-down crack damage, N_{f-td-i} = N_f for top-down cracks for period i

TP = number of periods, n_i = traffic cycles in a period

$C_1^* = -2C_2^*$, $C_2^* = -2.40874 - 39.748(1+h_{ac})^{-2.85609}$ where h_{ac} = height of the AC layer.

4.4.2 Traffic, Climate and Pavement Structure Inputs in Pavement ME Software

Since this study included both laboratory characterization and field construction of the asphalt mixtures, the actual location, climate and pavement structure has been used in the analyses. The climate data is obtained from Lansing, MI climate station (Station 14863). The traffic information used in the analysis is provided in Table 12. Other inputs were setup at default values.

Table 12 Design traffic information

Traffic Parameters	
Initial two-way AADTT	8200
Number of lanes in design direction	1
Trucks in design direction (%)	50
Trucks in design lane (%)	100
Operational Speed (km/h)	50
Design Life (years)	20

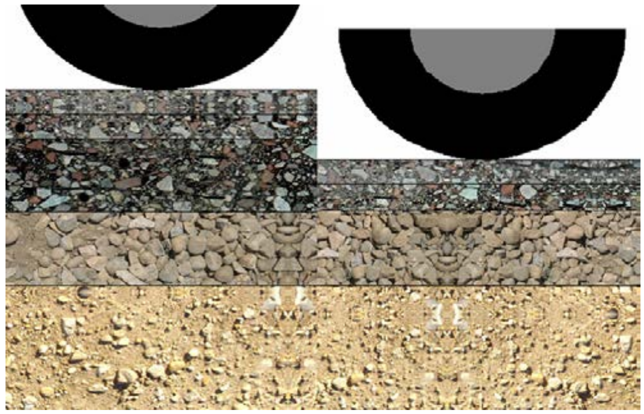


Figure 16 Pavement structure design for thick and thin layers

Two pavement structures (thick and thin asphalt concrete layers) are illustrated in Figure 16. Both structures included a 150 mm (6”) thick unbound base (AASHTO soil classification: A-1-a) overlain by a semi-infinite subgrade layer (AASHTO soil classification: A-1-a). The AC layer thicknesses for the thick structure were 50mm, 50mm and 150mm for the surface, intermediate and base courses, respectively. The AC layer thicknesses for the thin layered structure were 50mm and 50mm for the surface and base courses, respectively.

4.4.3 Mixture and Binder Level-1 Data Inputs

Dynamic Modulus |E*| of the asphalt mixtures and Complex Shear Modulus (|G*|) of the asphalt binders were input as level-1 for flexible pavement analysis using Pavement ME software. |E*| of each mixture was obtained by averaging at least 3 replicates tested at 5 temperature and 6 frequency levels. Table 13 shows the control mixture |E*| values used as a level-1 input in Pavement ME software. Another level-1 input is the binder complex shear modulus (|G*|) and phase angle (δ). Rheological parameters |G*| and δ are obtained by performing temperature- frequency sweep test on DSR equipment. In order to cover wide range of temperatures, test is performed by using 8 mm and 25 mm parallel plate geometries.

Table 13 Example |E*| data used in Pavement ME

Temperature (°C)	**Frequency (Hz)**					
	0.1	**0.5**	**1**	**5**	**10**	**25**
-10	18022.5	21608.5	22793.5	24870	25690	26673
10	8247.5	10457.5	11488	13959	15140	16627.5
21	4058	5546	6310	8408	9396	10933
37	1148	1906.5	2293.5	3624.5	4301.5	5227
54	271.85	492.25	630.05	1142	1469.5	1953

Although |G*| and δ are acquired for a range of frequencies at various temperature levels, only the values at 1.59 Hz (10 rad/sec) at each temperature level are used as inputs into the software. Table 14 shows an example of |G*| and δ values entered in to software as level 1 data.

Table 14 Example of complex shear modulus and phase angle data used in Pavement ME

Temperature (°C)	**Shear Modulus (Pa)**	**Phase Angle (deg)**
15	3133539	55.40
30	735688	59.12
46	107333	58.36
60	28181	58.67
70	7218	59.60

4.4.4 Sensitivity of the Cracking to the Calibration Factors to β_{f1}, β_{f2} and β_{f3}

The effect of β_{f1}, β_{f2} and β_{f3} values on accumulated percent BU and TD fatigue damage is investigated. The analysis was performed by changing one β_f value at a time. First, all β_f values were kept 1.0 by default. Second analysis is performed by only changing β_{f1} (given in Table 10) and maintaining others at their default values "1.0". Third and fourth analyses are performed in the same manner, by changing β_{f2} and β_{f3} and keeping the others at 1.0, respectively. Last analysis was conducted by changing all β_f parameters. This way, the effect of each individual β_f parameter on the accumulated damage over the design life could be evaluated.

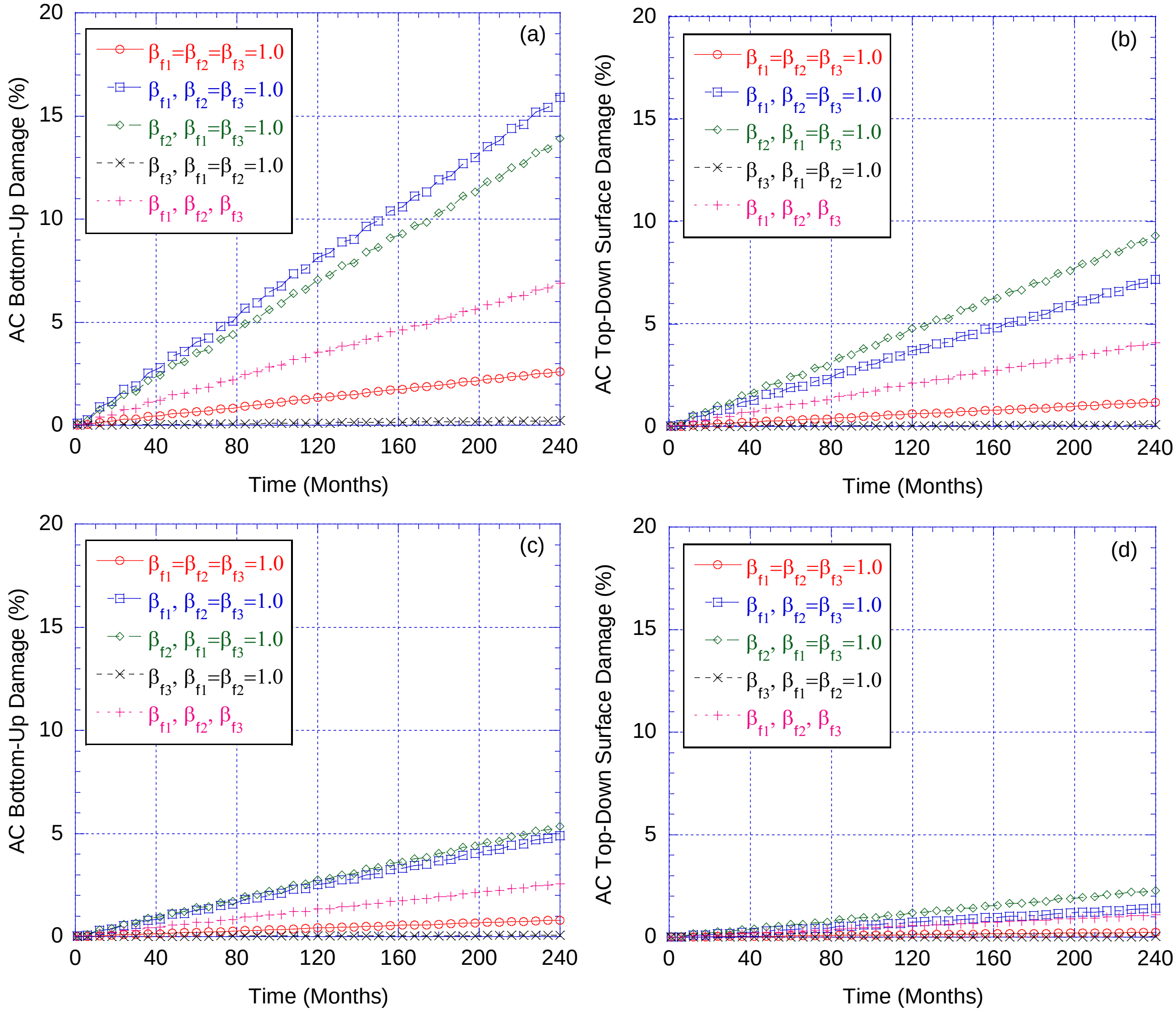


Figure 17 Change in percent (a) Bottom-Up & (b) Top-Down damage in thin and (c) Bottom-Up & (d) Top-Down damage in thick layered AC pavement structures with changing β_f parameters for control mixture

Figure 17 illustrates the change in percent bottom-up and top down damage over time for control mixture with changing β_f parameters for thin and thick layered pavement structure. While the β_{f1} parameter has the highest impact on asphalt concrete bottom-up damage, effect of β_{f2} was higher than other parameters on top-down damage accumulation in thin layered pavement structure. However, the effect of β_{f2} was dominant for both top-down and bottom-up damage accumulation in thick layered structure. Damage accumulation figures of the other mixtures for thin and thick layered structures are provided in the appendix section.

4.4.5 Asphalt Concrete Bottom-Up Cracking Results

The AC bottom-up (BU) cracking is one of the most common distress types that occurs on the flexible pavement over time. The percent BU fatigue cracking of the mixtures analyzed are provided in Figure 18. While the Figure 18(a) shows the results obtained from thin structure, Figure 18(b) illustrates the total BU cracking at the end of design life on thick structure. The analysis was run by targeting 90% distress reliability. According to the obtained results, none of the asphalt mixtures failed under BU cracking for both pavement structure types.

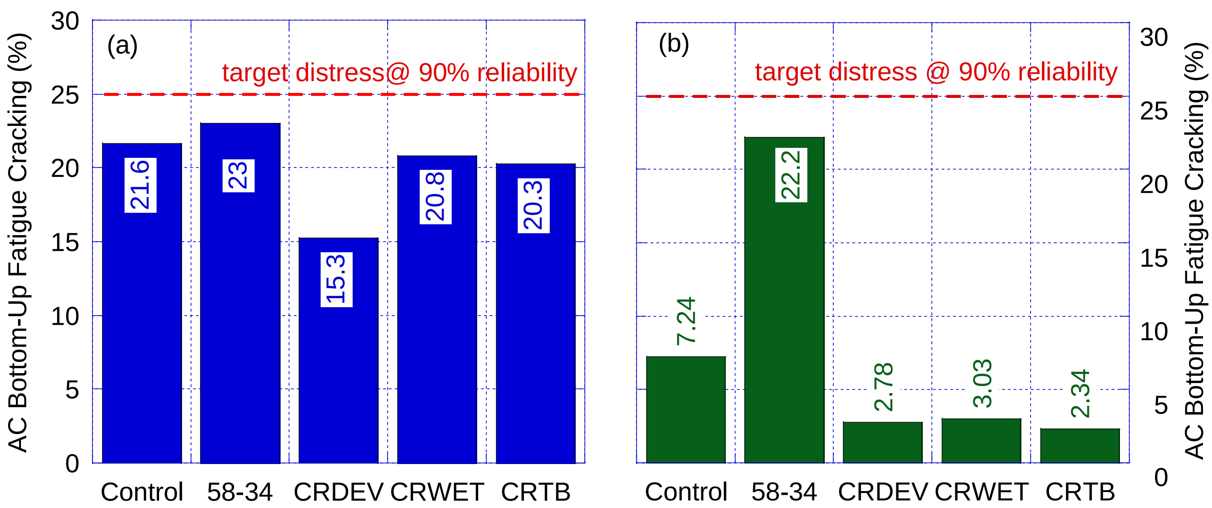


Figure 18 Asphalt mixtures Bottom-Up fatigue cracking for (a) thin layered (b) thick layered pavement structures

Compared to control and soft (PG58-34) mixtures, CR modified asphalt mixtures outperformed on BU fatigue cracking. While the mixture prepared by using devulcanized asphalt rubber had the least amount of percent BU cracking on thin layer pavement structure, CRTB mixture showed the best resistance by accumulating only 2.34% BU cracking in thick layered structure. It is worth noting that this trend is very similar to the one obtained using VECD analysis.

4.4.6 Asphalt Concrete Top-Down Cracking Results

Asphalt concrete Top-Down (TD) or Longitudinal fatigue cracking is another type of pavement distress commonly observed on flexible pavements. The analyses of thin and thick pavement structures are shown in Figure 19.

While the Figure 19(a) illustrates the thin pavement structure analysis results, Figure 19(b) shows the TD cracking of thick AC structure at the end of 20-year design life. According to the 90% distress reliability, thin layered pavement structure with control and 58-34 mixtures failed on TD fatigue cracking at the end of 19.7 and 8 years, respectively, whereas only thick layered 58-34 asphalt concrete accumulated more than the threshold value of 387.8 m/km TD cracking to fail at the end of 14 years.

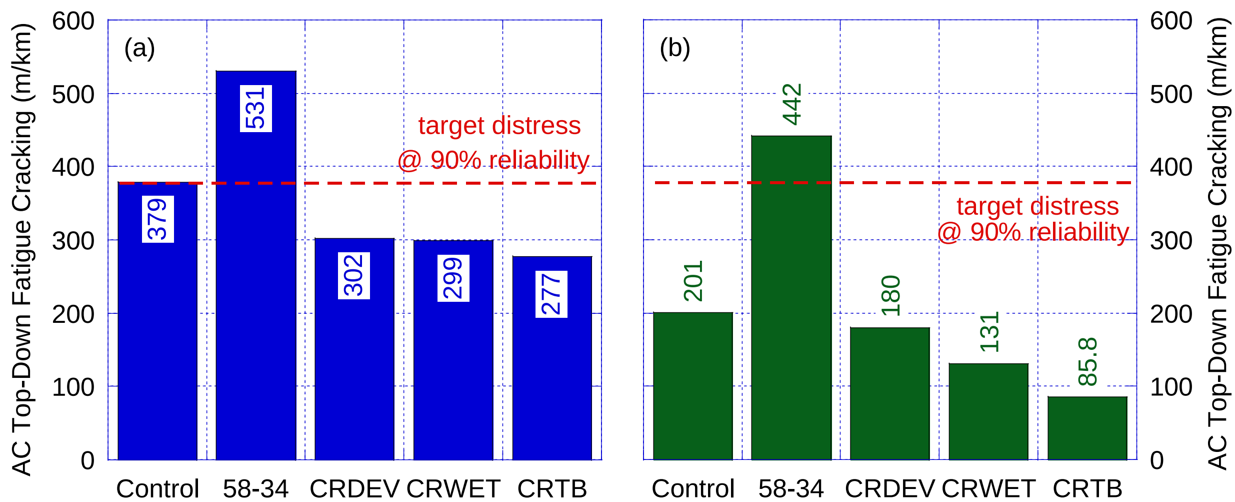


Figure 19 Asphalt mixtures Top-Down fatigue cracking for (a) thin layered (b) thick layered pavement structures

As in the case of bottom-up fatigue cracking, crumb rubber modified asphalt mixtures performed better than control and soft (PG 58-34) mixtures. CRTB mixture ranked the best both on thin and thick layered analyses by accumulating lowest top-down fatigue cracking at the end of design life.

4.4.7 Fatigue Life Comparison of the Mixtures by Using Pavement ME Design Software

The impact of mixture type on the fatigue lives is investigated by using Pavement ME analyses for thin and thick layered pavement structures. While the threshold (T) value of 10% for bottom-up and 200 m/km for top-down fatigue cracking are selected for initiation of the pavement rehabilitation in thin layered pavement structures, these values are adjusted for 2% and 100 m/km for thick pavement structure, respectively. These values are just chosen to provide a comparison between mixtures and can be adjusted by taking into account numerous parameters, such as the availability of funding.

Figure 20 shows the (a) bottom-up and (b) top-down fatigue cracking change over time for thin layered asphalt concrete pavement structure.

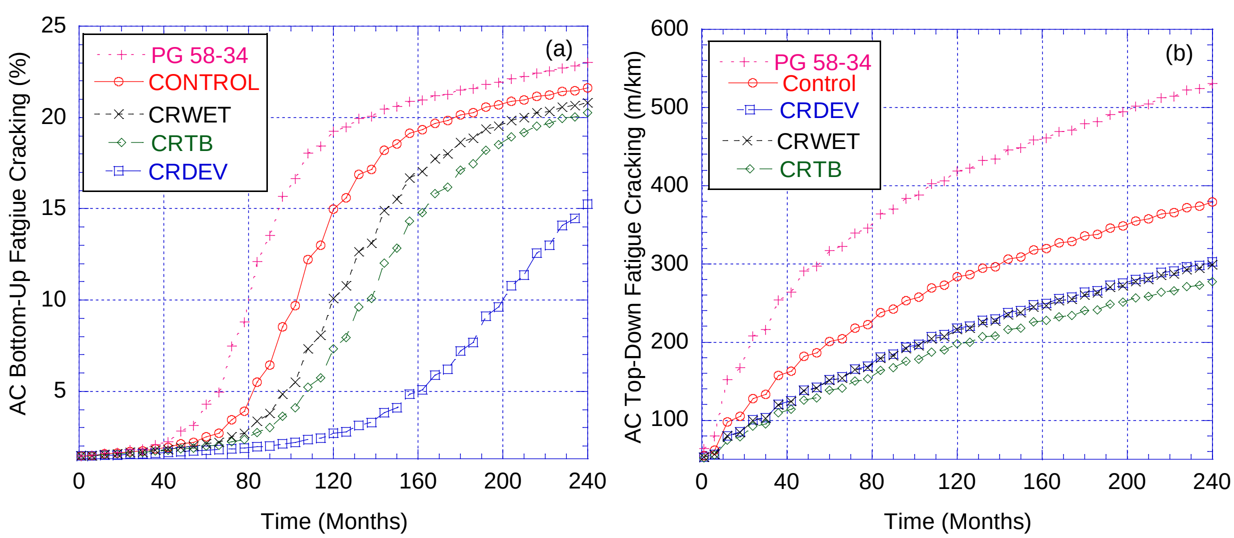


Figure 20 Comparison of mixtures (a) Bottom-Up and (b) Top-Down fatigue cracking over time for thin layered pavement structure

According to the selected T-values, bottom-up and top-down maintenance initiation times in months for a thin layered pavement structure are provided in Table 15.

Table 15 Bottom-up and top-down fatigue lives (in months) of the mixtures for a selected threshold value in a thin layered asphalt pavement structure

	PG 58-34	Control	CRDEV	CRWET	CRTB
Bottom-Up ("T=10%") Fatigue Cracking	80.5	103.2	201	119.4	134.1
Top-Down ("T=200m/km") Fatigue Cracking	22.8	59.7	104.8	106	126.2

While the PG58-34 pavement rehabilitation is suggested to take place after almost two-years for the selected threshold value, CRTB pavements would require the rehabilitation initiation approximately 9 more months later than the PG 58-34 pavements, which is about 5.5 times better top-down fatigue life.

Figure 21 illustrates the change in (a) bottom-up and (b) top-down fatigue cracking over time for a thick layered pavement structure for different mixtures. The specified threshold values for both fatigue types are shown in the graphs as well.

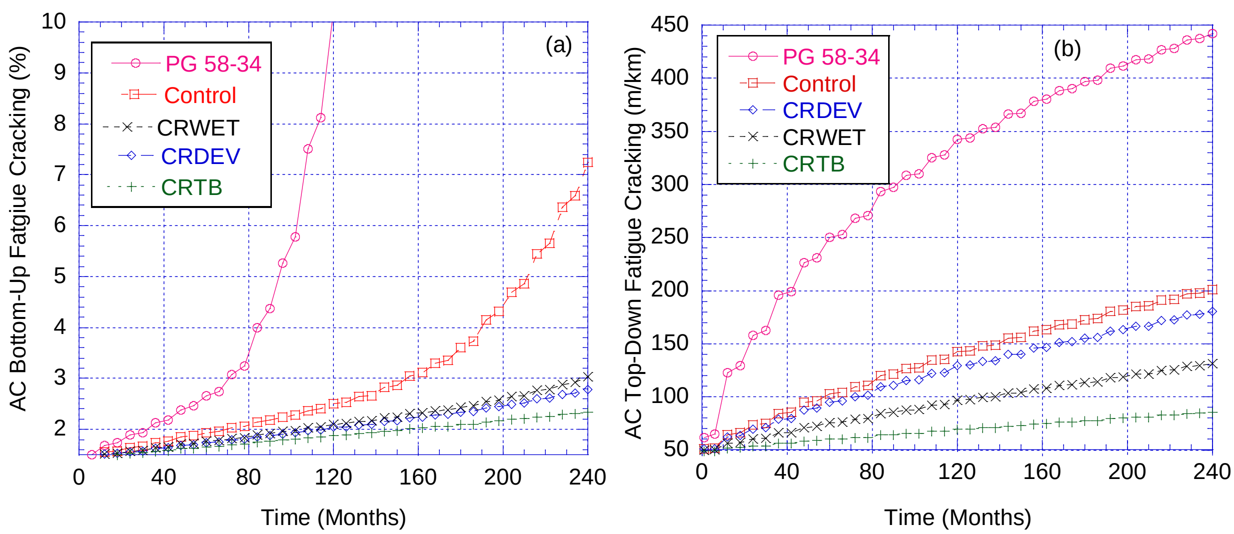


Figure 21 Comparison of mixtures (a) Bottom-Up and (b) Top-Down fatigue cracking over time for thick layered pavement structure

Table 16 Bottom-up and top-down fatigue lives (in months) of the mixtures for a selected threshold value in a thick layered asphalt pavement structure

	PG 58-34	Control	CRDEV	CRWET	CRTB
Bottom-Up ("T=2%") Fatigue Cracking	33.8	70.6	116.6	106	154.8
Top-Down ("T=100m/km") Fatigue Cracking	10.5	58.5	71.5	141	N/A

Pavement maintenance starting times in months for a specified T-value for a thick layer pavement structure are given in Table 16. Rehabilitation beginning times of the mixtures are governed by longitudinal fatigue cracking other than CRWET and CRTB mixtures for which alligator type of fatigue cracking is dominant. The similar ranking of the mixtures observed in thin pavement analysis for fatigue life is also acquired in thick structure analysis. The rehabilitation starting time difference between PG 58-34 and CRTB pavements is more than 20 times. While the PG 58-38 pavement would require maintenance after 10.5 months, CRTB pavements would not reach the specified top-down cracking threshold within the design life. Since the pricing information of the mixtures is subjected to change depending on numerous parameters, the fatigue life differences of the rubber modified, soft and control mixtures can provide a selection criterion for the authorities.

4.5 CONCLUSIONS

This chapter presented a study to investigate the feasibility of substituting crumb rubber modified asphalt mixtures in lieu of grade bumped asphalt binder in mixtures that contain high percentages of RAP. Thermal and fatigue cracking performances of several mixtures prepared with rubberized, soft and base binders were compared. The crumb rubber modification methods and binders included in this study were (i) devulcanized rubber, (ii) crumb rubber terminally blend, (iii) crumb rubber wet process, (iv) soft binder (PG 58-34) and (v) base binder (PG 58-

28). Based on the foregoing, the following conclusions can be made:

- The |E*| tests revealed that at high temperatures/low frequencies, CRTB had the highest stiffness, followed by the Control and CRWET. The PG58-34 and CRDEV mixtures had very similar stiffness and lower than the rest of the mixtures at high temperatures/low frequencies. On the other hand, at low temperatures/high frequencies, CRTB (again) had the highest stiffness, followed by the Control. The CRWET, CRDEV and PG58-34 mixtures all had less stiffness than the CRTB and Control at low temperatures/high frequencies.
- In terms of low temperature cracking susceptibility, mixtures prepared by using CR modified binders provided as good or better performance than the mixtures made with grade bumped PG58-34 binder. This suggests that the CR modified binders may be used in lieu of grade bumping in Michigan.
- Fatigue resistances of rubberized binder mixtures were generally better than the control and soft binder mixtures. Fatigue performance of CRWET is the most favourable among all the mixtures tested.
- AASHTOWare Pavement ME analyses revealed that the CR modified mixtures perform better than the soft and control mixtures both on BU and TD fatigue cracking in thin and thick layered pavement structures.
- Overall, it can be concluded that crumb rubber modified asphalt technologies can be used in lieu of grade-bumped (soft) binders, without negatively affecting the performance. Since this study only included one type of binder and mixture and RAP content, further studies are needed to verify this conclusion in other types of mixtures and binder types.

APPENDIX

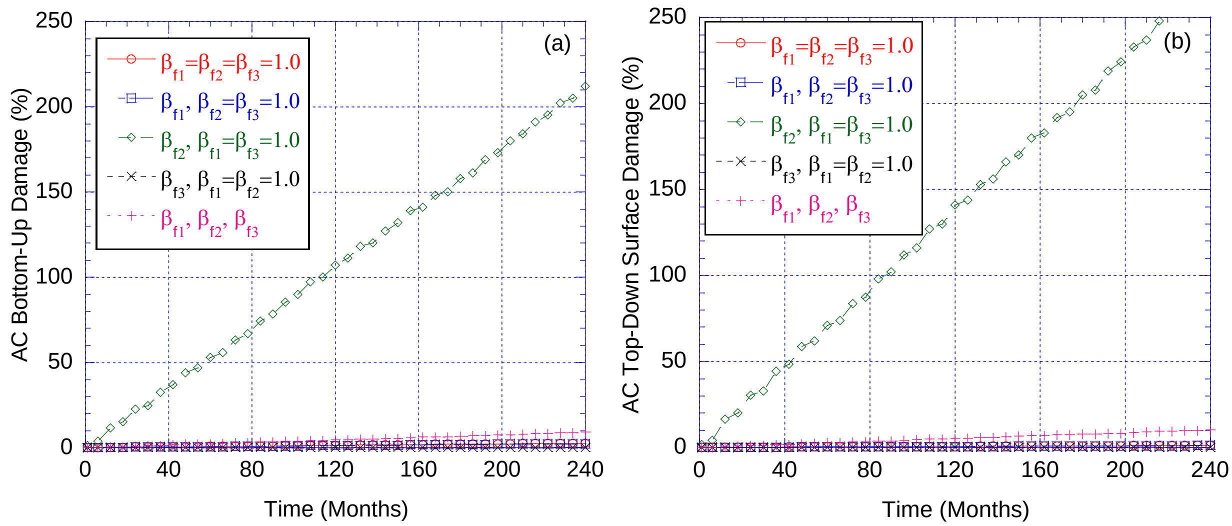


Figure 22 Change in percent (a) Bottom-Up & (b) Top-Down damage with changing β_f parameters in thin layered pavement structure for 58-34 mixture

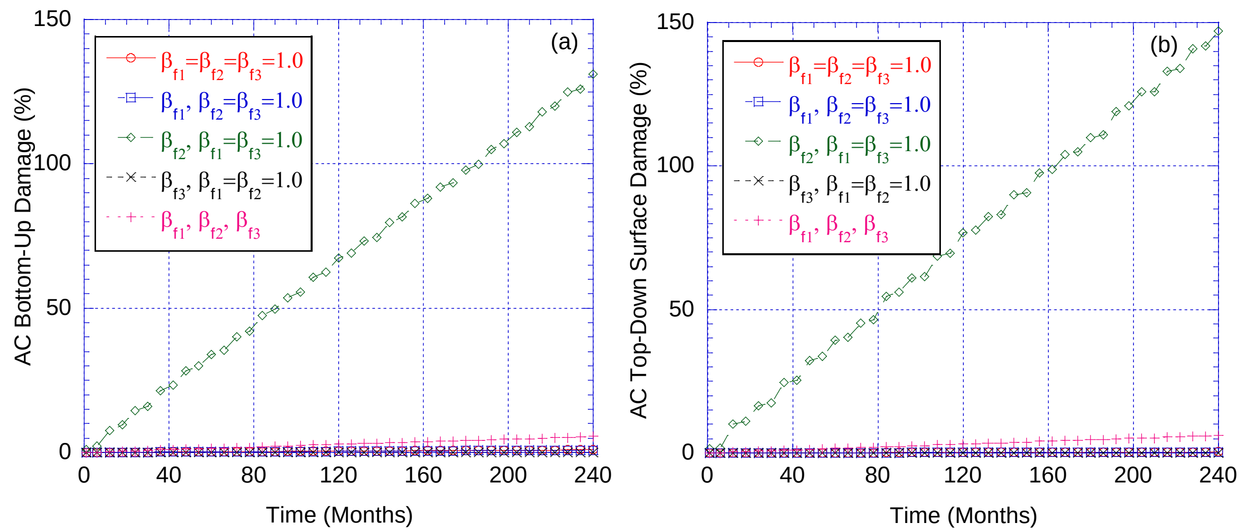


Figure 23 Change in percent (a) Bottom-Up & (b) Top-Down damage with changing β_f parameters in thick layered pavement structure for 58-34 mixture

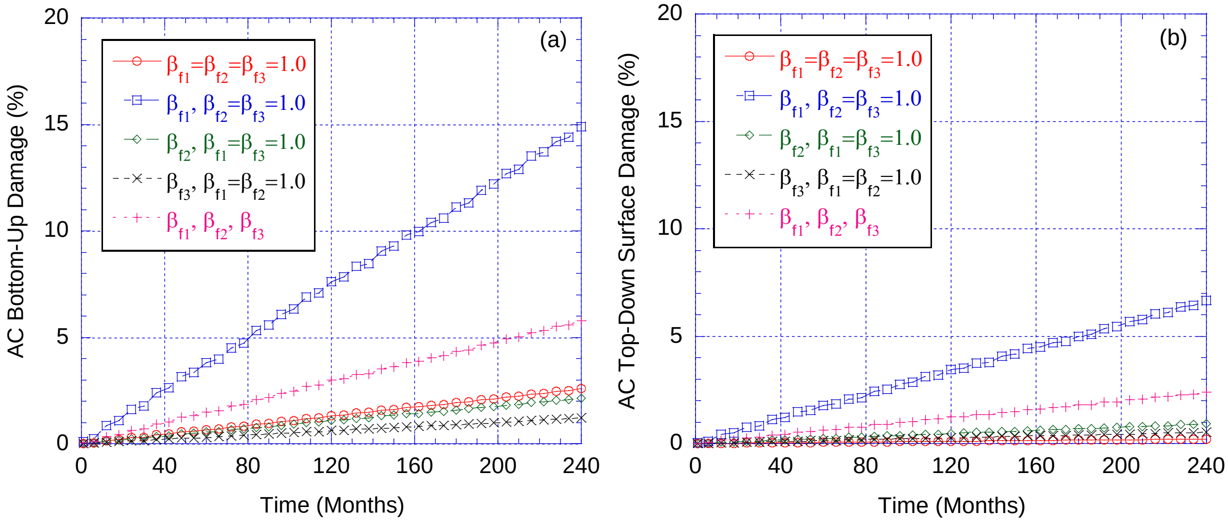


Figure 24 Change in percent (a) Bottom-Up & (b) Top-Down damage with changing β_f parameters in thin layered pavement structure for CRWET Mixture

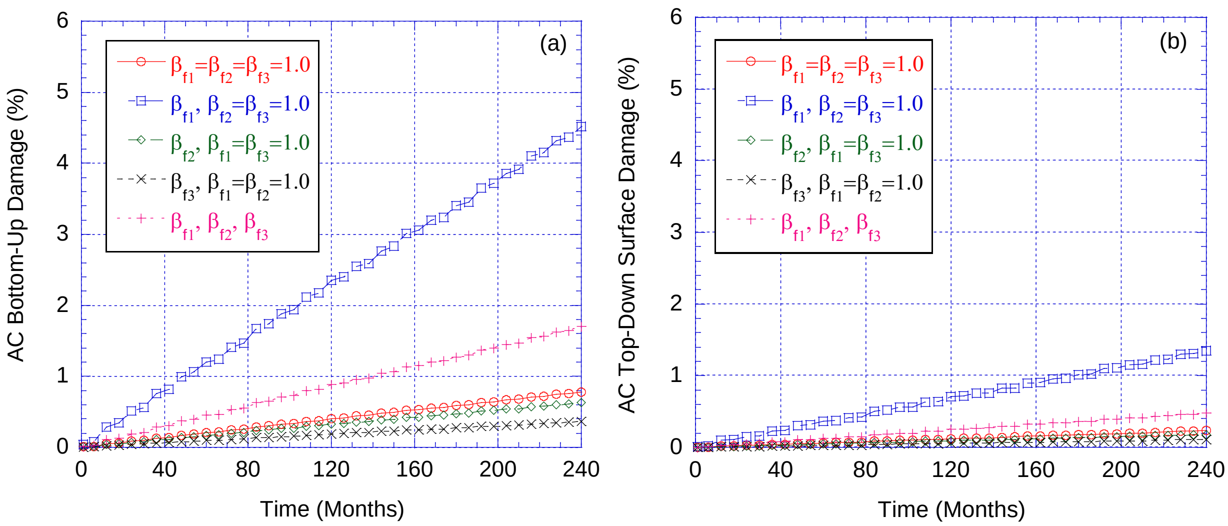


Figure 25 Change in percent (a) Bottom-Up & (b) Top-Down damage with changing β_f parameters in thick layered pavement structure for CRWET Mixture

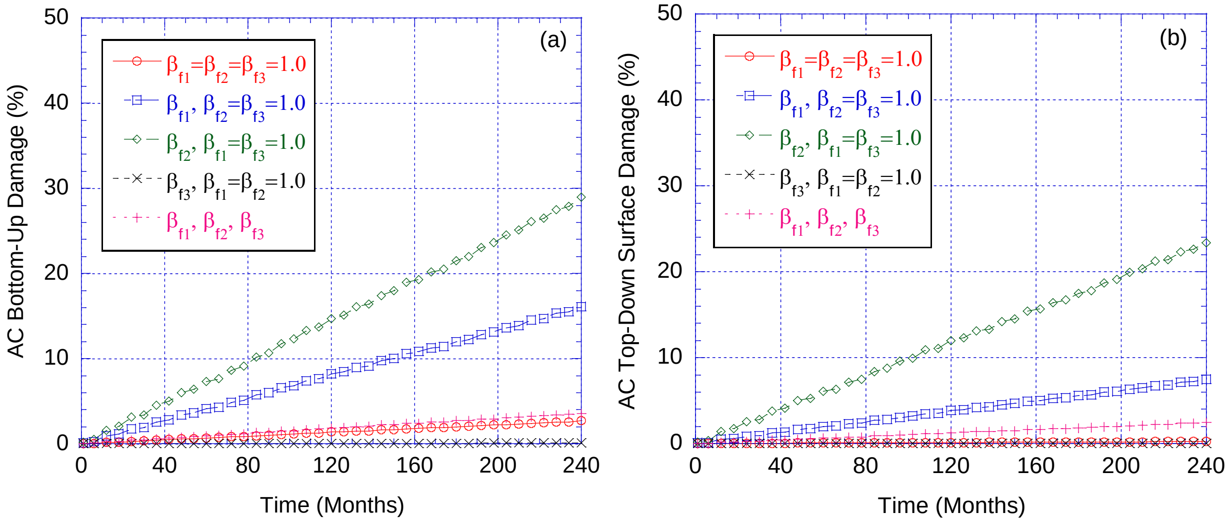


Figure 26 Change in percent (a) Bottom-Up & (b) Top-Down damage with changing β_f parameters in thin layered pavement structure for CRDEV mixture

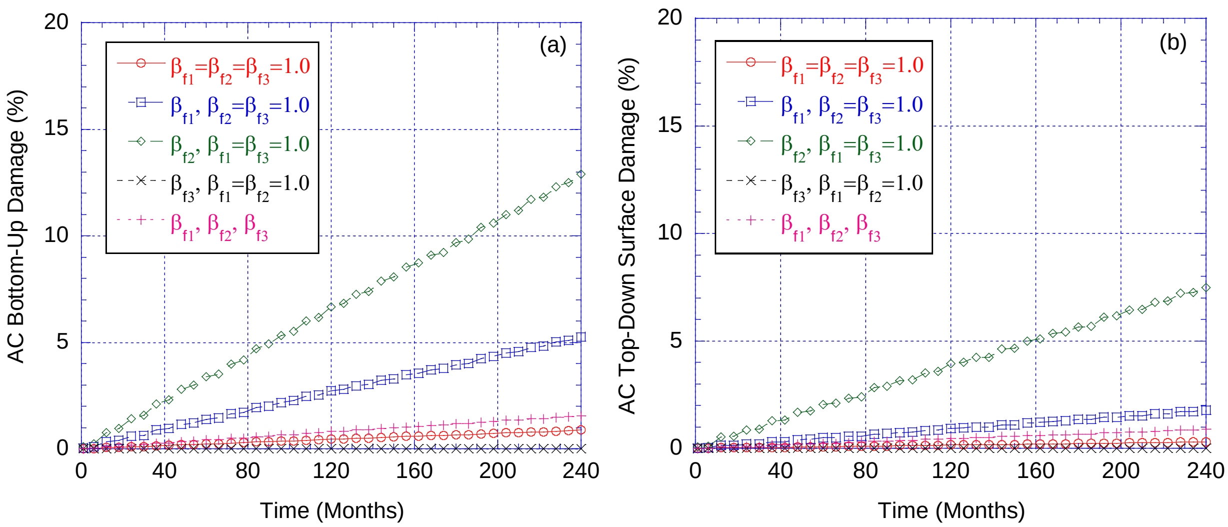


Figure 27 Change in percent (a) Bottom-Up & (b) Top-Down damage with changing β_f parameters in thick layered pavement structure for CRDEV Mixture

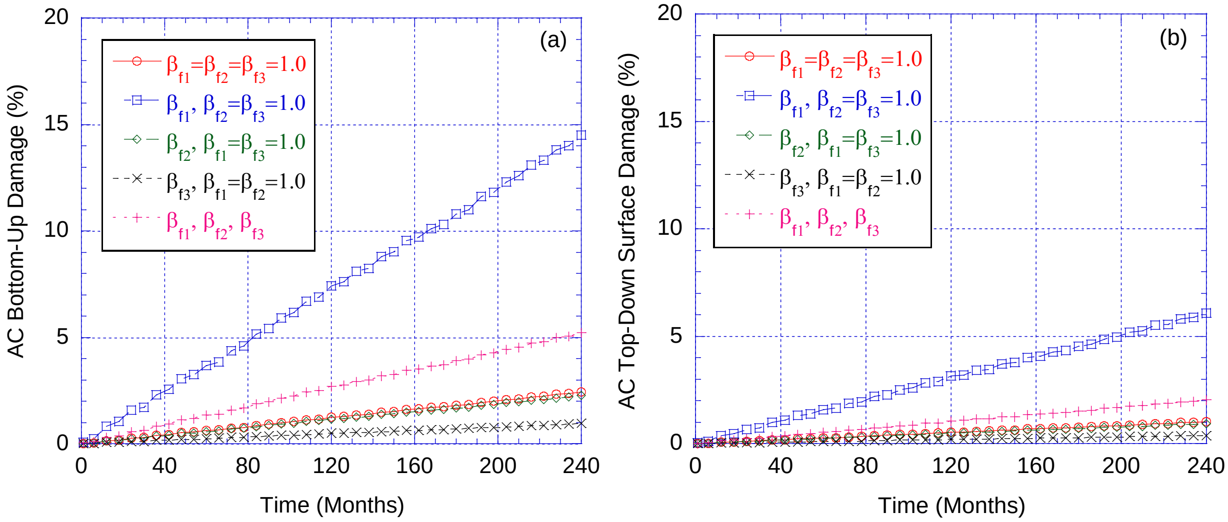


Figure 28 Change in percent (a) Bottom-Up & (b) Top-Down damage with changing β_f parameters in thin layered pavement structure for CRTB mixture

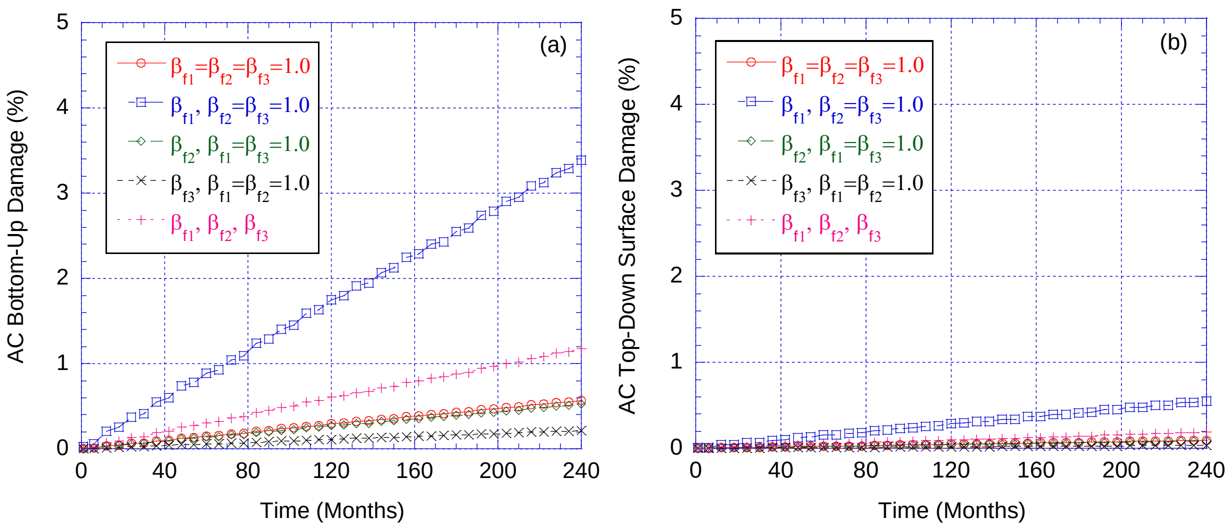


Figure 29 Change in percent (a) Bottom-Up & (b) Top-Down damage with changing β_f parameters in thick layered pavement structure for CRTB Mixture

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CHAPTER 5 COMBINED EFFECT OF SBS AND RECYCLED TIRE RUBBER MODIFICATIONS ON PERFORMANCE GRADE, FATIGUE CRACKING AND RUTTING RESISTANCES

5.1 INTRODUCTION

Asphalt pavements have been experiencing more heavy traffic volume as the population of the world keeps growing in recent years (Nahas et al. 1990). Conversely, the funding shortages for maintenance and higher costs of pavements have been leading to thinner pavements, hence decrease in the service life (Lewandowski 1994). The use of polymer-modified binders has been seen as one of the alternatives to address these problems. Polymer modified binders typically provide more crack- and rut-resistant asphalt mixtures. While they increase the initial cost of the asphalt mixture by 15 to 20% (per ton of mixture), the superior performance of polymer modified binders may lead to a decrease in pavement thickness and in turn to a possible decrease in the overall cost (Caltrans 2005, Rogue et al.2005).

Similar to polymer-modified binders, crumb rubber (CR) modified asphalt binders have been used by many roadway agencies due to their crack-resistant performance (Caltrans 2005). However, the current Performance Grading (PG) system is not approved for use in traditional crumb rubber modified binders (e.g., CRTB – Crumb Rubber Terminally Blend and CRWet – Crumb Rubber Wet processes). This is because the PG system was developed primarily for conventional asphalt binders. Typical crumb rubber modified binders are suspensions, i.e., the rubber particles are visible in the asphalt binder. Therefore, the PG system is still not approved by the AASHTO and alternative and more empirical testing methods are used to specify them. One of the recently re-introduced crumb rubber modified asphalt technologies is the so-called

"De-Vulcanized Rubber (DVR)". The primary advantage of DVR is that, when mixed with asphalt binder, the rubber particles completely dissolve within the binder. The final product, i.e., the DVR modified binder is a complete fluid, not a suspension. As a result, the PG system can easily be applied to specify DVR binder. The unit cost of DVR binder is less than the typical polymers, therefore, if they can be used to partially replace the polymer, more economical and sustainable pavements can be constructed.

5.2 OBJECTIVES AND SCOPE

The main objective of this research was to investigate the relative and combined performances of the SBS polymer and recycled tire rubber (RTR) in an asphalt binder. The scope of this study included modifying one type of binder (Marathon PG58-28) by using two types of polymers (SBS), CR - #20 size and the DVR obtained from a manufacturer located in Ohio, U.S. Figure 30 demonstrates the pictures of the modifiers used in this study.

Polymer modified binders were prepared at three percentages, 1%, 2% and 3% by weight of binder. Although there were various polymer types that have been used to modify asphalt binders, SBS was chosen to conduct this research. Major motives behind selecting SBS could be stated as the cost, ease of implementation, State DOT acceptance, phase stability, and industry and research experience.

The SBS polymers were linear (Kraton D1101AT) and sequential (LCY 3710) block copolymer based on styrene and butadiene with bound styrene 31% and 30% by mass, respectively. The content of vinyl component in SBS polymers were chosen as 10% and 30% in order to evaluate the effect of this ingredient on the modification. DVR and CR modified binders were prepared at three percentages 3%, 6% and 9% by weight of binder. The reason for preparing the RTR modified binders at 3 times higher percentages than the SBS modified binders is the preliminary research indicating that 3% DVR increased the stiffness of binder at high temperatures approximately the same amount as the 1% D1101AT did. Moreover, 3% CR modification increased the high temperature stiffness almost the same amount as 1% LCY-3710. It is further noted that the unit cost of DVR is approximately one third of the SBS cost in today's current market.

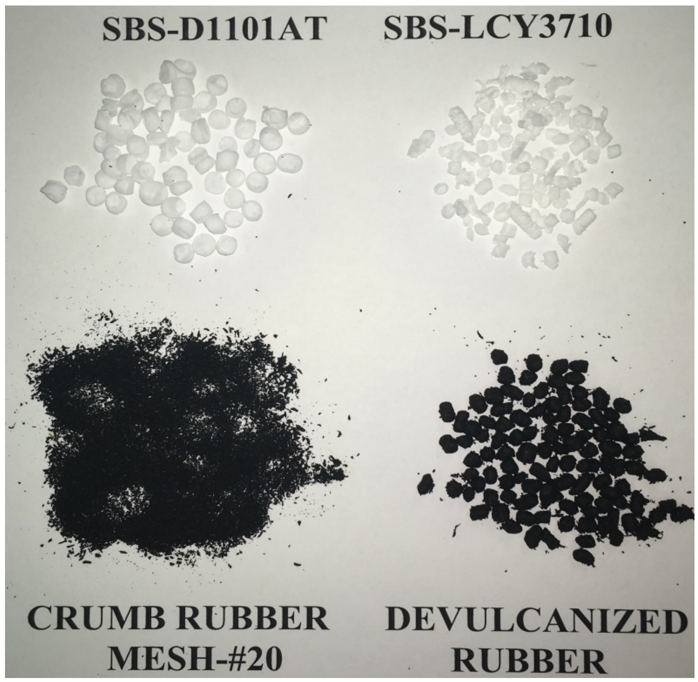

Figure 30 Styrene-butadiene-styrene and recycled tire rubber modifiers

In addition to individual SBS and RTR modifications, SBS+DVR and SBS+CR modified binders were prepared and tested at the following combinations: 1%SBS+3%DVR (or CR), 1%SBS+6%DVR (or CR), 2%SBS+3%DVR (or CR), and 2%SBS+6%DVR (or CR). All combinations are shown in Table 17. The performance tests included determination of continuous performance grades (PGs) (at high, intermediate and low temperatures), fatigue resistance using the Linear Amplitude Sweep (LAS) test and rutting potential and presence of elastomeric modifier using Multiple Stress Creep Recovery (MSCR) test.

Table 17 Binder modification matrix

Modification Matrix			DVR				CR		
			0%	3%	6%	9%	3%	6%	9%
PHASE 1	SBS1- (D1101AT)	0%	✓	✓	✓	✓	✓	✓	✓
		1%	✓	✓	✓	✗	✓	✓	✗
		2%	✓	✓	✓	✗	✓	✓	✗
		3%	✓	✗	✗	✗	✗	✗	✗
PHASE 2	SBS2- (LCY3710)	1%	✓	✓	✗	✗	✓	✗	✗
		2%	✓	✗	✓	✗	✗	✓	✗
		3%	✓	✗	✗	✗	✗	✗	✗

5.3 SAMPLE PREPARATION AND TESTING METHODOLOGY

Preparation of the asphalt binders for testing is one of the most important phases to obtain repeatable results. Moreover, the reproducibility of the modified asphalt binders in mass amounts for large scale projects in binder plants has to be taken into account. To achieve this objective, research team collaborated with the leading asphalt binder manufacturers/modifiers and their experiences and practices were utilized to modify the asphalt binder in Michigan State University's Advanced Asphalt Characterization Laboratories (MSU-AACL). Testing of the modified asphalt binders was performed according to American Association of State Highway and Transportation Officials (AASHTO) standards and specifications.

5.3.1 Binder Modification

As mentioned earlier, three asphalt modifiers were used in this research: (i) styrene-butadiene-styrene (SBS) linear block copolymer, (ii) devulcanized rubber (DVR) and (iii) crumb rubber (CR). DVR and CR were manufactured from recycled tire rubber (RTR). Both individual and combined modifications were achieved to investigate the discrete and combined effects of SBS and/or RTR on the asphalt binders. In order to ensure the repeatability of the binder modification, at least two binder batches were prepared with the same combinations. PG and binder performance tests were conducted at least two replicates from each binder batch. Minimal variability of the test results indicated successful achievement of binder modification and testing.

5.3.1.1 Styrene-Butadiene-Styrene (SBS) modification of asphalt binders

SBS modification is performed by using high and low shear mixers successively. First step during SBS modification of the asphalt binders is to heat up the neat binder to 163^{o}C. Having obtained hot liquid asphalt binder, SBS pellets are milled into the binder by using a rotor-stator at 5000 revolutions per minutes (rpm). Fine dispersion and size reduction of the SBS pellets are achieved by using a slotted stator head during high shear mixing. High shear mixing or milling process takes approximately 30 minutes and during this process there is no external heat provided to the system. Temperature of the system is maintained almost constant because of the fact that internal heat is produced during the size reduction of the SBS pellets. Following the milling process, the SBS-binder mixture is transferred to a low shear mixer where it is further mixed for 120 minutes at 180^{o}C. Temperature is maintained constant during low shear mixing with the aid of either circulatory heating oil bath or adjustable heating mantle. The rotational speed is kept at 1000 rpm. At the end of 90-minute low shear mixing, liquid Sulphur is introduced to the binder mixture as a cross-linking (XL) agent at a weight ratio of 20:1 (SBS:XL). Subsequently, binder mixture is moved to a pre-heated oven at 163^{o}C for another 16 hours in oxygen free environment. This process is called static aging and required to complete the reaction between asphalt binder, polymer and XL agent.

5.3.1.2 Devulcanized rubber (DVR) modification of asphalt binders

DVR modification of asphalt binders follows the same process described as in polymer modification with a slight change. Since DVR comes in pellets as well, high and low shear mixing processes follow the same routine. The only difference between SBS and DVR modifications is the addition of XL agent. In DVR, the weight ratio of DVR to XL is 40 to 1. This ratio is obtained to minimize the use of XL agent without affecting the PG of the modified binder. Combined modifications of SBS and DVR are performed by pre-mixing SBS and DVR pellets prior to milling process in a high shear mixer. The rest of the modification process follows the same procedure explained in the "SBS modification of asphalt binders" section. Figure 31(a) and Figure 31(b) show the high shear and low shear mixers used in this study,

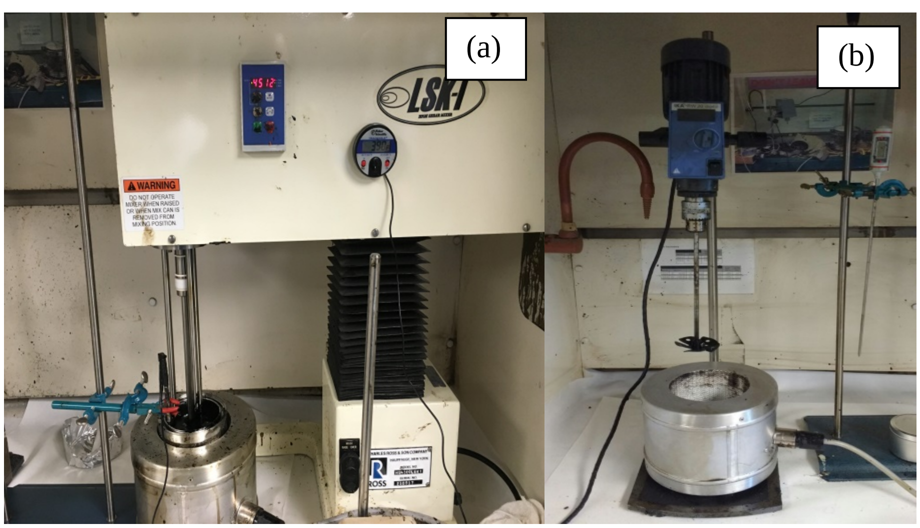

Figure 31 Pictures of (a) high-shear mixer and (b) low shear mixture

5.3.1.3 Crumb rubber (CR) modification of asphalt binders

Binder modification by using CR requires only low shear mixing. CR particles are added to pre-heated binder at 180^{o}C and mixed for 60 minutes at 1000 rpm. The temperature of the system is maintained by using adjustable heating mantle or hot plate. It should be kept in mind that there has to be enough room in the mixing container to accommodate the swollen CR particles. Modifications with combined SBS and CR were performed in this research as well. To produce SBS+CR modified binders, first SBS milling in high shear mixer is performed for 30 minutes. Subsequently, low shear mixing is conducted at 180^{o}C for 2 hours. During the last 60 minutes of low shear mixing, CR particles are introduced into the binder mixture over 5 minutes. As the last step, XL agent is added after 1.5 hours of low shear mixing before transferring the binder mix into oven for static conditioning.

5.3.2 Performance Grade Testing Methods of Modified Asphalt Binders

Superpave performance grading (PG) is represented by two numbers. While the first number is the average seven-day maximum pavement temperature, the second number reports the minimum pavement temperature likely to be expected in degrees Celsius, which is a negative value. The first set of tests for modified binders includes determining the performance grade (PG). PG tests involve short and long term aging of the binders, dynamic shear rheometer tests both on original and aged binders and bending beam rheometer test on long term aged binders.

5.3.2.1 Rolling thin film oven (RTFO) & pressurized aging vessel (PAV) aging tests

Modified binders need to be aged to simulate the field conditions for short and long term.

Since the asphalt binder is obtained from crude oil, it undergoes to stiffening (aging) over time. Although there are various factors contribute to the oxidation, loss of volatiles and oxidation are the ones experienced during RTFO and PAV aging. In broad terms, aging can simply be explained as the formation of carbonyl compounds over time as the binder reacts with the oxygen atoms in the surrounding environment. Since aging requires the presence of oxygen atoms, it is also called as oxidation and which is a function of mainly temperature and pressure.

RTFO aging is performed on unaged binders. It uses elevated temperature (163^{o}C) and air flow (4000 ml/min) to quantitatively measure the loss of volatiles present in the asphalt binder. It is used to simulate the short term aging that happens during mixing and placement of asphalt mixtures. Test takes around 85 minutes and carrousel rotates at 15 rpm. RTFO aging is performed according to AASHTO T240-13. Figure 32 (a) illustrates the picture of RTFO used throughout this study to short-term age the asphalt binders.

PAV aging is performed on RTFO aged binders. This test takes place 20 hours in a heated vessel pressurized to 2.10 MPa (305 psi). All PAV aging in this work was performed at 100^{o}C for consistency purposes. PAV simulates long term aging that happens during in-service life of the asphalt binder. Therefore, any tests required to predict in-service condition of the asphalt binder have to be conducted on PAV aged binders. PAV aging is performed according to AASHTO R28-12. Picture of PAV is shown in Figure 32 (b).

Although the use of degassing oven to remove the induced air bubbles inside PAV aged binders is still under debate, degassing was performed in all PAV aged binders prior to the preparation of testing samples (Anderson et al. 2014).

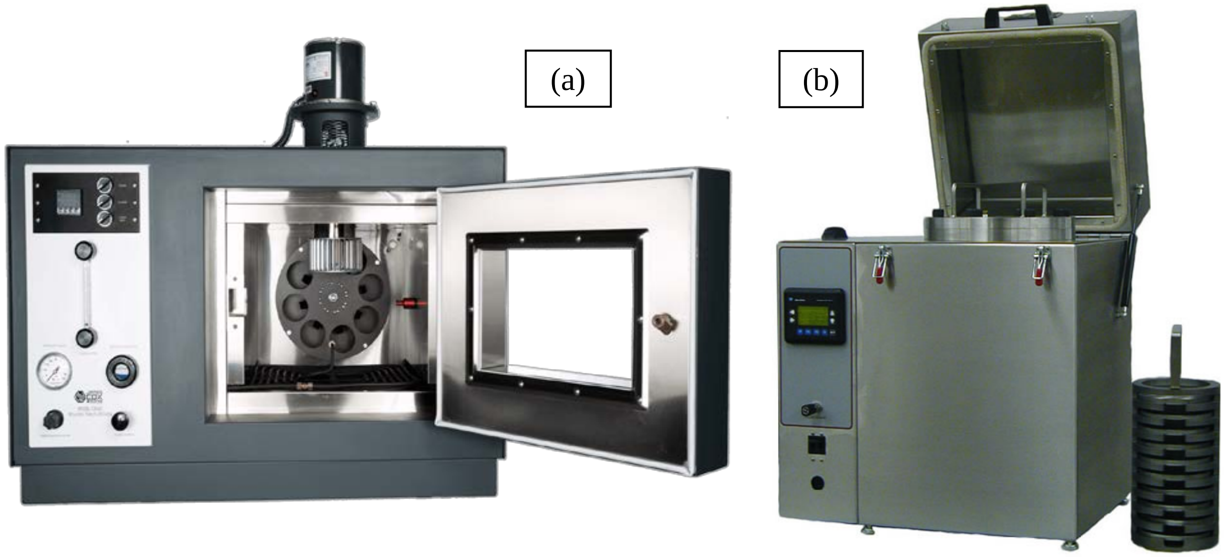

Figure 32 Pictures of (a) rolling thing film oven and (b) pressurized aging vessel

5.3.2.2 Dynamic shear rheometer (DSR) tests on original binder

DSR test on original (a.k.a, unaged, neat or virgin) binder makes up the first test on a series of tests to determine PG of the binder. A 25-mm diameter parallel plate geometry with 1000 micron gap is used at high temperatures (>35^{o}C) to find out the high PG of original binder. It measures the complex shear modulus (|G*|) and phase shift angle (δ) (a.k.a, phase angle) in degrees at discrete temperatures. The loading temperature of the sample to provide enough adhesion between binder sample and the plates is chosen as 58^{o}C. It is also taken as the first testing temperature. Testing continues until failure with increasing 6^{o}C intervals. Failure herein means that $|G^*|/\sin(\delta) < 1.0$ kPa, where sin (δ) is sine of phase shift angle. DSR setup used in this research is Anton Paar MCS 302 hardware with RheoCompass2 software. Figure 33 demonstrates the picture of DSR used in this research. High PG testing of original binder is performed according to AASHTO T315-12 standard.

Figure 33 Picture of dynamic shear rheometer

5.3.2.3 DSR tests on RTFO aged binder

DSR test on RTFO aged binder is the second criterion to establish high PG of the binder. The same steps as in original binder testing are followed in RTFO aged binder testing as well. The only difference between original binder and RTFO aged binder is the failure criterion. The failure in RTFO aged binder occurs at a temperature where $|G^*|/\sin(\delta) < 2.20$ kPa. The high PG obtained from RTFO aged DSR test is the highest passing temperature before the failure happens. High PG of the binder is the lower of the highest passing temperatures obtained from original binder and RTFO aged binder DSR test results.

5.3.2.4 DSR tests on PAV aged binder

DSR is utilized to determine the intermediate PG of the binder as well. Although it is not recorded on PG denotation, it is an important test to understand the fatigue cracking behavior of the asphalt binder. Testing is performed on 8 mm diameter parallel plate geometry with 2000 micron gap.

Since the intermediate PG testing temperatures are generally lower than the room temperatures, the sample is loaded to the plates at 58^{o}C to ensure the enough adhesion between the testing plates and the asphalt cement. Once the sample is loaded, the testing temperature (which is obtained from the table at AASHTO M320-10 according to the high PG of the binder) is entered into the script in the software, DSR setup performs the testing according to AASHTO T315. The failure criterion takes place when $|G^*|\times\sin(\delta) > 5000$ kPa. In comparison to high and low performance grades, intermediate PG testing temperatures changes in 3^{o}C intervals. The lowest temperature before the failure temperature is the intermediate PG of the binder.

5.3.2.5 Bending beam rheometer (BBR) tests on PAV aged binder

The last test to fully establish the PG of a binder is BBR testing on PAV aged binder. This test delivers the stiffness and relaxation properties of asphalt binders at low temperatures which are the indicators of asphalt binder's ability to resist to thermal cracking. Hence, it is performed to find out the low PG of the binder. Creep stiffness, (i.e., the reciprocal of creep ($S(t)=1/D(t)$) and the slope of the S(t) versus time curve at 60th second (i.e., m-value) are the outcomes obtained at the end of BBR test.

The failure of the asphalt beam occurs at a testing temperature when creep stiffness "S">300 MPa and/or logarithmic creep rate "m-value" < 0.300 at the end of 60th second. Similar to high performance grade testing, low PG testing temperature is increased or decreased at 6^{o}C intervals. The BBR test was conducted using a Cannon Instrument TE-BBR setup shown in Figure 34 according to AASHTO T313-12.

Figure 34 Picture of thermo-electric bending beam rheometer

5.3.3 Additional Performance Test Methods of Modified Asphalt Binders

Having established the performance grade of modified asphalt binders, second set of tests includes determining the performances of the binders. Binder performance tests, Linear Amplitude Sweep (LAS) and Multiple Stress Creep and Recovery (MSCR), were conducted using dynamic shear rheometer equipment under standardized loading and temperature conditions.

5.3.3.1 Linear amplitude sweep (LAS) test

LAS test is performed to investigate the fatigue life of asphalt binders using the DSR equipment. It is intended to assess the asphalt binder's ability to resist fatigue damage under cyclic loading at increasing amplitudes. LAS test consists of two parts. While first part is the frequency sweep test that determines the damage analysis "alpha" parameter, second part of the test is amplitude sweep which uses oscillatory shear in strain-controlled mode with an increased strain amplitude. It uses 8 mm parallel plate geometry with a 2000 micron gap is used in LAS test. This test can be performed on either RTFO aged or PAV aged binders. Although the LAS testing temperature is somewhat vague and weakest chain of the standard, AASHTO TP101-12 suggests conducting the test at the intermediate pavement temperature obtained from AASHTO M320. In this study LAS test was conducted on PAV aged specimen and at the temperature when $|G^*| \times \sin(\delta) = 5000$ kPa which is the continuous intermediate performance grade of the asphalt binders.

5.3.3.2 Multiple stress creep recovery (MSCR) test

The changing practices of production and modification processes require newer tests to better understand the mechanistic properties of the asphalt binders in addition to the current performance graded asphalt binder specification. PG plus test (such as elastic recovery, ductility, forced ductility, toughness-tenacity and maximum DSR phase angle) have been implemented as supplemental tests to investigate the premium asphalt binders by state agencies and local authorities. However, the requirement of expensive equipment, inefficiency and time consuming process of PG plus tests resulted in the necessity of a substitute testing. As an alternative to PG plus tests, Multiple Stress Creep Recovery (MSCR) test which requires the use of readily available DSR equipment could overcome these problems.

MSCR test has become an AASHTO Standard with designation T350 (formerly AASHTO TP70) in 2014 under the name of “Standard Method of test for Multiple Stress Creep Recovery Test of Asphalt Binder Using a Dynamic Shear Rheometer”. The associated Performance Grade specification was standardized under designation M332 (formerly MP19) in 2014 under the name of “Standard Specification for Performance-Graded Asphalt Binder using Multiple Stress

Creep Recovery Test" as well. The MSCR replaces the current AASHTO M320 DSR test for characterizing the high temperature performance of asphalt binders.

Use of MSCR test will result in a change in binder performance grading system similar to the change was made in 1997 when it was switched from Penetration Grading to Performance Grading. 2016 will be the transition year for most of state DOTs, federal and local agencies to implement PG-MSCR grading.

New system requires testing binders at regional high temperatures, which is 58^{o}C for the State of Michigan, instead of higher temperatures the asphalt pavement would never experience in that specific region. The grade bumping with MSCR test is done according to the traffic level. The binder grade includes a letter next to it is regional high temperature. The letter selection is "S" for standard traffic, "H" for heavy traffic, "V" for very heavy traffic and "E" for extremely heavy traffic. Binder does not contain any polymer in "S" designation.

The "S" designation is used for traffic levels less than 10 million Equivalent Single Axle loads (ESALs) and standard traffic speed greater than 44 mph (>70 km/h). "H" designation is for traffic levels of 10 to 30 million ESALs or traffic speed between 12 to 44 mph (20 to 70 km/h). "V" designation is assigned for traffic levels more than 30 million ESALs or standing traffic speed with less than 12 mph (<20 km/h). The last designation "E" in most situations is used for traffic level with more than 30 million ESALs and standing traffic less than 12 mph (<20 km/h) such as toll plazas and port facilities (AASHTO MP19 2010 and M332 2014, MN-DOT 2015).

MSCR test is used to estimate the percent recovered and unrecovered strains of asphalt binders. Elastic response of an asphalt binder was conducted under shear creep and recovery at 0.1kPa and 3.2kPa stress levels. A 1-second shear creep part is followed by 9-second recovery portion for total of ten cycles at each stress level. Test is performed on short term aged asphalt binders. MSCR test is conducted by using DSR with 25 mm parallel plate geometry and 1000 micron gap setting as described in AASHTO TP70-13 both at high PG temperature of the asphalt binder according to AASHTO MP19-10 and regional temperature according to AASHTO M332.

The traffic selection is performed according to non-recoverable creep compliance "J_{nr}" at 3.2kPa shear stress. $J_{nr3.2}$ requirement for traffic levels should be as follows:

- Extremely Heavy Traffic "E-grade" is achieved when;

  Jnr at 3.2kPa shear stress < 0.5 kPa^{-1}

- Very Heavy Traffic "V-grade" is achieved when;

  0.5 kPa^{-1} < Jnr at 3.2kPa shear stress < 1.0 kPa^{-1}

- Heavy Traffic "H-grade" is achieved when;

  1.0 kPa^{-1} < Jnr at 3.2kPa shear stress < 2.0 kPa^{-1}

- Standard Traffic "S-grade" is achieved when;

  2.0 kPa^{-1} < Jnr at 3.2kPa shear stress < 4.5 kPa^{-1}

Moreover, the stress sensitivity in MSCR test is limited. The percent difference between non-recoverable creep compliance obtained from stress levels 0.1kPa and 3.2kPa should not be greater than 75%. This insures that the binder is not overreacting to unexpected heavy loads and high temperatures.

Although AASHTO MP19-10 states that the specification is not valid for asphalt binders which have discrete particles larger than 250 micrometers in size, MSCR test has been performed on CR modified asphalt binder to provide relative information with respect to DVR and SBS modifications. Moreover, MSCR test by using concentric cylinder (CC) geometry was performed on the same CR and SBS+CR modified asphalt binders since the CC (a.k.a bob and cup) testing geometry was proposed to measure the PG of CR modified binders more accurately. CC geometry was run by using Anton Paar MSC 302 dynamic shear rheometer (Figure 33) with B-CC17SP-25 concentric cylinder setup (Figure 35b). Since the common practice in today's market is to produce rubberized asphalt binders around 20% CR modifiers, only in this part of the study 20% CR wet process asphalt binder was prepared and tested. Figure 35 illustrates the parallel plate and concentric cylinder setup just after the MSCR tests were performed.

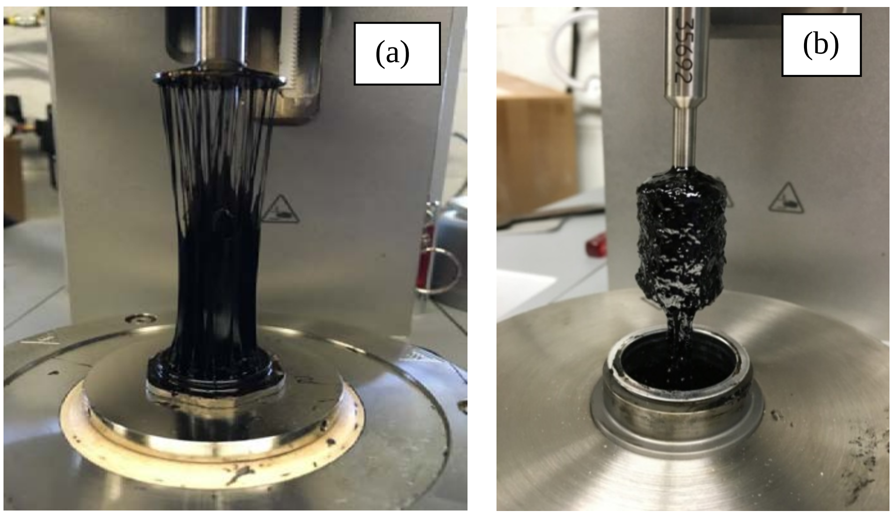

Figure 35 (a) Parallel plate and (b) concentric cylinder setups

5.4 TEST RESULTS AND DISCUSSIONS

5.4.1 Continuous Performance Grade Results

Use of continuous grading temperatures is an important part of forensic and research studies. It is mainly used to evaluate the blending, producing and modifying the asphalt binders. Although there have been various procedures used by researchers such as non-linear, parabolic, exponential curve fitting, extrapolation etc., this study adopted linear interpolation between absolute highest passing and lowest failure temperatures for all three continuous PGs. The method used in this research to discover the continuous grading temperatures and continuous grades is in accordance with ASTM D7643-10 “Standard Practice for Determining the Continuous Grading Temperatures and Continuous Grades for PG Graded Asphalt Binders”. In order to apply ASTM D7643-10, all other Superpave PG grading standard specifications procedures were followed to determine high, intermediate and low PG of the binders. According to ASTM D7643-10, continuous grade is denoted by CHPG-CLPG(CIPG) where CHPG, CLPG and CIPG are continuous high, low and intermediate performance grades (e.g. 73.6-29.7(16.2)).

Linear interpolation method used in this study was two point linear relation between test results ($\log_{10}$ scale) and the test temperature (arithmetic scale) other than m-value which is interpolated by using arithmetic scale only. Standards state how to perform the interpolation to find the continuous grades in equation 1 and equation 2. While equation 1 applies to all test results, equation 2 is only valid to find the continuous grade based on m-value.

$$T_c = T_1 + \left(\frac{\log_{10}(P_s) - \log_{10}(P_1)}{\log_{10}(P_2) - \log_{10}(P_1)} \right) \times (T_2 - T_1) \tag{5.1}$$

$$T_c = T_1 + \left(\frac{P_s - P_1}{P_2 - P_1} \right) \times (T_2 - T_1) \tag{5.2}$$

where:

T_c= Continuous grading temperature, ^oC,

T_1= Lower of the two test temperatures, ^oC,

P_s= Specification requirement for property in question; determined at the respective PG grading temperature for the respective property,

P_1= Test result for the specification property in question at T_1,

P_2= Test result for the specification property in question at T_2, and

T_2= Higher of two test temperatures, ^oC.

5.4.1.1 Continuous high PG results

High PG of a binder is determined by using DSR on the original and RTFO aged binders. All binders were tested both in original and short term aged conditions until the failure is reached to discover continuous high PGs. After finding the high PG of original and RTFO-aged binders, continuous high PGs were calculated by using linear interpolation method between highest passing temperature and the lowest failing temperature where the specification requirement was included. Subsequently, the smaller of the value obtained from original and RTFO aged binder tests was assigned as the continuous high PG of the binder. Table 18 summarizes the application continuous high PG construction for SBS D1101AT modified binders. Batch 1 average is the average of minimum two replicates obtained from first modified binder batch. While the upper part of the table shows the test results obtained from original binder testing, lower part includes the test results from RTFO aged binders. Standard deviation (Stdev) and coefficient of variation (COV) of the test results between batch-1 and batch-2 are provided to ensure the repeatability of the modification procedure is successfully achieved. High PG construction tables for other modified and unmodified asphalt binders are provided in the appendix section.

The results of continuous high PG measurements of all modified and unmodified binders are given in Table 19. While the left portion of the table provides the results of modifications performed by using AT SBS, the right part shows the results obtained by using LCY SBS. Both sides have the same values for individual RTR (DVR and CR) modifications. Some of the combined modifications by using LCY elastomeric polymer were not performed since they were not included in the testing matrix of this study. The results of these modifications were marked as “N/A” (not available) in Table 19.

Table 18 Continuous high PG construction of SBS D1101AT modified asphalt binders

	D1101AT Original								
	1%			2%			3%		
	Batch 1 Average	Batch 2 Average	Average	Batch 1 Average	Batch 2 Average	Average	Batch 1 Average	Batch 2 Average	Average
Tc_original (°C)	65.13	65.26	65.19	69.81	70.05	69.93	73.01	73.33	73.17
T1 (°C)	64.00	64.00	**Stdev**	63.99	70.00	**Stdev**	70.00	70.00	**Stdev**
T2 (°C)	70.00	70.00	0.09	69.99	76.00	0.27	76.00	76.00	0.23
P1 (kPa)	1.140	1.160	**COV**	1.910	1.010	**COV**	1.300	1.320	**COV**
P2 (kPa)	0.569	0.571	0.14%	0.980	0.533	0.38%	0.770	0.800	0.31%
Ps (kPa)	1.00	1.00		1.00	1.00		1.00	1.00	
	D1101AT RTFO								
	1%			**2%**			**3%**		
	Batch 1 Average	**Batch 2 Average**	**Average**	**Batch 1 Average**	**Batch 2 Average**	**Average**	**Batch 1 Average**	**Batch 2 Average**	**Average**
Tc_rtfo (°C)	65.96	66.18	66.07	71.04	70.54	70.79	73.71	74.62	74.17
T1 (°C)	64.01	64.01	**Stdev**	70.00	70.00	**Stdev**	70.00	70.00	**Stdev**
T2 (°C)	69.98	69.99	0.16	76.00	76.01	0.35	76.00	76.00	0.65
P1 (kPa)	2.78	2.86	**COV**	2.46	2.33	**COV**	3.13	3.3	**COV**
P2 (kPa)	1.36	1.39	0.24%	1.29	1.23	0.50%	1.77	1.95	0.87%
Ps (kPa)	2.20	2.20		2.20	2.20		2.20	2.20	
Tc (°C)	65.19			69.93			73.17		

Stdev= standard deviation, COV= coefficient of variation, Tc= continuous high PG, Tc_original and Tc_rtfo= original and rtfo aged continuous high PGs

Table 19 Continuous high PG test results

Continuous High Performance Grade							
Modification with SBS-D1101AT	Con.High PG ^{O}C	Original Based PG ^{O}C	RTFO Based PG ^{O}C	RTFO Based PG ^{O}C	Original Based PG ^{O}C	Con.High PG ^{O}C	Modification with SBS-LCY3710
Unmodified	60.1	60.1	61.2	61.2	60.1	60.1	Unmodified
Aged*	61.9	61.9	62.5	62.5	61.9	61.9	Aged*
1% SBS	65.2	65.2	66.1	64.7	64.0	64.0	1% SBS
2% SBS	69.9	69.9	70.8	67.0	67.1	67.0	2% SBS
3% SBS	73.2	73.2	74.2	72.2	71.9	71.9	3% SBS
3% DVR	66.2	66.2	67.0	67.0	66.2	66.2	3% DVR
6% DVR	69.0	69.0	69.9	69.9	69.0	69.0	6% DVR
9% DVR	72.3	72.3	72.3	72.3	72.3	72.3	9% DVR
1% SBS +3% DVR	66.6	66.6	67.9	66.6	65.9	65.9	1% SBS +3% DVR
1% SBS +6% DVR	68.0	68.0	69.1	N/A	N/A	N/A	1% SBS +6% DVR
2% SBS +3% DVR	69.3	69.4	69.3	N/A	N/A	N/A	2% SBS +3% DVR
2% SBS +6% DVR	71.3	71.4	71.3	71.7	70.8	70.8	2% SBS +6% DVR
3% CR	64.0	64.0	64.9	64.9	64.0	64.0	3% CR
6% CR	68.6	68.6	69.3	69.3	68.6	68.6	6% CR
9% CR	72.3	72.3	74.3	74.3	72.3	72.3	9% CR
1% SBS +3% CR	69.9	69.9	70.3	67.4	66.4	66.4	1% SBS + 3% CR
1% SBS +6% CR	73.7	73.9	73.7	N/A	N/A	N/A	1% SBS + 6% CR
2% SBS +3% CR	70.3	70.5	70.3	N/A	N/A	N/A	2% SBS +3% CR
2% SBS + 6% CR	74.7	74.8	74.7	73.0	74.3	73.0	2% SBS +6% CR

Con.High PG: continuous high PG, DVR: devulcanized rubber, CR: crumb rubber -#20 mesh size, SBS; styrene-butadiene-styrene

Results revealed that the high PG of the binders was mainly governed by original DSR measurement in SBS, DVR and SBS+DVR modifications. This phenomenon remained true for CR modifications as well other than 6% CR modification. However, as the amount of modifier increased in combined SBS+CR modifications, RTFO-aged binders became dominant to dictate the high PGs.

It should be noted that sample denoted with Aged* underwent high and low shear mixing cycles without addition of any modifier. The aim of preparing Aged* binder was to investigate the impact of heating and mixing during modification process on the Performance Grade (valid for intermediate and low PGs as well). Unmodified binder in Table 19 was the base binder used for all modifications and it had an as-received PG of 58-28. The same base binder was used for all modifications to minimize the source effect of crude oil.

The impact of aging during the modification was negligible. It affected the continuous high PG as much as short term aging affected. Hence, continuous high PG of the original Aged* binder (61.9^{o}C) is slightly higher than that of RTFO-aged Unmodified binder (61.2^{o}C).

The first phase of high PG determination included the modification and testing of the binder with Kraton® SBS-D1101AT (here in referred as D1101AT). Individual D1101AT, CR, DVR and combined D1101AT+DVR and D1101AT+CR modifications were performed within this phase.

Second phase included the effect of SBS polymer type on the continuous PG. For this purpose, chemical compositions of various SBS polymers were investigated. LCY Elastomers® 3710 (herein called as LCY3710) linear sequential was selected and used as a second SBS modifier to study this. The reason for selecting the LCY3710 as the second modifier was its high vinyl content and very similar styrene and butadiene co-block amounts. While D1101AT had

10% vinyl content, the amount in LCY3710 was 30%. Moreover, the butadiene and styrene contents were 70% to 69% butadiene and 30% to 31% styrene by weight, respectively.

Figure 36 illustrates the effect of modifications on continuous high PG for SBS and RTR modifications. It is clear that each modification improved the continuous high PG.

In Figure 36(a), it can be seen that polymer D1101AT improves the high PG better than LCY 3710. Binders modified with 1% LCY3710 barely achieved one grade high PG bump. It might be inferred that as the vinyl content increases, binder modification capacity of SBS polymer may decrease. This phenomenon needs to be further investigated.

Both D1101AT and LCY3710 modifications achieved one grade bump on high PG at 1%. While 2% D1101AT modification increased the high PG by about 8 degrees and binder PG changed from PG58-28 to PG64-28 (but very close to PG70-28, 69.8^{o}C), the temperature increase was around 5^{o}C for 2% LCY3710. 3% SBS modification for both polymer types achieved two grade bumps from 58-XX to 70-YY.

Figure 36(b) demonstrates that DVR modification yielded better results than CR modification at both 3% and 6% RTR modifications, achieving one grade bump. Moreover, at 9% RTR modifications, DVR modification was still better than CR modification by 0.5^{o}C and both modifications achieved two grade bumps.

When Figure 36(a) and Figure 36(b) were compared, DVR modification showed a very similar behavior as D1101AT. It appears that, in general, every 3% DVR created the similar effect of 1% SBS. It is noted that the current price of DVR is about 1/3rd of the SBS polymer, therefore the price of 3% DVR is about the same as 1% SBS. This phenomenon can clearly be seen in Figure 37 which demonstrates high PG achievement of SBSs, DVR and CR modifications.

Figure 37(a) demonstrates the impact of SBS modification on high PG of the modified binders. Both D1101AT (herein after referred to as AT) and LCY3710 (herein after referred as LCY) SBSs have increasing linear relation with R^2 values 0.995 and 0.965, respectively. The relations are good enough to predict the high continuous PG of base binder with various percentages of AT and LCY SBS polymers.

Figure 37(b) shows the relationship between continuous high PG of RTR modified binders and RTR percentage. Linear correlation is performed for both DVR and CR modifications. The equations of the best-fit lines were provided with salient test statistics. R^2 values for both DVR and CR are very close to 1 (0.993 and 0.998, respectively). Mathematical expressions included both in Figure 37(a) and Figure 37(b) can be used to formulate the modifications and obtain required PG binders with similar type of binder, SBS and RTR modifiers.

The combined effect of SBS and RTR on performance grade was one of the main objectives of this study. There were two levels of combinations in the experimental plan of the study. The first level (Level-1) included 2%SBS modification and replacement of 1%SBS with 3%RTR while the second level (Level-2) included 3%SBS modification and replacement of either 1%SBS with 3%RTR or 2%SBS with 6%RTR. Figure 36 shows the effect of SBS and RTR modifications on the continuous high PG.

The 0% modification shown in any of the figure means the binder went through the same modification process without the addition of any modifier. It should be noted that the modification processes are not the same for all the modifiers used in this study. While SBS and/or DVR modification require the use of high and low shear mixing consecutively, only low shear blending is necessary for CR modification. This is the reason for having different values at 0% modifications in the figures for all continuous PG measurements when CR is used.

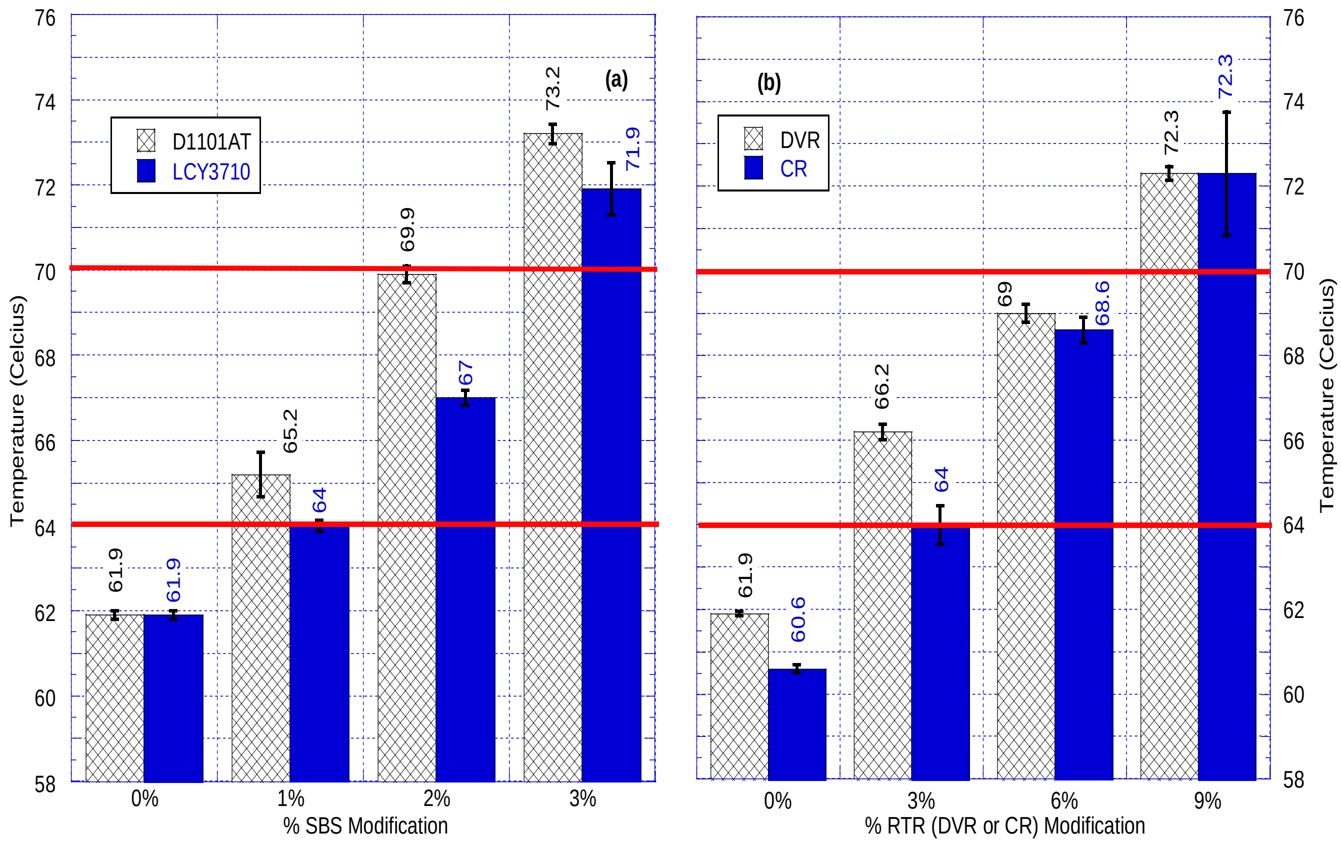

Figure 36 Effect of (a) SBS modification and (b) RTR modification on continuous high PG

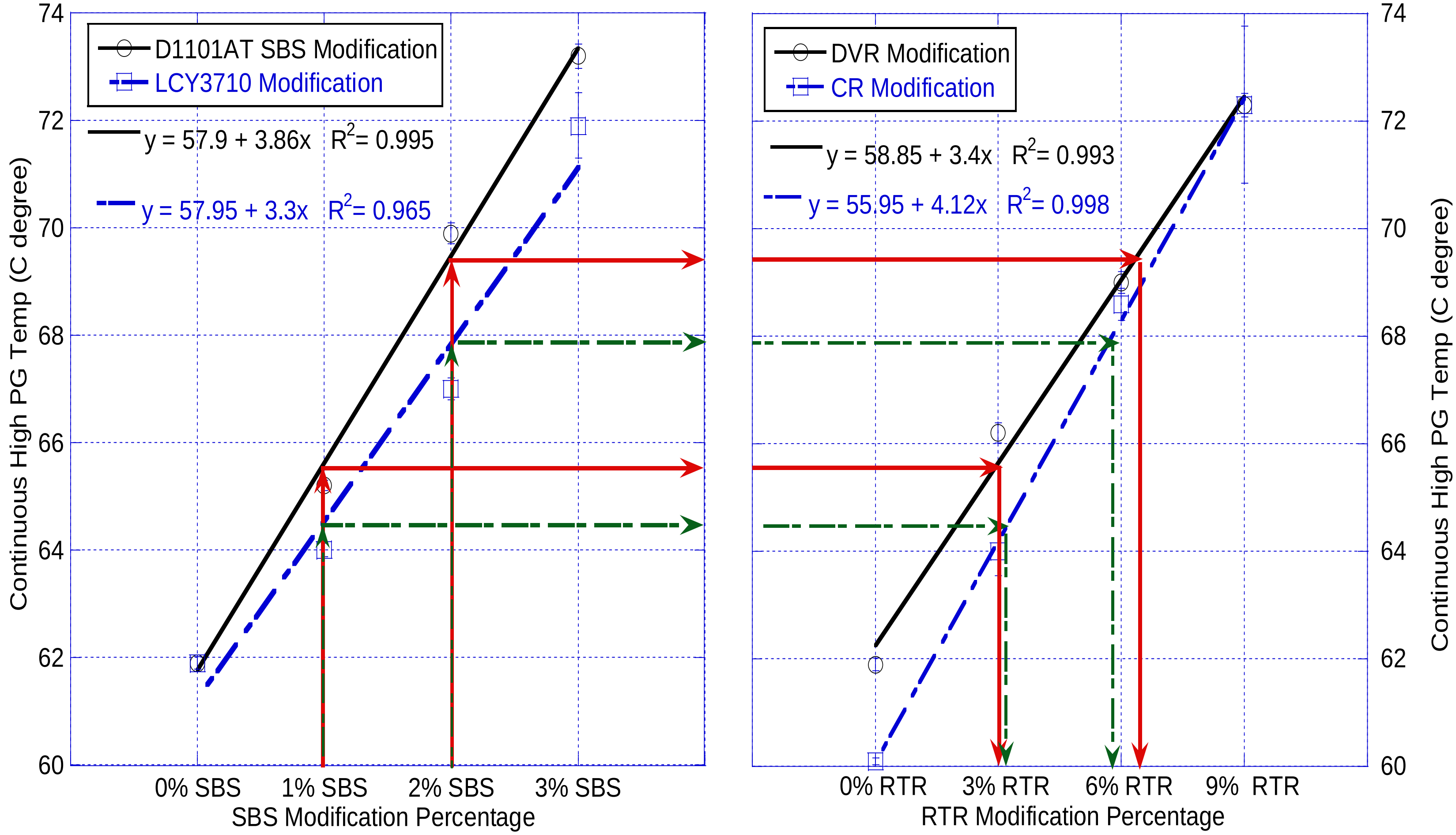

Figure 37 Effect of increasing (a) SBS and (b) RTR modifier on continuous high PG

Figure 38(a) illustrates the continuous high PG results of Level-1 combined modifications. Hypothesis for combinations was to obtain the same or even better continuous high PGs. Although hypothesis was proven for CR replacement for both SBS types, DVR replacement slightly failed to achieve it compared. The average high PG difference between SBS and SBS+CR modifications was 0.3^{o}C in favor of SBS modifications (both AT and LCY). On the other hand, the difference between SBS and SBS+DVR modifications was around 2^{o}C. This might be partially due to insufficient cross-linker (XL) volume. In comparison to 20:1 polymer to XL ratio used in SBS modified asphalt binders, this ratio was determined for 40:1 for devulcanized rubber modified asphalt binders. Since DVR particles dissolve inside the liquid binder and polymerize unlike swelling of CR particles, incorporation of enough XL into binder mix is vital to successfully complete the reaction.

Figure 38(b) demonstrates the continuous high PG results of combined modifications for level-2. Similar trend observed in level-1 was also obtained in level-2 combinations with higher differences. When 1%SBS was replaced with 3% CR (level-2, grade-1), the differences between PGs was around 3^{o}C. However, when it was replaced with 3%DVR continuous high PG dropped approximately 4^{o}C. In case of 2%SBS replacement with 6%CR (level-2, grade-2), the PG achievement was 0.4^{o}C in favor of combined modification. In contrast, substitution of 2%SBS with 6%DVR resulted in even higher performance grade drop, approximately 5^{o}C difference between the high PGs.

All these findings reinforce the observation about potential deficiency of XL agent to complete the polymerization reaction. Further studies will be needed to investigate the impact of XL on combination modifications.

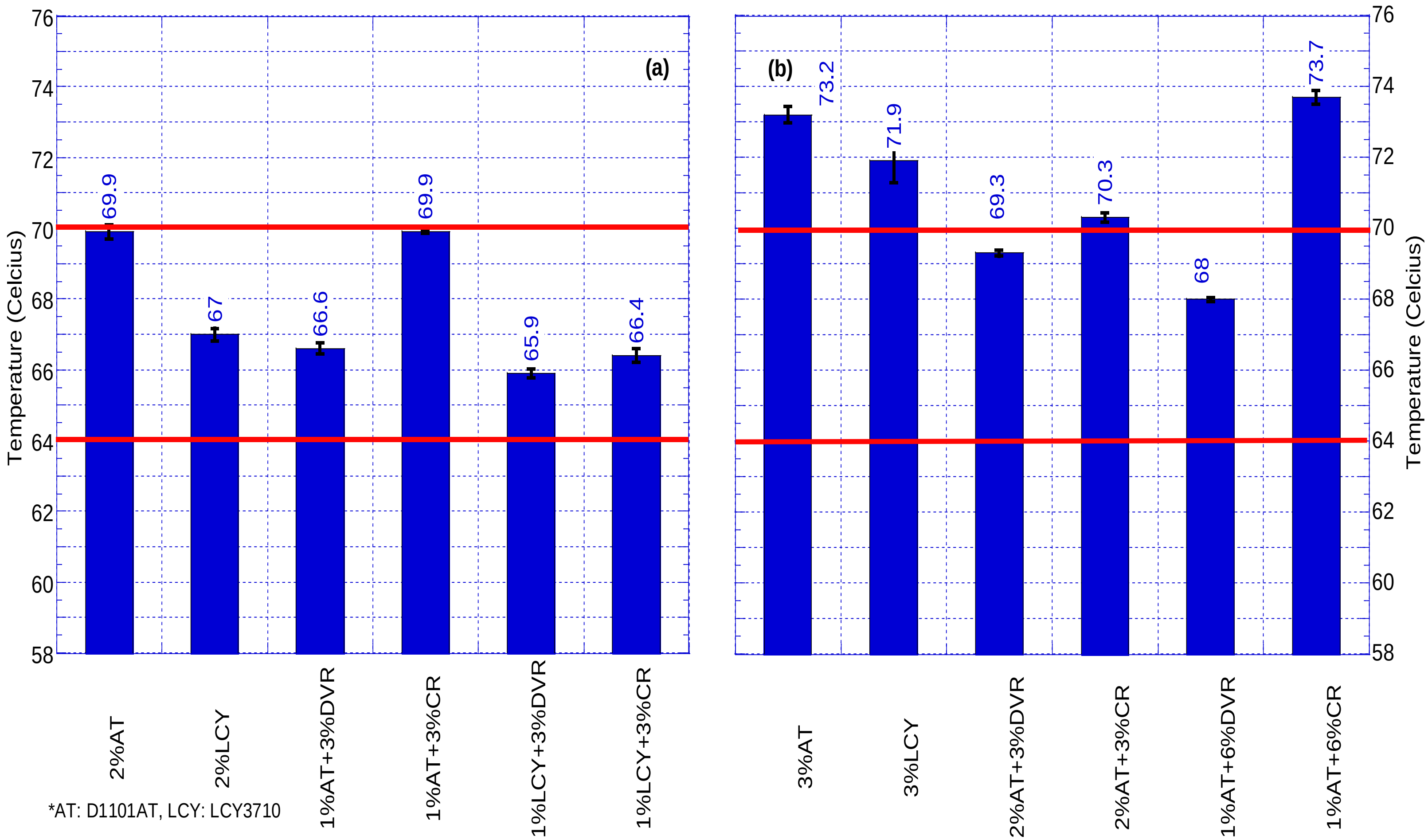

Figure 38 Continuous high PG of (a) Level-1 and (b) Level-2 combined modifications

5.4.1.2 Continuous intermediate PG results

Continuous intermediate PG of the modified binders was determined to investigate the fatigue cracking behavior of the binders according to Superpave PG system. As the intermediate PG decreases, it is believed that the binder is more flexible and performs better in fatigue cracking resistance. Intermediate PG is somewhat weak in addressing fatigue cracking alone and it needs to be accompanied with performance tests (e.g., fatigue). Table 20 illustrates the continuous intermediate (CI) PGs of all binders. 0%-0% intersection on the table is the CI-PG of aged binder (just mixing with high and low shear mixers without any modifiers addition) and neat (unmodified) binder had a CI-PG of 17.0^{o}C.

Table 20 Continuous intermediate PGs of tested binders

Modification Matrix			DVR				CR		
			0%	3%	6%	9%	3%	6%	9%
PHASE 1	SBS1- (D1101AT)	0%	17.9	16.6	16.4	16.5	15.3	14.2	14.6
PHASE 1	SBS1- (D1101AT)	1%	17.2	16.0	15.2	N/A	16.5	13.9	N/A
PHASE 1	SBS1- (D1101AT)	2%	17.4	14.7	14.7	N/A	15.2	14.4	N/A
PHASE 1	SBS1- (D1101AT)	3%	16.3	N/A	N/A	N/A	N/A	N/A	N/A
PHASE 2	SBS2- (LCY3710)	1%	16.3	16.6	N/A	N/A	16.2	N/A	N/A
PHASE 2	SBS2- (LCY3710)	2%	16.1	N/A	15.9	N/A	N/A	14.0	N/A
PHASE 2	SBS2- (LCY3710)	3%	15.8	N/A	N/A	N/A	N/A	N/A	N/A

Table 21 shows the steps of determining continuous intermediate performance grades of AT and LCY modified asphalt binders. Failure criterion $|G^*| \times \sin(\delta) = 5000$kPa was applied as per AASHTO M320 requirements. Standard deviation (Stdev) and coefficient of variation (COV) of the test results between batch-1 and batch-2 are provided to ensure the repeatability of the modification procedure. Intermediate PG construction tables for modified and unmodified asphalt binders are provided in the appendix section.

Figure 39 illustrates the effect of individual SBS and RTR modifications on continuous intermediate PG of modified asphalt binders. It should be noted that while 0%AT and 0%DVR illustrate the original binder CI-PG, 0%LCY and 0%CR show the CI-PG of aged binder.

Although there was not any clear trend between modifications and CI-PGs, the followings could be inferred from Figure 39(a) and Figure 39(b) for individual modifications.

- DVR modifications improved the CI-PG of original asphalt binder although they did not show significant enhancements. (It is noted that the lower the CI-PG, the better the binder). For all DVR modifications, CI-PG was within 0.5^{o}C improvement interval compared to original binder.
- Individual CR modifications showed an increasing improvement up to 6%CR addition, however the improvement seemed to have stopped at 9%CR. CI-PGs of combined CR modifications were enhanced with increasing CR content.
- Although D1101AT modifications showed an improvement with slight fluctuations, the improvement of LCY3710 followed a clear trend as the percent SBS increased.

Table 21 Construction of continuous intermediate PG of SBSs D1101AT and LCY3710 modified asphalt binders

	D1101AT								
	1%			2%			3%		
	Batch 1 Average	Batch 2 Average	Average	Batch 1 Average	Batch 2 Average	Average	Batch 1 Average	Batch 2 Average	Average
Tc (^oC)	16.95	17.34	17.15	17.50	17.30	17.40	16.40	16.09	16.25
T1 (^oC)	19.00	19.00	**Stdev**	19.00	19.00	**Stdev**	19.00	19.00	**Stdev**
T2 (^oC)	16.00	16.00	0.27	16.00	16.00	0.14	16.00	16.00	0.22
P1 (kPa)	3596.6	3707.7	**COV**	3976.3	3824.7	**COV**	3378.1	3048.0	**COV**
P2 (kPa)	5830.3	6356.7	1.58%	6288.7	6133.7	0.82%	5312.4	5076.9	1.36%
Ps (kPa)	5000.0	5000.0		5000.0	5000.0		5000.0	5000.0	
	LCY 3710								
	1%			**2%**			**3%**		
	Batch 1 Average	**Batch 2 Average**	**Average**	**Batch 1 Average**	**Batch 2 Average**	**Average**	**Batch 1 Average**	**Batch 2 Average**	**Average**
Tc (^oC)	16.00	16.55	16.28	16.18	16.05	16.12	15.54	16.03	15.78
T1 (^oC)	16.00	19.00	**Stdev**	19.00	19.00	**Stdev**	16.00	16.00	**Stdev**
T2 (^oC)	16.00	16.00	0.39	16.00	16.00	0.09	13.00	13.00	0.35
P1 (kPa)	3336.1	3362.0	**COV**	3150.7	3065.4	**COV**	4731.4	5015.3	**COV**
P2 (kPa)	5585.1	5470.2	2.41%	5152.3	5045.6	0.56%	6783.2	6913.3	2.19%
Ps (kPa)	5000.0	5000.0		5000.0	5000.0		5000.0	5000.0	

Stdev= standard deviation, COV= coefficient of variation, Tc= continuous intermediate PG

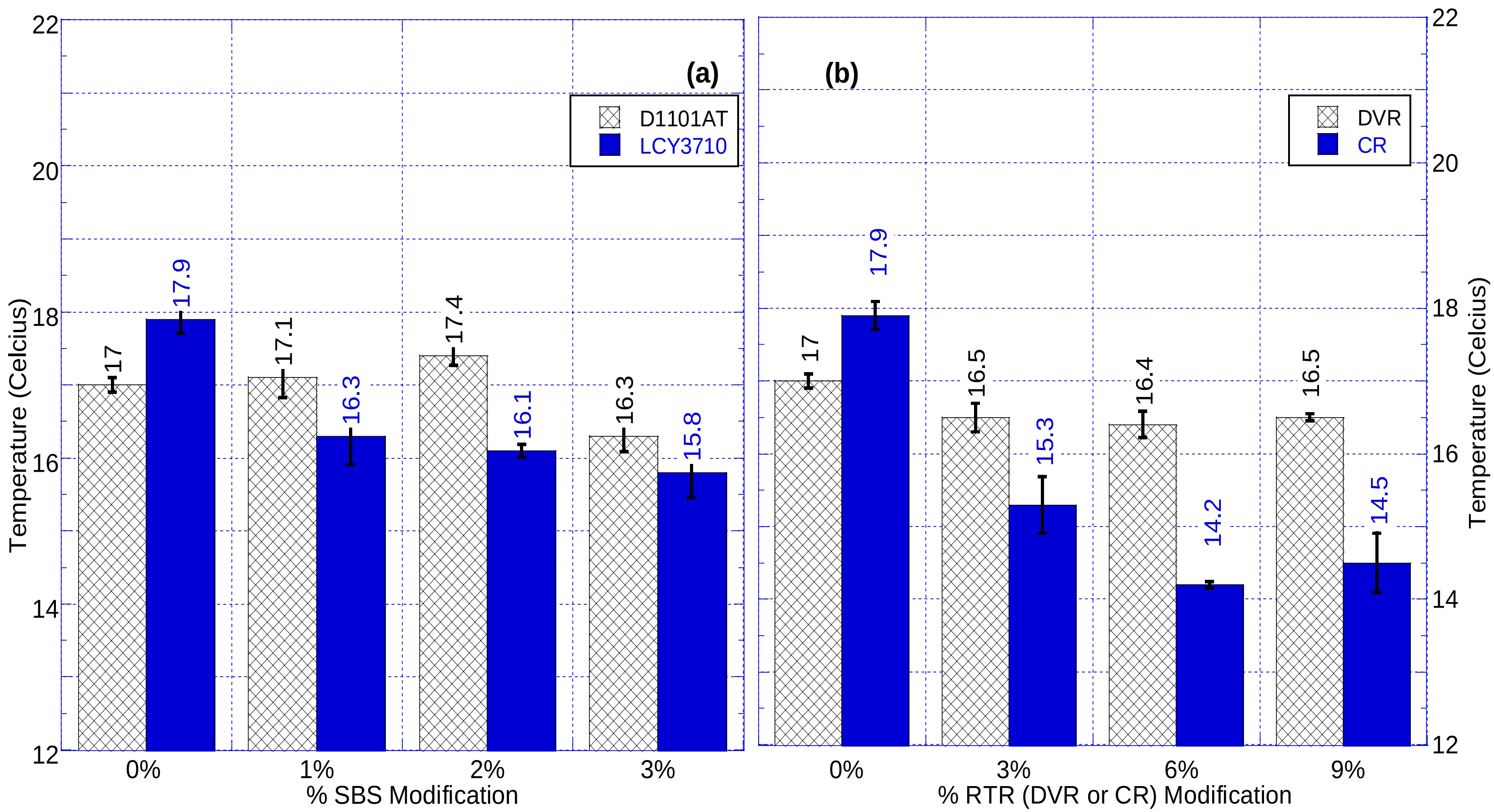

Figure 39 Effect of (a) SBS modification and (b) RTR modification on CI-PG

Figure 40 shows the effect of both level-1 and level-2 combined modifications on continuous intermediate (CI) PG of modified asphalt binders. It is clear that all combined modifications improved the CI-PGs. Figure 40(a) demonstrates the effect of level-1 combined modifications on CI-PG. Level-1 combined modifications with AT helped improving the CI-PG up to 1.4^{o}C. Compared to 17.4^{o}C CI-PG of 2%AT, 1%AT+3%DVR achieved 16.0^{o}C CI-PG. The achievement with the same AT+CR combination was 16.5^{o}C.Level-1 combinations between LCY and RTR were statistically insignificant and did not yield considerable achievements. Level-2 combined modifications are illustrated in Figure 40(b). CI-PG improvements at Level-2 were clearly significant. The best CI-PG was achieved at 1%D1101AT+6%CR combination with 13.9^{o}C. 1% and 2% AT SBS replacements with DVR and CR resulted in improvement in CI-PG by lowering the temperature between 1.1^{o}C and 2.5^{o}C.

5.4.1.3 Continuous low PG results

SBS modifications have been performed to improve the high PG of asphalt binders. This study focused on full scale performance grading to discover whether intermediate and low PGs were adversely affected with individual and combined modifications.

BBR tests were conducted at -12^{o}C, -18^{o}C and -24^{o}C since the low PG of the unmodified binder was -28^{o}C. Subsequently, continuous low PGs were calculated based on S and m-value criteria by using linear interpolation method according to ASTM D7643-10. Continuous low (CL) PG of the modified binders was determined according to stiffness (S) and logarithmic creep rate (m-value) values obtained from BBR test results at different temperatures. CL-PG was assigned as the absolute smaller of stiffness or m-value based continuous low PGs.

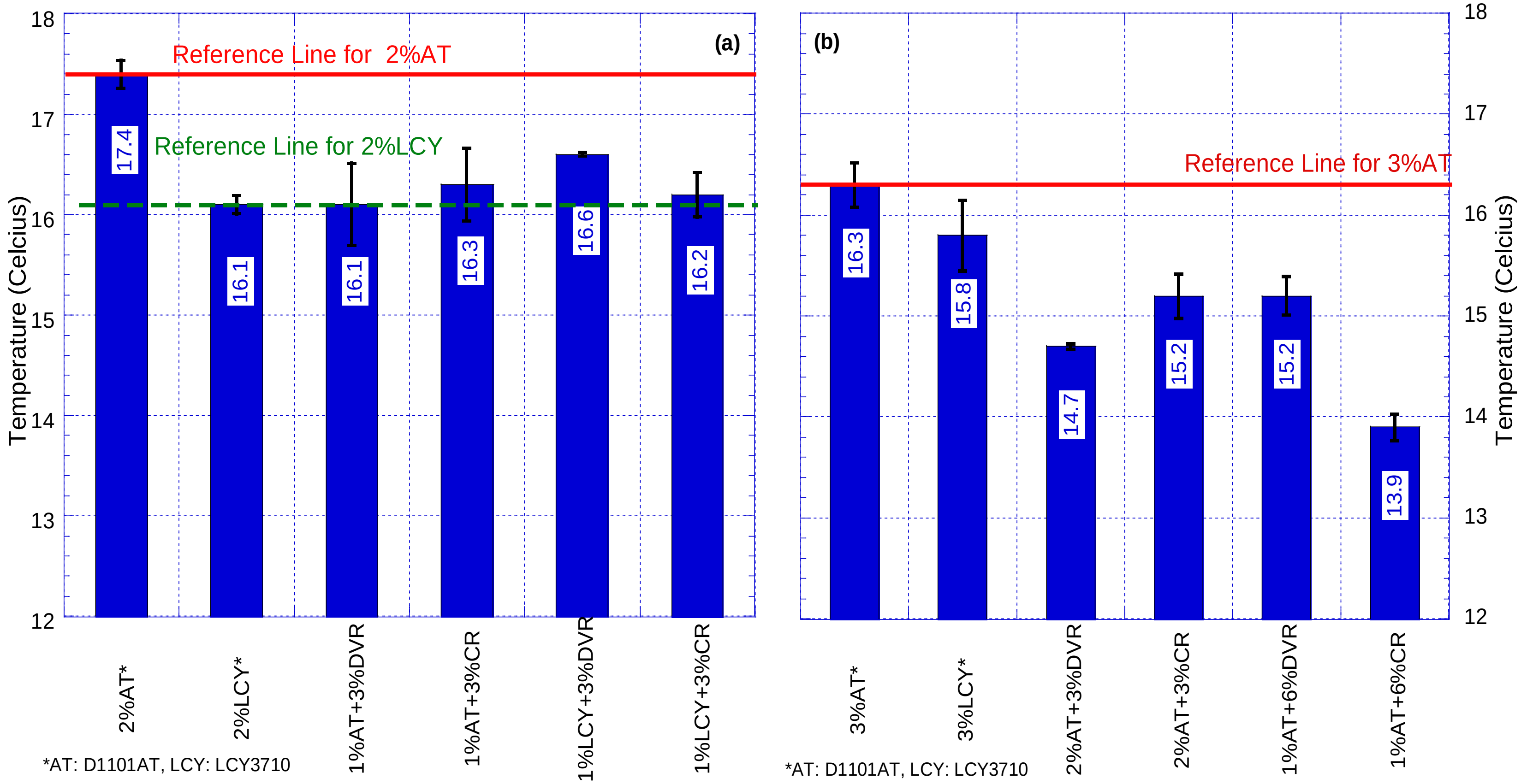

Figure 40 Continuous intermediate PG of (a) Level-1 and (b) Level-2 combined modifications

Table 22 demonstrates the construction of continuous low performance grades of AT modified asphalt binders. While the upper portion of the table shows the calculations of the continuous low temperatures based on stiffness (T_c_stiffness), lower portion shows the calculations for finding the continuous low temperatures based on BBR m-value results (T_c_m-value) measured at discrete temperatures. The last row on Table 22 illustrates the continuous low temperatures (Tc) based on the absolute (value) smaller of T_c_stiffness and T_c_m-value as per AASHTO T-313 and ASTM D7643. Standard deviation (Stdev) and coefficient of variation (COV) of the test results between batch-1 and batch-2 are calculated to ensure the successful repeatability of different batches. Continuous low temperatures construction tables for other modified and unmodified asphalt binders are provided in the appendix. It is worth noting that continuous low temperatures (T_c values) are different from the continuous low PG temperatures. Actual CL-PGs will be obtained by adding -10^{o}C to the calculated T_c values as per AASHTO M320 and the actual CL-PG values can be seen in Table 23.

Table 23 summarizes the continuous low PGs of all the binders tested. It also shows the stiffness and m-value based CL-PGs. While the left portion of the table provides the results of modifications performed by using AT SBS, right part shows the results obtained by using LCY polymer. Both sides have the same values for individual RTR (DVR and CR) modifications. Similar to RTR modifications, CL-PG of base binder (unmodified) and Aged* binder are also provided in both sides. Some of the combined modifications by using LCY elastomeric polymer were not performed since they were not included in the testing matrix of this study. The results of these modifications were marked as “N/A” (not available) in Table 23.

Table 22 Construction of continuous low temperatures of D1101AT SBS modified asphalt binders

	D1101AT Stiffness-based								
	1%			2%			3%		
	Batch 1 Average	Batch 2 Average	Average	Batch 1 Average	Batch 2 Average	Average	Batch 1 Average	Batch 2 Average	Average
Tc stiffness (^oC)	-18.85	-18.63	-18.74	-19.07	-18.87	-18.97	-20.2	-20.1	-20.15
T1 (^oC)	-18.00	-18.00	**Stdev**	-18.00	-18.00	**Stdev**	-18.0	-18.0	**Stdev**
T2 (^oC)	-24	-24	0.16	-24	-24	0.14	-24	-24	0.06
P1 (kPa)	269	277	**COV**	263	270	**COV**	227	230	**COV**
P2 (kPa)	579	589	0.83%	551	560	0.75%	486	490	0.32%
Ps (kPa)	300	300		300	300		300	300	
	D1101AT m_value-based								
	1%			**2%**			**3%**		
	Batch 1 Average	**Batch 2 Average**	**Average**	**Batch 1 Average**	**Batch 2 Average**	**Average**	**Batch 1 Average**	**Batch 2 Average**	**Average**
Tc m-value (^oC)	-18.56	-18.45	-18.50	-18.94	-19.00	-18.97	-20.8	-21.3	-21.07
T1 (^oC)	-18.00	-18.00	**Stdev**	-18.00	-18.00	**Stdev**	-18.0	-18.0	**Stdev**
T2 (^oC)	-24	-24	0.08	-24	-24	0.04	-24	-24	0.36
P1 (kPa)	0.304	0.303	**COV**	0.308	0.309	**COV**	0.323	0.326	**COV**
P2 (kPa)	0.261	0.263	0.41%	0.257	0.255	0.22%	0.274	0.279	1.69%
Ps (kPa)	0.3	0.3		0.3	0.3		0.3	0.3	
Tc (^oC)	-18.50			-18.97			-20.15		

Table 23 Continuous low PG test results

Continuous Low Performance Grade							
Modification with SBS D1101AT	Con. Low PG °C	S-Based Con. Low PG °C	m-Based Con. Low PG °C	m-Based Con. Low PG °C	S-Based Con. Low PG °C	Con. Low PG °C	Modification with SBS LCY3710
Unmodified	-29.1	-29.1	-30.6	-30.6	-29.1	-29.1	Unmodified
Aged*	-26.9	-28.3	-26.9	-26.9	-28.4	-26.9	Aged*
1% SBS	-28.5	-28.8	-28.5	-28.8	-29.4	-28.8	1% SBS
2% SBS	-29.0	-29.0	-29.0	-30.8	-29.6	-29.6	2% SBS
3% SBS	-30.2	-30.2	-31.1	-31.7	-30.0	-30.0	3% SBS
3% DVR	-28.7	-28.7	-28.9	-28.9	-28.7	-28.7	3% DVR
6% DVR	-28.1	-29.0	-28.1	-28.1	-29.0	-28.1	6% DVR
9% DVR	-28.6	-29.7	-28.6	-28.6	-29.7	-28.6	9% DVR
1% SBS + 3% DVR	-29.3	-29.3	-29.4	-28.9	-29.5	-28.9	1% SBS + 3% DVR
1% SBS + 6% DVR	-30.0	-30.0	-30.5	N/A	N/A	N/A	1% SBS + 6% DVR
2% SBS + 3% DVR	-30.5	-30.5	-30.7	N/A	N/A	N/A	2% SBS + 3% DVR
2% SBS + 6% DVR	-30.2	-30.4	-30.2	-28.1	-31.0	-28.1	2% SBS + 6% DVR
3% CR	-30.0	-30.0	-30.5	-30.5	-30.0	-30.0	3% CR
6% CR	-30.7	-31.3	-30.7	-30.7	-31.3	-30.7	6% CR
9% CR	-30.2	-32.4	-30.2	-30.2	-32.4	-30.2	9% CR
1% SBS +3% CR	-28.2	-29.2	-28.2	-28.9	-29.7	-28.9	1% SBS +3% CR
1% SBS +6% CR	-28.7	-31.5	-28.7	N/A	N/A	N/A	1% SBS +6% CR
2% SBS +3% CR	-30.7	-30.7	-30.7	N/A	N/A	N/A	2% SBS +3%CR
2% SBS +6% CR	-29.2	-31.3	-29.2	-28.4	-31.2	-28.4	2% SBS +6% CR

Con.Low PG: continuous low PG, S-Based Con. Low PG: stiffness based continuous low PG, m-Based Con. Low PG: m value based continuous low PG

One of the main observations that can be obtained from Table 23 is that the increasing SBS content resulted in a consistent decrease both in the stiffness and m-value. However, results for CR and DVR modified binders did not follow the same trend. Although the creep stiffness of RTR modified binders decreased with increasing RTR content, change in logarithmic creep rate (m-value) was inconsistent. This complies with the findings of the previous researchers.

Figure 41 demonstrates the effect of SBS and RTR modifications on continuous low PG. While Figure 41(a) shows the effect of individual AT and LCY SBS modifications, Figure 41(b) demonstrates the effect of individual DVR or CR modifications. 0% SBS modification and 0% RTR modification are the same CL-PG values of unmodified and Aged* asphalt binders. They are provided to show the comparisons between base and modified asphalt binders. Based on Figure 41, the following statements can be inferred for individual modifications;

- Aged* binder had the highest CL-PG in compliance with the high PG results since aging made the binder stiffer, better for high PG and worse for low PG. The effect was enough to reduce the binder grade from PG58-28 to PG58-22.
- Increasing the SBS content improved the CL-PG with an increasing trend for both AT and LCY elastomeric polymers. The greatest achievement was acquired with 3%AT SBS, -30.2^{o}C.
- Corresponding modifications with CR and DVR enhanced the CL-PG as good as SBS modifications. 6% CR modification achieved the greatest CL-PG with -30.7^{o}C.
- Overall, SBS and RTR modifications did not show great impact on CL-PG compared to CH-PG. This is because both modifiers are mainly used to improve the rutting performance of the asphalt binders.

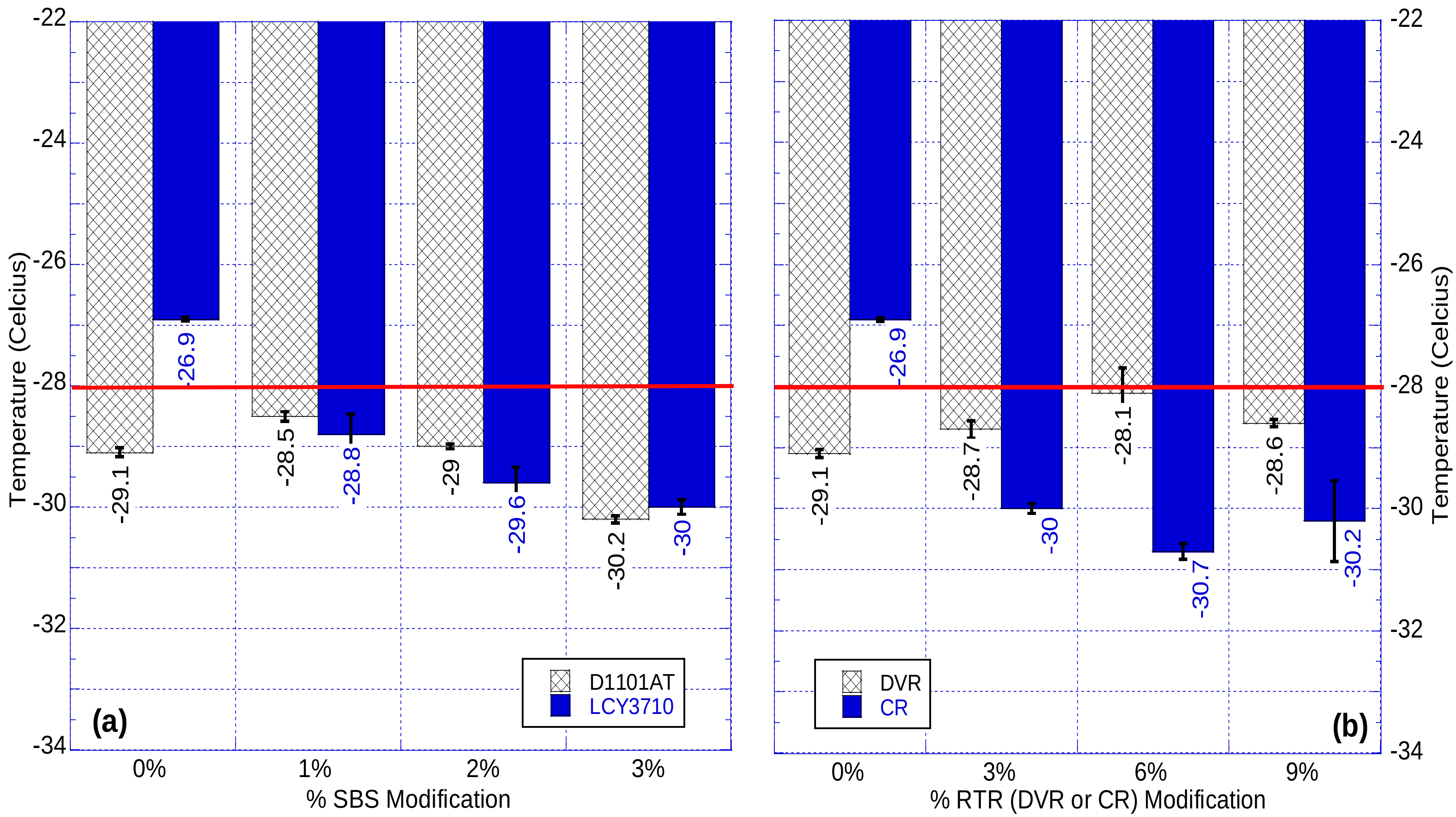

Figure 41 Effect of (a) SBS modification and (b) RTR modification on continuous low PG

Figure 42 illustrates continuous low PGs of combined modifications. While Figure 42(a) shows the effect of Level-1 on CL-PG, Figure 42(b) focuses on CL-PG of Level-2 SBS and RTR combinations.

Although Level-1 combinations with AT polymer was almost the same as AT alone, LCY combined modifications did not improve the CL-PGs. Compared to -29.8^{o}C of 2%LCY, -28.9^{o}C CL-PG is obtained on both 1%LCY+3%DVR and 1%LCY+3%CR combined modifications. This might be the impact of high vinyl content of LCY polymer and further studies are needed to investigate this phenomenon. Overall, Level-1 combinations enhanced the CL-PG of modified binders compared to original and Aged* binders.

Level-2 combined modifications can be seen in Figure 42(b). The effect of RTR replacement at grade-1 (replacement of 1%SBS with 3%RTR) helped improving the CL-PG average -0.4^{o}C. However, in the grade-2 (replacement of 2%SBS with 6%RTR) replacements, the CL-PG of DVR was almost the same as 3%AT and CR did not achieve better results.

The improvements on continuous low PG for Level-2 modifications can be stated as follows:

- CL-PG of 3% D1101AT (-30.2^{o}C) $\cong$ CL- PG of 1% D1101AT + 6% DVR (-30.0)
- CL-PG of 3% D1101AT (-30.2^{o}C) > CL-PG of 2% D1101AT + 3% CR (-28.2^{o}C)
- CL-PG of 3% D1101AT (-30.2^{o}C) < CL-PG of 2%D1101AT + 3% DVR (-30.5^{o}C) & 2%D1101AT + 3% CR (-30.7^{o}C)
- CL-PG of 3% D1101AT (-30.2^{o}C) $\cong$ CL-PG of 3% LCY3710 (-30.0^{o}C)

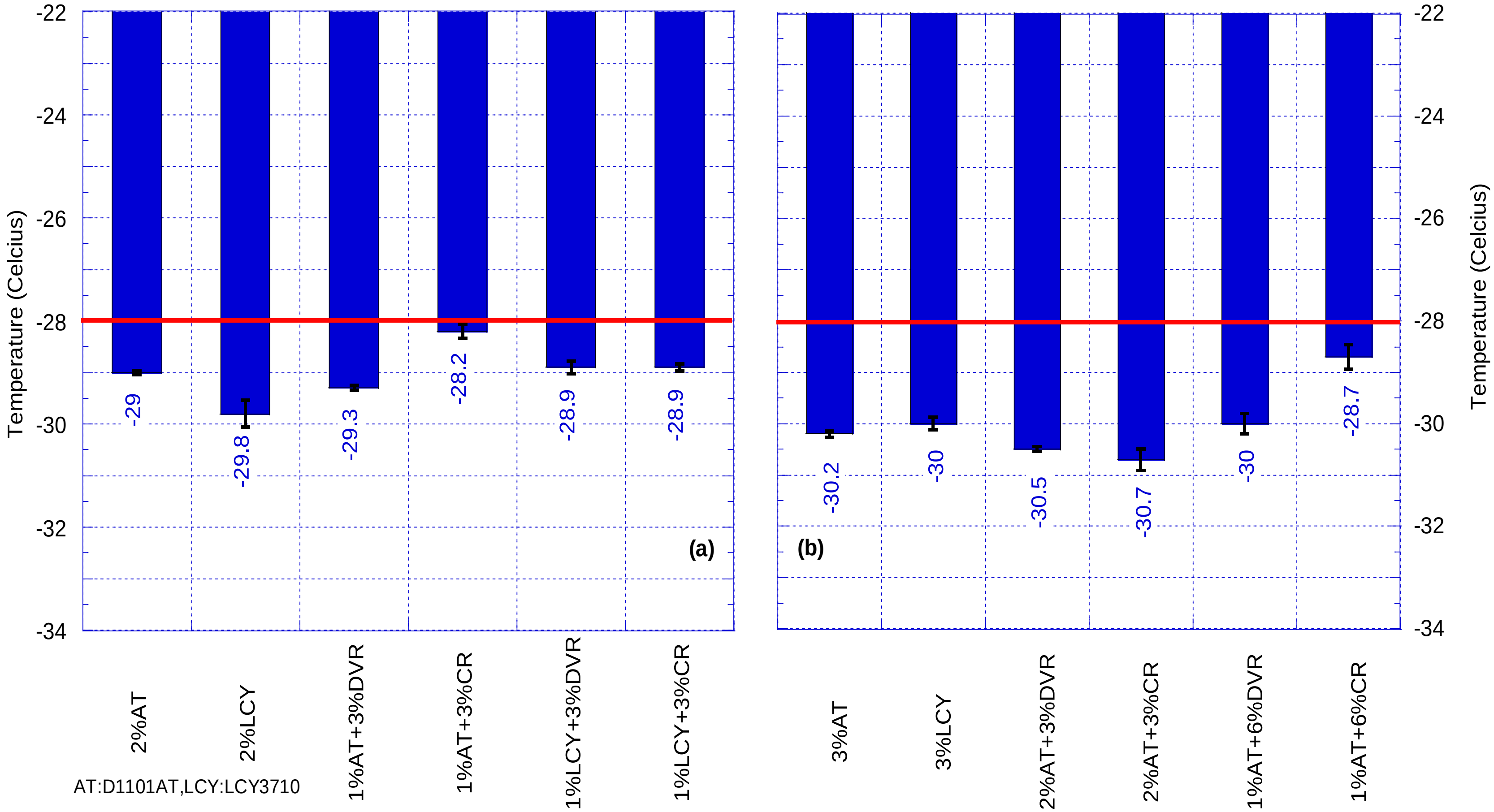

Figure 42 Continuous low PG of (a) Level-1 (b) Level-2 combined modifications

5.4.2 Additional Binder Performance Test Results

Asphalt binders are advertised and sold according to PGs in conformity with Superpave binder specifications. Since continuous performance grades of LCY3710 SBS polymer modifications did not perform better than D1101AT, further binder performance tests were not conducted for LCY modified asphalt binders, including the combinations.

5.4.2.1 Linear amplitude sweep test results

Linear Amplitude Sweep (LAS) test was performed to investigate the fatigue life of modified binders. LAS test was analyzed using a special type of viscoelastic continuum damage model. It determines the cycles to failure at different strain levels. In this study, strain levels 2.5% and 5.0% were chosen. LAS tests were performed at continuous intermediate PG temperatures of modified and unmodified binders. Testing temperatures can be seen in Table 20.

In Figure 43, LAS test results of Unmodified, Aged*, 1% D1101AT, 3%DVR and 3%CR binders were presented both at 2.5% and 5.0% strain levels. Although the modifications did not result in improvement at 2.5% strain level, 1%SBS and 3%DVR modifications advanced the fatigue life at 5.0% strain level compared to unmodified and Aged* binders. This achievement is vital because heavy truck traffic induce higher strains on the asphalt pavement. The effect of CR modification at 3% did not enhance the fatigue life at both strain levels.

Figure 44 illustrates the fatigue life obtained using LAS test on Level-1 and Level-2 asphalt binder modifications at 2.5% and 5.0% strain levels. The similar improvement of Figure 43 was observed in Figure 44 for individual SBS and DVR modifications as well.

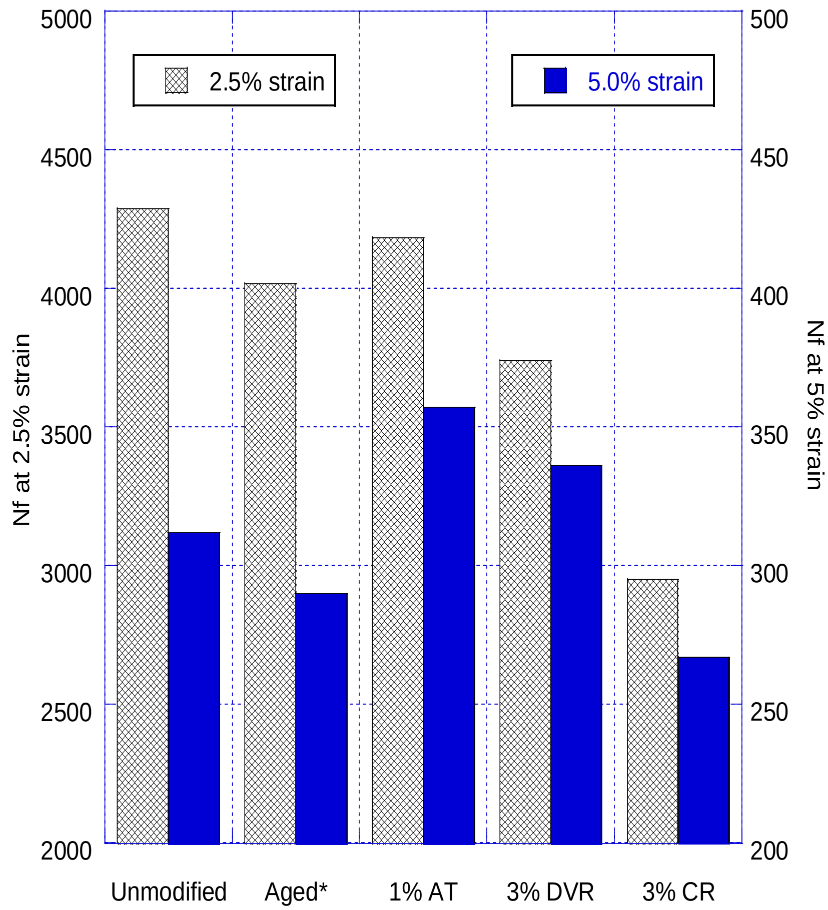

Figure 43 LAS results of 1% D1101AT and corresponding RTR modifications

In Figure 44(a), it can be observed that doubling the amount of SBS and DVR modifiers resulted in improvements at both strain levels in individual modifications. At this level individual CR modification did not enhance the fatigue life considerably. Moreover, combined effects with DVR and CR did not show any improvements. However, the fatigue lives of both combined modifications at high strain level are so close to each other. This can be attributed to the presence of SBS polymer.

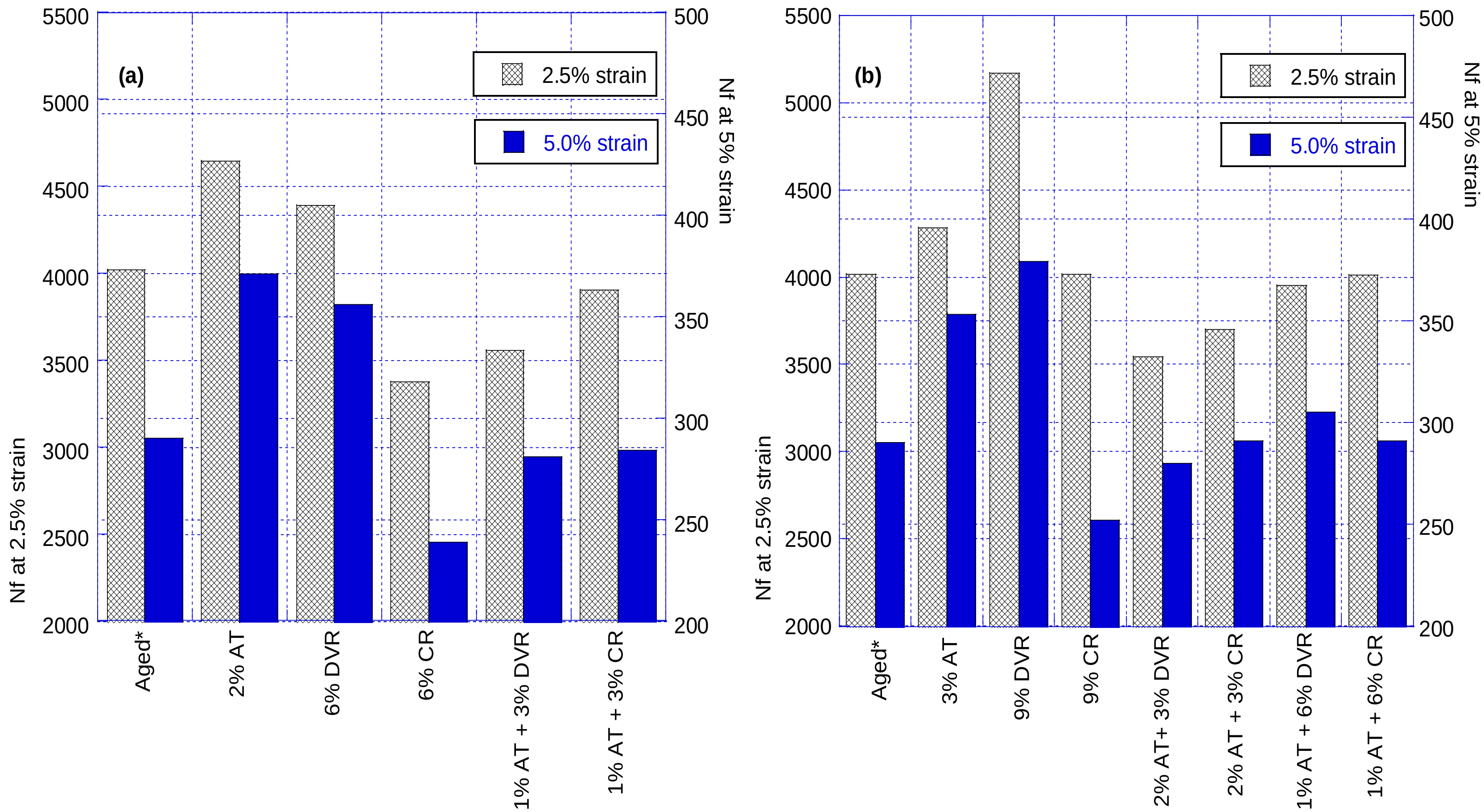

Figure 44 LAS results of (a) Level-1 and (b) Level-2 combined modifications

Figure 44(b) demonstrates the LAS fatigue life results of Level-2 combined modifications along with 3% SBS and 9% RTR modifications. Individual DVR modification helped improving the fatigue life at both strain levels compared to SBS modification. DVR modification showed an enhancing fatigue life trend with increasing modifier percentage. While DVR modification was comparable to SBS modification at 3% and 6%, individual DVR modification at 9% outperformed any other individual and combined modifications, reaching N_f value over 5000 at 2.5% strain level. Modified binders prepared with SBS+DVR combinations did not enhance the fatigue life compared to individual SBS and DVR modifications. However, addition of SBS to individual CR modified binders enhanced the fatigue life both at 2%+3% and 1%+6% combinations. When Figure 44(a) and Figure 44(b) are compared, any DVR or CR combinations had the similar fatigue life at both strain levels. This could be due to the saturation level combined modifications are reaching. It is worth to investigate this phenomenon in detail during future researches.

5.4.2.2 MSCR test results

MSCR test was performed for all modified binders instead of the elastic recovery test. Percent recovery and non-recoverable creep compliance of the modified binders were investigated according to AASHTO T350 and TP70. The data was analyzed at different SBS, CR and DVR combinations. J_{nr} difference for all modified binders was less than 75% according to AASHTO M332 other than 20% CR modified binder which had a value of 177%. The reason for having a higher percent J_{nr} difference was the fact that heavily modified asphalt binder barely experienced non-creep compliance at 0.1 kPa shear stress. This research suggests improvements to be done on AASHTO M332-14 “percent J_{nr} difference” section for heavily modified binders.

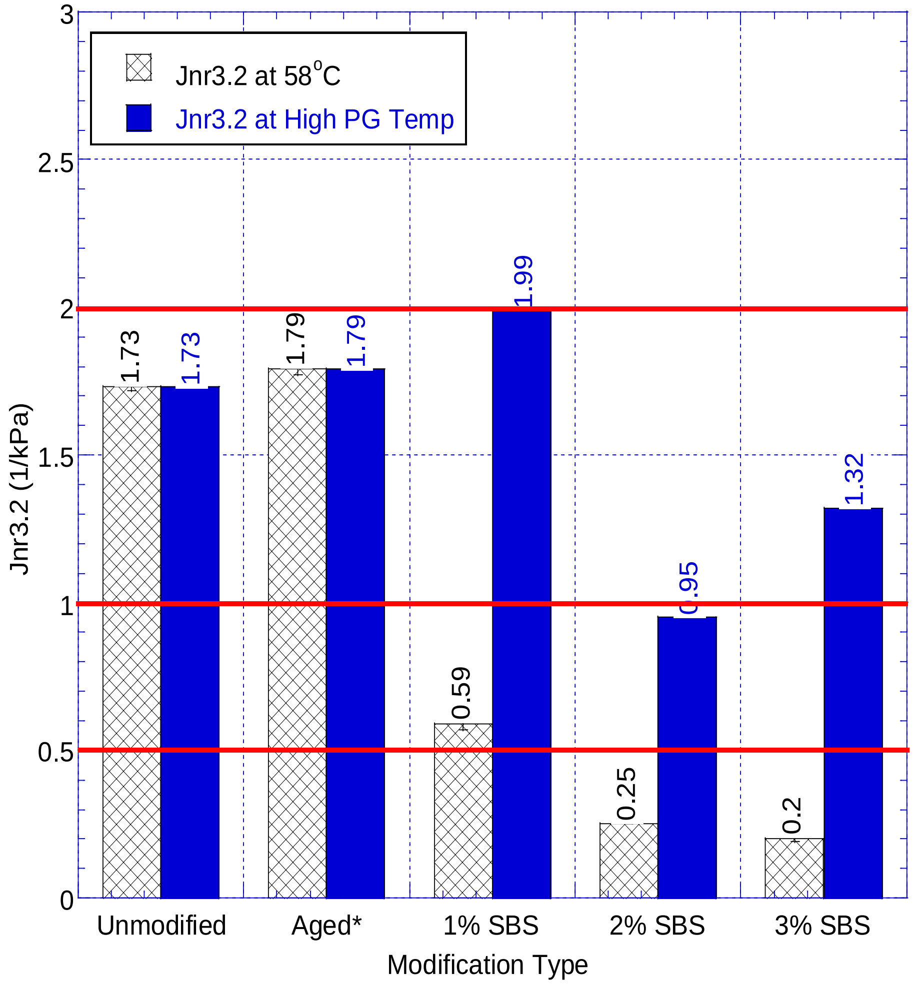

Figure 45 MSCR results of SBS modifications

Since this research started earlier than the approved standards, test results include both approved and provisional standard measurements namely T350 and TP70, respectively. Main difference between these standards is the testing temperature.

While the MP19 stated that MSCR testing should be performed at high PG temperatures of the asphalt binders by using TP70, M332 suggests testing at regional/local temperatures using T350. The tests were performed at both high PG temperatures and location temperatures for grade bumping.

Table 24 MSCR test results of original and modified asphalt binders at 58°C

Modified Binder Type	**Average $J_{nr3.2}$**	**St.Dev $J_{nr\ 3.2}$**	**Average Jnr diff**	**Traffic Grade**	**COV**	**Average $R_{3.2}$**	**St.Dev $R_{3.2}$**	**COV**
	1/kPa		**%**			**%**		
Original	1.73	0.013	14	H	0.8%	1.03	0.042	4.1%
Aged	1.79	0.018	14	H	1.0%	0.92	0.014	1.5%
1%SBS	0.59	0.019	20	V	3.2%	17.4	0.049	0.3%
2%SBS	0.25	0.000	23	E	0.0%	42.34	0.262	0.6%
3%SBS	0.20	0.008	48	E	4.0%	67.52	0.764	1.1%
3%DVR	0.75	0.006	19	V	0.8%	7.04	0.042	0.6%
6%DVR	0.49	0.002	24	E	0.4%	12.93	0.064	0.5%
9%DVR	0.20	0.005	24	E	2.5%	30.09	0.028	0.1%
1%SBS & 3%DVR	0.73	0.010	26	V	1.4%	15.12	0.021	0.1%
1%SBS & 6%DVR	0.62	0.005	30	V	0.8%	17.99	0.028	0.2%
2%SBS & 3%DVR	0.47	0.000	36	E	0.0%	35.09	0.064	0.2%
2%SBS & 6%DVR	0.27	0.001	44	E	0.4%	44.94	0.509	1.1%
2%SBS & 7%DVR	0.18	0.000	41	E	0.0%	64.51	0.000	0.1%
3%CR	1.10	0.002	44	H	0.1%	11.11	0.134	1.2%
6%CR	0.59	0.006	46	V	0.9%	19.88	0.545	2.7%
9%CR	0.29	0.003	38	E	1.1%	32.54	0.926	2.9%
20%CR	0.07	0.001	177	E	1.6%	68.94	0.46	0.7%
1%SBS & 3%CR	0.55	0.022	37	V	4.0%	21.07	0.056	0.3%
1%SBS & 6%CR	0.22	0.010	68	E	4.5%	54.23	0.530	1.0%
2%SBS & 3%CR	0.40	0.016	57	E	3.9%	38.86	0.898	2.3%
2%SBS & 6%CR	0.24	0.005	74	E	2.2%	40.75	0.134	0.3%

SBS: styrene-butadiene-styrene D1101AT, DVR: devulcanized rubber, CR: crumb rubber -#20 mesh, St.Dev: standard deviation

Table 24 demonstrates the average non-recoverable creep compliance at 3.2 kPa ($J_{nr3.2}$), average percent J_{nr} difference, average percent recovery at 3.2 kPa (%$R_{3.2}$), traffic grade, standard deviation (St.Dev) and coefficient of variation (COV) for $J_{nr3.2}$ and %$R_{3.2}$ for all modified and original asphalt binders at regional testing temperature 58^{o}C.

Figure 45 illustrates the MSCR results for SBS (D1101AT) modifications. The aging during the modification process did not affect the MSCR results considerably (Note: Aged* binder only went through the same high and low shear mixing processes as SBS modified binders). On the other hand, the addition of SBS resulted in a decrease at $J_{nr3.2}$ at 58^{o}C. As the percentage of SBS increased, the decrease on Jnr became more pronounced. However, the trend was not the same for tests at high PG temperatures. While traffic grade of unmodified/aged binder was "H" at local and PG temperatures, 1% SBS modification had V-grade and 2% and 3% had E-grade traffic level at regional testing temperature.

Figure 46(a) illustrates $J_{nr3.2}$ results of original (unmodified), Aged*, 1% SBS and 3% RTR modifications. The difference between unmodified and aged* binders was negligible. At temperature (58^{o}C), the addition of 1% SBS or 3%DVR resulted in the traffic grade bump from H-grade to V-grade. 3% CR addition also helped to lower the non-recoverable creep compliance without achieving a traffic grade level bump.

At high PG temperature, $J_{nr3.2}$ values of the modified binders increased since each of them had different testing temperatures and viscosity of the binders were lowered with an increasing temperature.

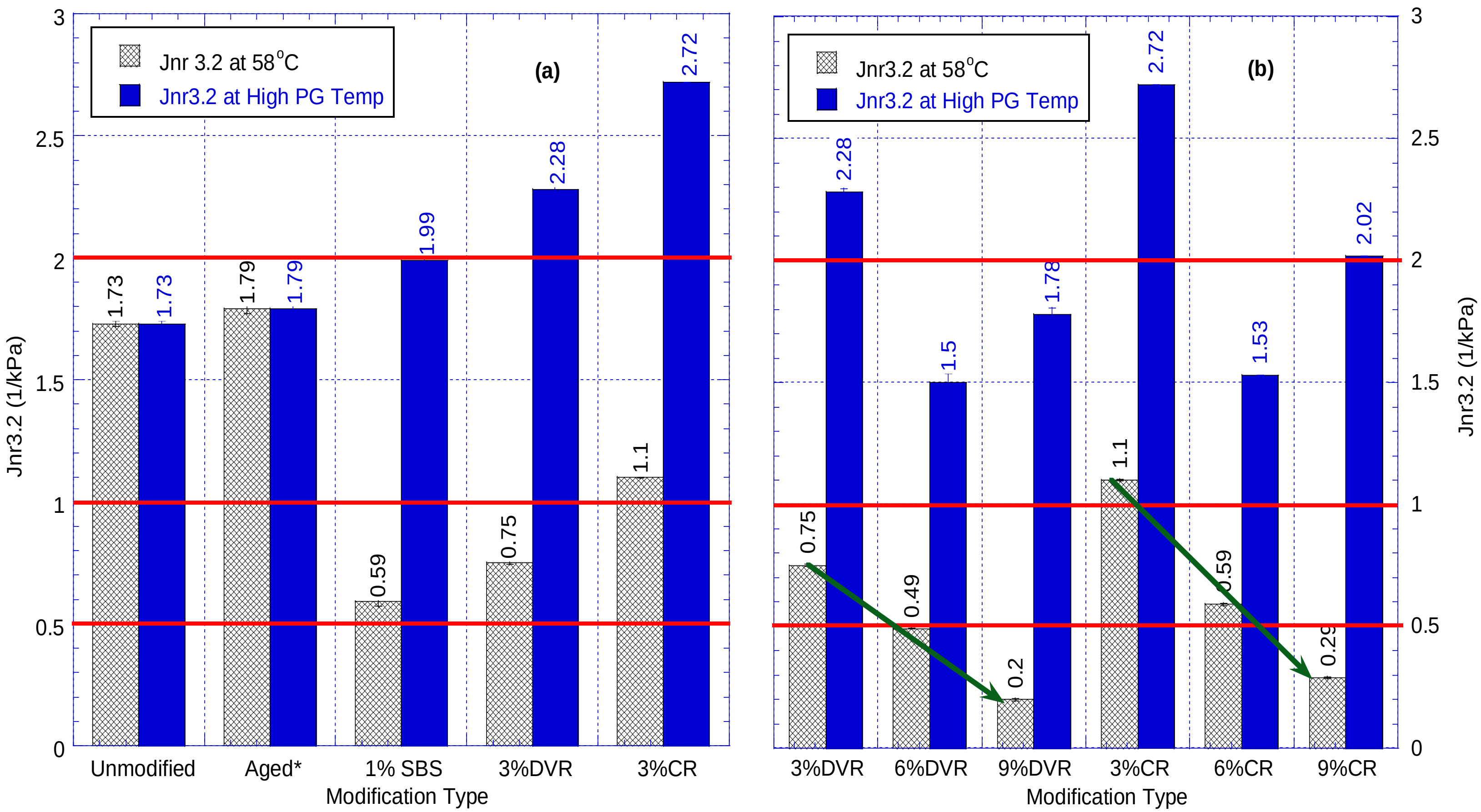

Figure 46 MSCR results of SBS and corresponding RTR modifications

The effect of RTR modifications on MSCR Jnr values are provided in Figure 46(b). Although there was not a clear trend between $J_{nr3.2}$ and percent modifier at high PG testing temperatures, the increasing percentage of RTR yielded better (lower) $J_{nr3.2}$ results. Moreover, the results of non-recoverable creep compliance at 3.2 kPa for DVR modified asphalt binders were better than CR modified binders at both temperature levels. 6 % DVR addition at local testing temperature achieved 2-traffic grade bumping from heavy to extremely heavy level which corresponds PG 70-XX according to AASHTO M320. On the other hand, addition of 6% CR only achieved one traffic grade bump at the same testing temperature. Both DVR and CR modifications at 9% successfully accomplished extremely heavy traffic grade “E”.

Figure 47(a) demonstrates the MSCR test results for Level-1 combined modifications compared to individual SBS and RTR modifications. $J_{nr3.2}$ results at regional testing temperature (58^{o}C) show that the differences between individual and combined modifications were small. The combined modifiers can conveniently be used as replacement to each other to produce more economic modified asphalt binders at Level-1 combinations.

Level-2 combined modifications and non-recoverable creep compliance at 3.2 kPa obtained from MSCR tests are demonstrated in Figure 47(b). Test results at 58^{o}C show that the all Level-2 modifications achieved the extremely heavy traffic grade “E” other than 1%SBS+6DVR which is very close to E-grade limit. Figure 47(b) implies that replacement of SBS with RTR at Level-2 combinations is quite possible without adversely affecting MSCR traffic grade level. Testing at high PG temperatures did not follow any trend between $J_{nr3.2}$ and percent modifiers.

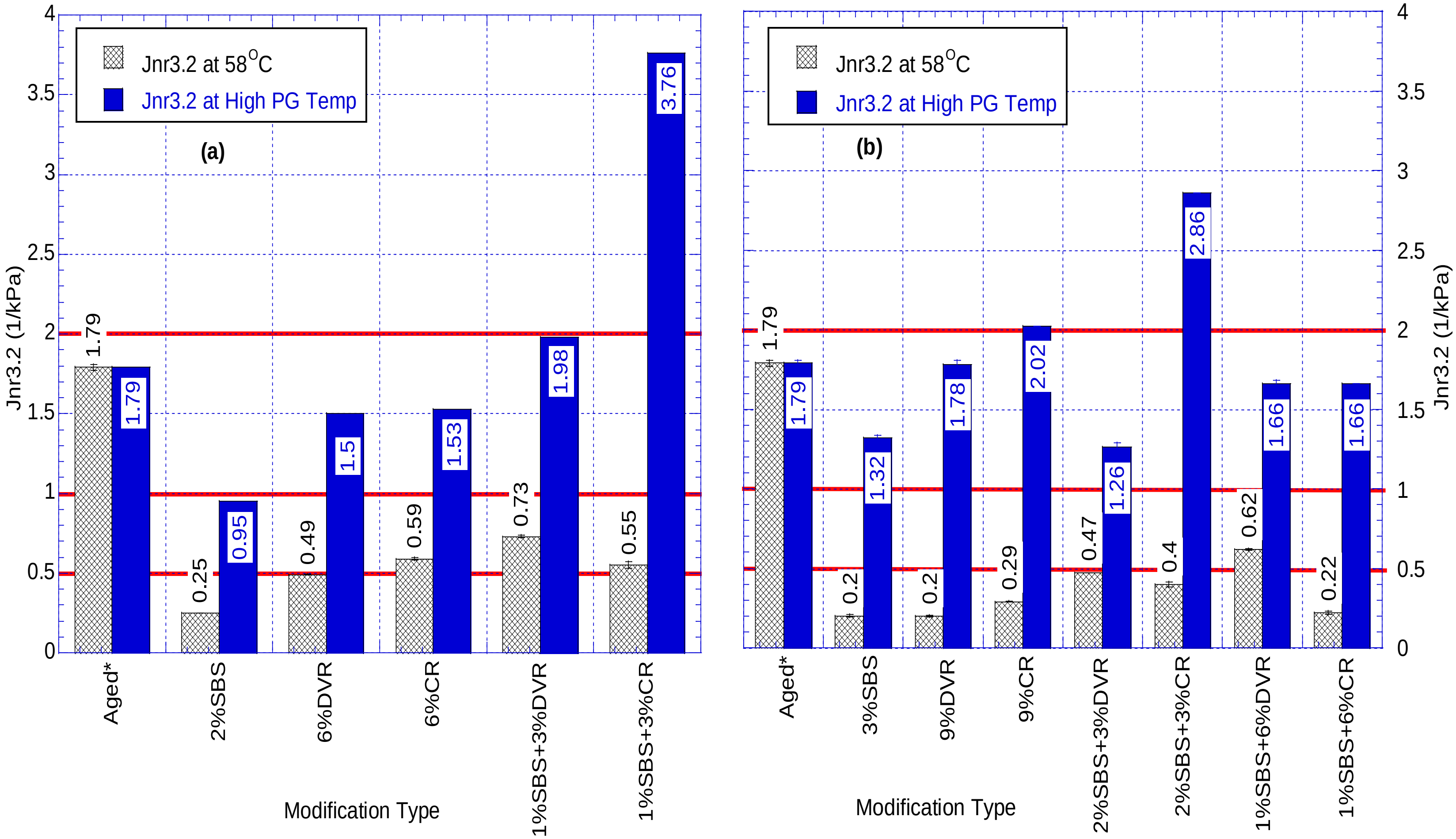

Figure 47 MSCR nonrecoverable creep compliance at 3.2kPa results of (a) Level-1 and (b) Level-2 combined modifications

Percent recovery is another parameter obtained during MSCR test. Figure 48 illustrates the elastic response curve and percent recovery of individual and combined SBS and DVR modified asphalt binders along with unmodified and aged ones. According to the AASHTO M332, binders falling above the elastic curve have enough elastic modifiers. This curve is an efficient way to demonstrate the traffic grade of the asphalt binders as well. While the y-axis shows the $\%R_{3.2}$, x-axis illustrates the corresponding $J_{nr3.2}$ values tested at the same temperature.

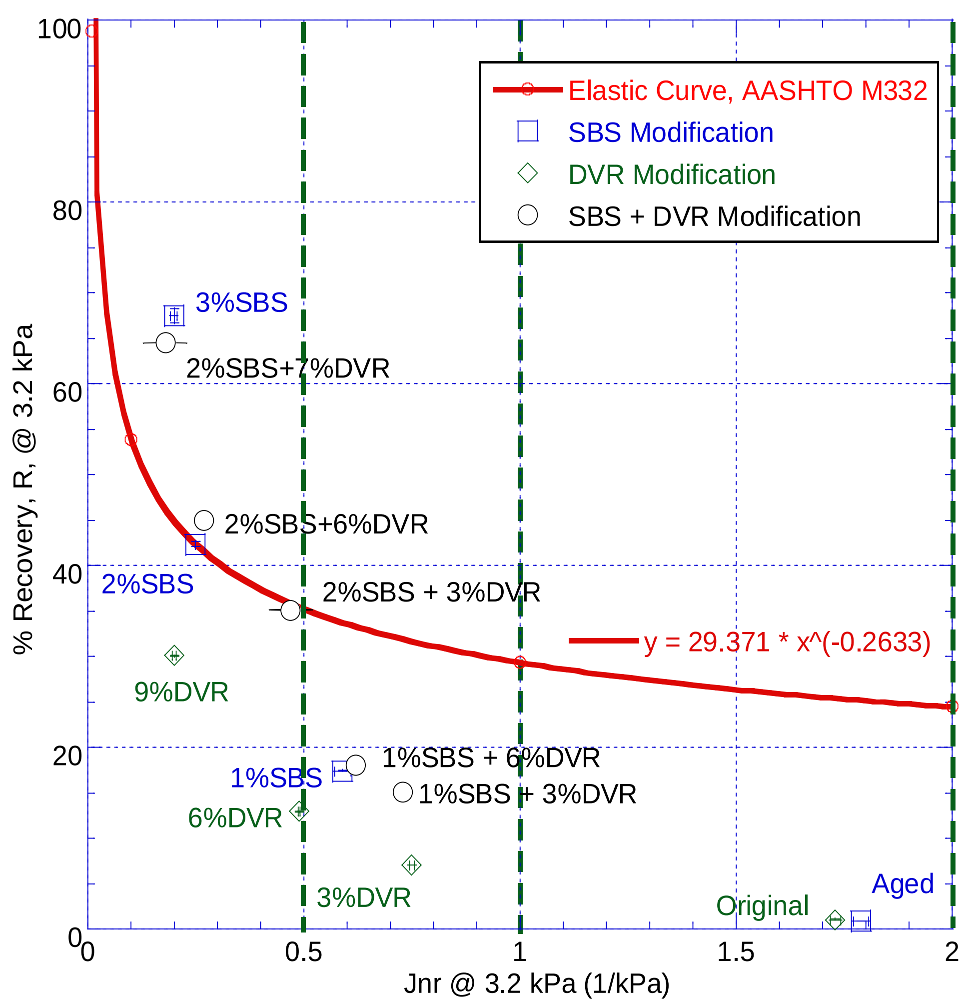

Figure 48 Nonrecoverable creep compliance versus percent recovery graph for SBS, DVR and SBS+DVR modifications

Results shown in Figure 48 were obtained from MSCR tests conducted at regional temperature (58^{o}C) according to recent AASHTO T332-14. Addition to previous combined SBS and DVR modifications, only for this part of the study 2%SBS+7% DVR modification was performed since this combination will be used for asphalt pavement test sections which will be constructed in the State of Michigan.

Although none of the individual DVR modifications were able to pass the elastic curve, combinations with 2%SBS help achieving this goal. It is clear that minimum 2%SBS is required to satisfy the elastic recovery requirement. While any individual and combinations with 2%SBS accomplished to stay above the elastic response curve, 1%SBS modification and its combinations failed to have enough elastic response. It is further worth to note that all individual and combined modifications having enough elastic response achieved “E” traffic grade as well.

AASHTO MP19-10 and M332 state that the specification is not valid for asphalt binders which have discrete particles larger than 250 micrometer in size. Since the concentric cylinder was proposed to measure the PG of CR modified asphalt binders, MSCR test for CR and SBS+CR modified binders were performed both parallel plate (PP) and concentric cylinder (CC) geometries. The results of MSCR test measurements performed with PP and CC geometries were consistent with each other.

Figure 31 shows the average $J_{nr3.2}$ and $\%R_{3.2}$ values along with standard deviations and coefficient of variations obtained from MSCR tests by using PP and CC measuring geometries for CR and SBS+CR modified asphalt binders. Since the common practice to produce rubberized asphalt rubber (RAR) uses around 20% CR with wet process, in this section of the study 20%CR modification was performed and results are provided in the data set.

Table 25 MSCR test results of CR and SBS+CR modified asphalt binders by using parallel plate (PP) and concentric cylinder (CC) geometries

Modified Binder Type	**Average $J_{nr3.2}$**	**St. Dev $J_{nr\ 3.2}$**	**Average Jnr diff**	**COV**	**Average $R_{3.2}$**	**St.Dev $R_{3.2}$**	**COV**
	1/kPa		**%**		**%**		
Original (PP)	1.73	0.013	14	0.7%	1.03	0.042	4.1%
Original (CC)	2.21	0.009	7	0.4%	1.89	0.064	3.4%
Aged (PP)	1.79	0.018	14	1.0%	0.92	0.014	1.5%
Aged (CC)	1.81	0.025	7	1.4%	2.87	0.12	4.2%
3%CR (PP)	1.1	0.002	44	0.1%	11.11	0.134	1.2%
3%CR (CC)	1.33	0.002	16	0.1%	11.39	0.007	0.1%
6%CR (PP)	0.59	0.006	46	0.9%	19.88	0.544	2.7%
6%CR (CC)	0.58	0.010	19	1.6%	26.69	0.615	2.3%
9%CR (PP)	0.29	0.003	38	1.1%	32.54	0.926	2.8%
9%CR (CC)	0.31	0.022	25	7.2%	41.88	1.732	4.1%
20%CR (PP)	0.08	0.019	369	25.7%	65.95	5.657	8.6%
20%CR (CC)	0.07	0.001	177	1.6%	68.94	0.46	0.7%
1%SBS & 3%CR (PP)	0.55	0.022	37	3.9%	21.07	0.057	0.3%
1%SBS & 3%CR (CC)	0.57	0.008	15	1.4%	26.33	0.099	0.4%
1%SBS & 6%CR (PP)	0.22	0.01	68	4.5%	54.23	0.53	1.0%
1%SBS & 6%CR (CC)	0.28	0.016	23	5.6%	46.34	0.997	2.2%
2%SBS & 3%CR (PP)	0.4	0.016	57	3.9%	38.86	0.898	2.3%
2%SBS & 3%CR (CC)	0.4	0.006	26	1.5%	44.7	0.891	2.0%
2%SBS & 6%CR (PP)	0.24	0.005	74	2.2%	40.75	0.134	0.3%
2%SBS & 6%CR (CC)	0.26	0.003	28	1.1%	54.67	0.233	0.4%

PP: Parallel plate geometry, CC: Concentric cylinder geometry, St.Dev: standard deviation

It is important to note that all %J_{nr} difference was less than specified 75% other than 20%CR modifications. Since %J_{nr} difference is the percent difference between the non-recoverable creep compliance values obtained at 0.1 kPa and 3.2 kPa for the same asphalt sample, $J_{nr0.1}$ for 20%CR modification was almost negligible and this results in higher %Jnr difference. While %Jnr difference was 369% for PP geometry, this value was obtained as 177% for CC geometry.

Figure 49 shows the relation between $J_{nr3.2}$ obtained PP and CC geometries and elastic response curve of CR and SBS+CR modified asphalt binders. Figure 49(a) demonstrates the relation between MSCR non-recoverable creep compliance at 3.2 kPa obtained from PP and CC testing geometries for CR and SBS+CR modified asphalt binders. The linear trend-line was fitted with zero intercept and without zero intercept. Both equations were provided in Figure 49. The goodness of the fit for linear regression (R^2) with a value of 0.985 was achieved for either trend-line. Analyses show that either PP or CC geometries can be used to conduct MSCR test for CR modified asphalt binders as long as it is approved by the standard specifications.

Figure 49(b) illustrates MSCR non-recoverable creep compliance versus percent recovery curve along with AASHTO M332 elastic response curve. Analyzed data was obtained from CC geometry for CR and CR+SBS modified binders since PP and CC geometries provided the very similar results. Binders fail to pass the elastic recovery requirement fall below the elastic response curve while the plotted points fall above the same curve indicates that binders were modified with required amount of elastomeric polymers. Although individual CR modifications achieve good traffic grade bumps, they were not as good to provide elastic response at 3% and 6% modifications. However combinations with SBS helped gaining elasticity and other than 1%SBS+3%CR, all other combined modifications were laid above the elastic response curve.

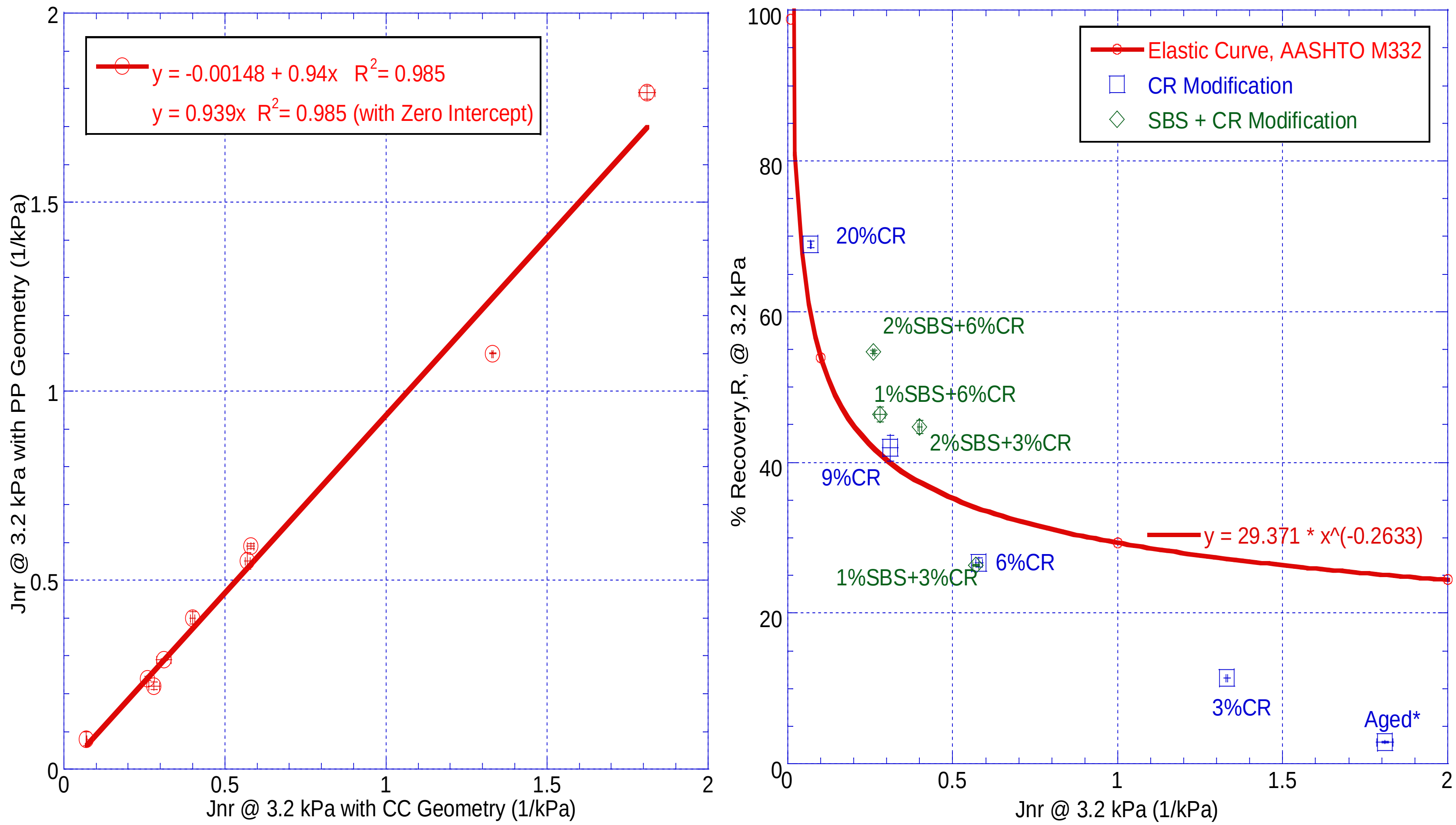

Figure 49(a) Relationship between Jnr 3.2kPa obtained from MSCR test results of CR and SBS+CR modified binders by using PP and CC measuring geometries (b) Elastic response curve (non-recoverable creep compliance versus percent recovery at 3.2 kPa) for CR and SBS+CR modified asphalt binders

5.5 CONCLUSIONS

In this chapter, the effects of binder modification with styrene-butadiene-styrene (SBS) elastomeric polymer and recycled tire rubber (RTR) at different combinations and concentration levels were investigated. RTR was used in the form of de-vulcanized rubber (DVR) and crumb rubber (CR).

First, the effect of modification on three continuous performance grades of binder was investigated. Results showed that each modification helped improving the continuous PGs compared to original asphalt binder. Combination modifications were as good as or even better than individual SBS and RTR modifications. It was observed that, in general, every 3% DVR created the effect of 1% SBS in PG testing. It was further noted that the current price of DVR is about 1/3rd of the SBS, therefore the price of 3% DVR is about the same as 1% SBS. LCY resulted in better continuous low and intermediate PGs than AT. This could be attributed to high vinyl content.

Second, fatigue resistance behavior was studied by using the Linear Amplitude Sweep (LAS) procedure at continuous intermediate PG temperatures. The ultimate outcome of the LAS test was the number of cycles to failure. LAS (fatigue) test results indicated that the SBS performed slightly better than the DVR at 1% to 3% and 2% to 6% replacement. However, 9% DVR modification outperformed all other individual and combined modifications. Individual and combined CR modifications did not perform as well as DVR modification.

Third, Multiple Stress Creep Recovery (MSCR) test was conducted for all modified asphalt binders. This test measured the percent recovery and non-recoverable creep compliance of the binders. MSCR test was suggested as a replacement of PG+ test to determine the existence of elastomeric modifiers inside the modified asphalt binders. Although individual DVR and CR modifications mainly failed to possess the required elastic recovery, combined modifications showed considerable achievements by passing above the elastic response curve. The other measurement result obtained from MSCR test was non-recoverable creep compliance. This value was used to determine the traffic grade of asphalt binders according to new binder grading specification. All modified binders achieved to have desirable results. Individual and combined DVR modifications were very close to SBS modifications. SBS and DVR replacements at any level felt within the same traffic level. CR and SBS+CR modified binders were tested by using parallel plate and concentric cylinder geometries since CC geometry was commonly used for determining the PG of CR modified asphalt binders. MSCR result indicated that either PP or CC could conveniently be used to measure the CR modified binders as long as it is approved by standard specifications. Moreover, combined and high percentage individual CR modifications (9% and up) satisfied elastic recovery requirements and achieved "E-traffic grade".

Although MSCR tests were performed at high PG and regional temperatures, test results obtained from regional temperature was mainly investigated since the asphalt binder will most probably not experience higher temperatures in that specific region. This is one of the biggest achievements made on AASHTO M332.

APPENDIX

Table 26 Continuous high PG construction of SBS LCY3710 modified asphalt binders

	LCY 3710 Original								
	1%			2%			3%		
	Batch 1 Average	**Batch 2 Average**	**Average**	**Batch 1 Average**	**Batch 2 Average**	**Average**	**Batch 1 Average**	**Batch 2 Average**	**Average**
Tc_original (°C)	63.90	64.09	63.99	67.22	66.93	67.08	72.37	71.51	71.94
T1 (°C)	58.00	64.00	**Stdev**	63.99	64.00	**Stdev**	70.00	70.00	**Stdev**
T2 (°C)	64.00	70.00	0.13	69.99	70.00	0.21	76.00	76.00	0.61
P1 (kPa)	2.040	1.010	**COV**	1.420	1.380	**COV**	1.310	1.160	**COV**
P2 (kPa)	0.988	0.503	0.21%	0.741	0.714	0.31%	0.662	0.644	0.85%
Ps (kPa)	1.00	1.00		1.00	1.00		1.00	1.00	
	LCY 3710 RTFO								
	1%			**2%**			**3%**		
	Batch 1 Average	**Batch 2 Average**	**Average**	**Batch 1 Average**	**Batch 2 Average**	**Average**	**Batch 1 Average**	**Batch 2 Average**	**Average**
Tc_rtfo (°C)	64.89	64.57	64.73	67.12	66.87	67.00	72.76	71.58	72.17
T1 (°C)	64.00	64.00	**Stdev**	64.00	64.00	**Stdev**	70.00	70.00	**Stdev**
T2 (°C)	70.00	70.00	0.23	70.00	70.00	0.18	76.00	76.00	0.83
P1 (kPa)	2.45	2.37	**COV**	3.13	3.07	**COV**	2.98	2.65	**COV**
P2 (kPa)	1.19	1.08	0.36%	1.59	1.53	0.27%	1.54	1.31	1.15%
Ps (kPa)	2.20	2.20		2.20	2.20		2.20	2.20	
Tc (°C)	63.99			67.00			71.94		

Table 27 Continuous high PG construction of DVR modified asphalt binders

	DVR Original								
	3%			6%			9%		
	Batch 1 Average	**Batch 2 Average**	**Average**	**Batch 1 Average**	**Batch 2 Average**	**Average**	**Batch 1 Average**	**Batch 2 Average**	**Average**
Tc_original (^oC)	66.35	66.08	66.22	68.82	69.11	68.96	71.75	72.83	72.29
T1 (^oC)	64.00	64.00	**Stdev**	64.01	64.00	**Stdev**	70.00	70.00	**Stdev**
T2 (^oC)	70.00	70.00	0.19	70.00	70.00	0.21	76.00	76.00	0.76
P1 (kPa)	1.320	1.280	**COV**	1.760	1.870	**COV**	1.200	1.340	**COV**
P2 (kPa)	0.650	0.629	0.28%	0.870	0.897	0.30%	0.643	0.721	1.06%
Ps (kPa)	1.00	1.00		1.00	1.00		1.00	1.00	
	DVR RTFO								
	3%			**6%**			**9%**		
	Batch 1 Average	**Batch 2 Average**	**Average**	**Batch 1 Average**	**Batch 2 Average**	**Average**	**Batch 1 Average**	**Batch 2 Average**	**Average**
Tc_rtfo (^oC)	67.11	66.92	67.01	70.08	69.77	69.93	72.38	72.16	72.27
T1 (^oC)	64.00	63.99	**Stdev**	70.00	64.00	**Stdev**	70.00	70.00	**Stdev**
T2 (^oC)	70.00	69.99	0.14	76.00	70.00	0.22	76.02	76.01	0.16
P1 (kPa)	3.23	3.17	**COV**	2.22	4.46	**COV**	2.85	2.8	**COV**
P2 (kPa)	1.54	1.5	0.20%	1.12	2.14	0.31%	1.48	1.43	0.22%
Ps (kPa)	2.20	2.20		2.20	2.20		2.20	2.20	
Tc (^oC)	66.22			68.96			72.27		

Table 28 Continuous high PG construction of CR modified asphalt binders

	CR Original								
	3%			**6%**			**9%**		
	Batch 1 Average	**Batch 2 Average**	**Average**	**Batch 1 Average**	**Batch 2 Average**	**Average**	**Batch 1 Average**	**Batch 2 Average**	**Average**
Tc_original (^oC)	64.28	63.64	63.96	68.42	68.85	68.63	73.36	71.30	72.33
T1 (^oC)	64.00	58.00	**Stdev**	63.99	64.00	**Stdev**	70.00	70.00	**Stdev**
T2 (^oC)	70.00	64.01	0.46	69.99	70.00	0.30	76.00	76.00	1.46
P1 (kPa)	1.030	1.970	**COV**	1.600	1.670	**COV**	1.710	1.150	**COV**
P2 (kPa)	0.548	0.956	0.71%	0.847	0.885	0.43%	0.656	0.603	2.01%
Ps (kPa)	1.00	1.00		1.00	1.00		1.00	1.00	
	CR RTFO								
	3%			**6%**			**9%**		
	Batch 1 Average	**Batch 2 Average**	**Average**	**Batch 1 Average**	**Batch 2 Average**	**Average**	**Batch 1 Average**	**Batch 2 Average**	**Average**
Tc_rtfo (^oC)	64.75	65.00	64.87	69.43	69.23	69.33	74.09	74.53	74.31
T1 (^oC)	64.00	64.00	**Stdev**	63.99	64.00	**Stdev**	70.00	70.00	**Stdev**
T2 (^oC)	70.00	70.00	0.17	69.99	70.00	0.15	76.00	76.00	0.31
P1 (kPa)	2.4	2.47	**COV**	3.99	4.05	**COV**	3.34	3.46	**COV**
P2 (kPa)	1.2	1.23	0.26%	2.07	2.01	0.21%	1.81	1.9	0.42%
Ps (kPa)	2.20	2.20		2.20	2.20		2.20	2.20	
Tc (^oC)	63.96			68.63			72.33		

Table 29 Continuous high PG construction of D1101AT + DVR modified asphalt binders

	D1101AT + DVR Original											
	1%SBS+3%DVR			1%SBS+6%DVR			2%SBS+3%DVR			2%SBS+6%DVR		
	Batch 1 Av	**Batch 2 Av**	**Av**	**Batch 1 Av**	**Batch 2 Av**	**Av**	**Batch 1 Av**	**Batch 2 Av**	**Av**	**Batch 1 Av**	**Batch 2 Av**	**Av**
Tc_original (^{o}C)	66.47	66.69	66.58	67.94	68.02	67.98	69.70	69.11	69.40	71.53	71.22	71.37
T1 (^{o}C)	64.00	64.00	**Stdev**	64.00	64.00	**Stdev**	64.00	64.00	**Stdev**	70.03	70.00	**Stdev**
T2 (^{o}C)	70.00	70.00	0.15	70.00	70.00	0.06	70.00	70.00	0.41	76.03	76.00	0.22
P1 (kPa)	1.330	1.360	**COV**	1.550	1.560	**COV**	1.670	1.720	**COV**	1.160	1.130	**COV**
P2 (kPa)	0.666	0.685	0.23%	0.795	0.803	0.08%	0.973	0.910	0.60%	0.641	0.619	0.31%
Ps (kPa)	1.00	1.00		1.00	1.00		1.00	1.00		1.00	1.00	
	D1101AT + DVR RTFO											
	1%SBS+3%DVR			1%SBS+6%DVR			2%SBS+3%DVR			2%SBS+6%DVR		
	Batch 1 Av	**Batch 2 Av**	**Av**	**Batch 1 Av**	**Batch 2 Av**	**Av**	**Batch 1 Av**	**Batch 2 Av**	**Av**	**Batch 1 Av**	**Batch 2 Av**	**Av**
Tc_rtfo (^{o}C)	67.97	67.85	67.91	69.04	69.17	69.11	69.26	69.39	69.33	71.48	71.05	71.27
T1 (^{o}C)	64.00	64.00	**Stdev**	64.00	64.00	**Stdev**	64.00	63.99	**Stdev**	70.00	70.00	**Stdev**
T2 (^{o}C)	70.00	70.00	0.08	70.00	70.00	0.09	70.00	69.99	0.09	76.00	76.00	0.31
P1 (kPa)	3.520	3.460	**COV**	3.940	3.990	**COV**	3.910	3.980	**COV**	2.570	2.460	**COV**
P2 (kPa)	1.730	1.710	0.12%	1.970	2.000	0.13%	2.030	2.060	0.13%	1.370	1.300	0.43%
Ps (kPa)	2.20	2.20		2.20	2.20		2.20	2.20		2.20	2.20	
Tc (^{o}C)	66.58			67.98			69.33			71.27		

Table 30 Continuous high PG construction of D1101AT + CR modified asphalt binders

	D1101AT + CR Original											
	1%SBS+3%CR			1%SBS+6%CR			2%SBS+3%CR			2%SBS+6%CR		
	Batch 1 Av	**Batch 2 Av**	**Av**	**Batch 1 Av**	**Batch 2 Av**	**Av**	**Batch 1 Av**	**Batch 2 Av**	**Av**	**Batch 1 Av**	**Batch 2 Av**	**Av**
Tc_original (°C)	69.92	69.86	69.89	73.72	74.07	73.89	70.97	69.99	70.48	74.68	74.85	74.76
T1 (°C)	64.00	64.00	**Stdev**	70.00	70.00	**Stdev**	70.00	69.99	**Stdev**	70.00	69.99	**Stdev**
T2 (°C)	70.00	70.00	0.04	76.00	76.00	0.25	76.00	75.99	0.69	76.00	75.99	0.12
P1 (kPa)	1.950	1.910	**COV**	1.430	1.510	**COV**	1.100	1.000	**COV**	1.500	1.550	**COV**
P2 (kPa)	0.991	0.985	0.06%	0.803	0.822	0.33%	0.611	0.553	0.99%	0.892	0.902	0.16%
Ps (kPa)	1.00	1.00		1.00	1.00		1.00	1.00		1.00	1.00	
	D1101AT + CR RTFO											
	1%SBS+3%CR			**1%SBS+6%CR**			**2%SBS+3%CR**			**2%SBS+6%CR**		
	Batch 1 Av	**Batch 2 Av**	**Av**	**Batch 1 Av**	**Batch 2 Av**	**Av**	**Batch 1 Av**	**Batch 2 Av**	**Av**	**Batch 1 Av**	**Batch 2 Av**	**Av**
Tc_rtfo (°C)	70.51	70.16	70.33	73.59	73.86	73.72	70.17	70.34	70.26	74.52	74.89	74.71
T1 (°C)	70.00	70.00	**Stdev**	70.00	69.99	**Stdev**	70.00	70.00	**Stdev**	70.00	69.99	**Stdev**
T2 (°C)	76.00	76.00	0.24	76.00	75.98	0.19	76.01	76.00	0.12	75.99	75.99	0.26
P1 (kPa)	2.330	2.240	**COV**	3.090	3.170	**COV**	2.240	2.280	**COV**	3.460	3.600	**COV**
P2 (kPa)	1.180	1.140	0.35%	1.750	1.800	0.26%	1.200	1.220	0.17%	1.900	1.970	0.35%
Ps (kPa)	2.20	2.20		2.20	2.20		2.20	2.20		2.20	2.20	
Tc (°C)	69.89			73.72			70.26			74.71		

Table 31 Continuous high PG construction of LCY3710 + DVR & LCY3710 + CR modified asphalt binders

	LCY3710 + DVR Original & LCY3710 + CR Original											
	1%SBS+3%DVR			2%SBS+6%DVR			1%SBS+3%CR			2%SBS+6%CR		
	Batch 1 Av	Batch 2 Av	Av	Batch 1 Av	Batch 2 Av	Av	Batch 1 Av	Batch 2 Av	Av	Batch 1 Av	Batch 2 Av	Av
Tc_original (°C)	65.76	65.94	65.85	70.67	71.01	70.84	66.56	66.29	66.42	74.89	73.75	74.32
T1 (°C)	63.99	63.99	**Stdev**	70.00	70.00	**Stdev**	64.00	64.00	**Stdev**	70.00	70.00	**Stdev**
T2 (°C)	69.99	69.99	0.12	76.00	76.00	0.24	70.00	70.00	0.20	76.00	76.00	0.81
P1 (kPa)	1.220	1.250	**COV**	1.070	1.110	**COV**	1.320	1.290	**COV**	1.550	1.420	**COV**
P2 (kPa)	0.622	0.628	0.19%	0.582	0.598	0.35%	0.689	0.661	0.30%	0.905	0.810	1.08%
Ps (kPa)	1.00	1.00		1.00	1.00		1.00	1.00		1.00	1.00	
	LCY3710 + DVR RTFO & LCY3710 + CR RTFO											
	1%SBS+3%DVR			2%SBS+6%DVR			1%SBS+3%CR			2%SBS+6%CR		
	Batch 1 Av	Batch 2 Av	Av	Batch 1 Av	Batch 2 Av	Av	Batch 1 Av	Batch 2 Av	Av	Batch 1 Av	Batch 2 Av	Av
Tc_rtfo (°C)	66.72	66.54	66.63	71.58	71.84	71.71	67.47	67.37	67.42	72.71	73.22	72.96
T1 (°C)	64.00	64.00	**Stdev**	70.00	70.00	**Stdev**	64.00	64.00	**Stdev**	70.00	69.99	**Stdev**
T2 (°C)	70.00	70.00	0.12	76.00	76.00	0.18	70.00	70.00	0.07	76.00	75.99	0.36
P1 (kPa)	3.020	2.960	**COV**	2.600	2.670	**COV**	3.240	3.180	**COV**	2.860	2.990	**COV**
P2 (kPa)	1.500	1.470	0.18%	1.380	1.420	0.25%	1.660	1.650	0.11%	1.600	1.690	0.49%
Ps (kPa)	2.20	2.20		2.20	2.20		2.20	2.20		2.20	220.00%	
Tc (°C)	65.85			70.84			66.42			72.96		

Table 32 Continuous high PG construction of original and Aged* asphalt binders

	M58-28 Original Binder					
	Original binder			**Aged**		
	Batch 1 Average	**Batch 2 Average**	**Average**	**Batch 1 Average**	**Batch 2 Average**	**Average**
Tc_original (°C)	60.09	60.16	60.13	61.92	61.79	61.86
T1 (°C)	58.00	58.00	**Stdev**	58.00	58.00	**Stdev**
T2 (°C)	64.00	64.00	0.05	64.00	64.00	0.09
P1 (kPa)	1.300	1.310	**COV**	1.650	1.630	**COV**
P2 (kPa)	0.612	0.619	0.08%	0.767	0.752	0.15%
Ps (kPa)	1.00	1.00		1.00	1.00	
	M58-28 Original Binder RTFO					
	Original Binder			**Aged**		
	Batch 1 Average	**Batch 2 Average**	**Average**	**Batch 1 Average**	**Batch 2 Average**	**Average**
Tc_rtfo (°C)	61.12	61.12	61.12	62.61	62.47	62.54
T1 (°C)	58.00	58.00	**Stdev**	58.00	58.00	**Stdev**
T2 (°C)	64.00	64.00	0.00	64.00	64.00	0.10
P1 (kPa)	3.330	3.310	**COV**	4.050	3.950	**COV**
P2 (kPa)	1.500	1.510	0.00%	1.830	1.800	0.16%
Ps (kPa)	2.20	2.20		2.20	2.20	
Tc (°C)	60.13			61.86		

Table 33 Continuous intermediate PG construction of DVR & CR modified asphalt binders

	Devulcanized Rubber (DVR)								
	3%			6%			9%		
	Batch 1 Average	**Batch 2 Average**	**Average**	**Batch 1 Average**	**Batch 2 Average**	**Average**	**Batch 1 Average**	**Batch 2 Average**	**Average**
Tc (°C)	16.41	16.69	16.55	16.52	16.26	16.39	16.56	16.49	16.52
T1 (°C)	19.00	19.00	**Stdev**	19.00	19.00	**Stdev**	19.00	19.00	**Stdev**
T2 (°C)	16.00	16.00	0.20	16.00	16.00	0.18	16.00	16.00	0.05
P1 (kPa)	3085.8	3385.3	**COV**	3246.8	3127.1	**COV**	3240.3	3309.1	**COV**
P2 (kPa)	5391.9	5621.5	1.23%	5468.3	5228.2	1.10%	5519.8	5419.2	0.29%
Ps (kPa)	5000.0	5000.0		5000.0	5000.0		5000.0	5000.0	
	Crumb Rubber (CR)								
	3%			**6%**			**9%**		
	Batch 1 Average	**Batch 2 Average**	**Average**	**Batch 1 Average**	**Batch 2 Average**	**Average**	**Batch 1 Average**	**Batch 2 Average**	**Average**
Tc (°C)	15.06	15.61	15.34	14.20	14.14	14.17	14.87	14.30	14.58
T1 (°C)	16.00	16.00	**Stdev**	16.00	16.00	**Stdev**	16.00	16.00	**Stdev**
T2 (°C)	13.00	13.00	0.39	13.00	13.00	0.04	13.00	13.00	0.41
P1 (kPa)	4352.2	4773.0	**COV**	3740.8	3672.8	**COV**	4125.6	3855.8	**COV**
P2 (kPa)	6778.4	6838.7	2.54%	6074.8	6048.1	0.30%	6879.6	6091.3	2.80%
Ps (kPa)	5000.0	5000.0		5000.0	5000.0		5000.0	5000.0	

Table 34 Continuous intermediate PG construction of D1101AT + DVR & D1101AT + CR modified asphalt binders

	D1101AT + DVR											
	1%SBS+3%DVR			**1%SBS+6%DVR**			**2%SBS+3%DVR**			**2%SBS+6%DVR**		
	Batch 1 Av	**Batch 2 Av**	**Av**	**Batch 1 Av**	**Batch 2 Av**	**Av**	**Batch 1 Av**	**Batch 2 Av**	**Av**	**Batch 1 Av**	**Batch 2 Av**	**Av**
Tc (^{o}C)	16.28	15.70	15.99	15.33	15.05	15.19	14.68	14.63	14.66	14.70	14.78	14.74
T1 (^{o}C)	19.00	19.00	**Stdev**	16.00	16.00	**Stdev**	16.00	16.00	**Stdev**	16.00	16.00	**Stdev**
T2 (^{o}C)	16.00	16.00	0.41	13.00	13.00	0.19	13.00	13.00	0.03	13.00	13.00	0.06
P1 (kPa)	3184.3	2968.3	**COV**	4582.2	4304.9	**COV**	3877.3	4041.5	**COV**	4112.9	4152.7	**COV**
P2 (kPa)	5240.3	4767.3	2.58%	6767.5	6918.4	1.28%	6897.1	6451.1	0.20%	6456.9	6559.2	0.39%
Ps (kPa)	5000.0	5000.0		5000.0	5000.0		5000.0	5000.0		5000.0	5000.0	

	D1101AT + CR											
	1%SBS+3%CR			**1%SBS+6%CR**			**2%SBS+3%CR**			**2%SBS+6%CR**		
	Batch 1 Av	**Batch 2 Av**	**Av**	**Batch 1 Av**	**Batch 2 Av**	**Av**	**Batch 1 Av**	**Batch 2 Av**	**Av**	**Batch 1 Av**	**Batch 2 Av**	**Av**
Tc (^{o}C)	16.78	16.26	16.52	13.76	13.94	13.85	15.37	15.05	15.21	14.14	14.71	14.42
T1 (^{o}C)	19.00	19.00	**Stdev**	16.00	16.00	**Stdev**	16.00	16.00	**Stdev**	16.00	16.00	**Stdev**
T2 (^{o}C)	16.00	16.00	0.36	13.00	13.00	0.13	13.00	13.00	0.22	13.00	13.00	0.40
P1 (kPa)	3560.4	3325.1	**COV**	3483.0	3680.3	**COV**	4609.0	4425.1	**COV**	3725.9	4123.5	**COV**
P2 (kPa)	5627.8	5197.7	2.20%	5647.1	5754.8	0.96%	6783.7	6514.4	1.47%	5989.1	6452.7	2.78%
Ps (kPa)	5000.0	5000.0		5000.0	5000.0		5000.0	5000.0		5000.0	5000.0	

Table 35 Continuous intermediate PG construction of LCY3710+DVR, LCY3710+CR modified & original and aged* asphalt binders

	LCY3710+DVR & LCY3710+CR											
	1%SBS+3%DVR			2%SBS+6%DVR			1%SBS+3%CR			2%SBS+6%CR		
	Batch 1 Av	**Batch 2 Av**	**Av**	**Batch 1 Av**	**Batch 2 Av**	**Av**	**Batch 1 Av**	**Batch 2 Av**	**Av**	**Batch 1 Av**	**Batch 2 Av**	**Av**
Tc (^{o}C)	16.63	16.66	16.64	15.94	15.93	15.93	16.38	16.07	16.23	14.04	13.99	14.02
T1 (^{o}C)	19.00	19.00	**Stdev**	19.00	19.00	**Stdev**	19.00	19.00	**Stdev**	16.00	16.00	**Stdev**
T2 (^{o}C)	16.00	16.00	0.02	16.00	16.00	0.01	16.00	16.00	0.22	13.00	13.00	0.04
P1 (kPa)	3434.5	3467.4	**COV**	3075.3	3059.6	**COV**	3215.1	3163.2	**COV**	3937.6	3723.8	**COV**
P2 (kPa)	5522.2	5543.8	0.14%	4952.3	4942.6	0.05%	5335.4	5057.6	1.36%	5677.7	5782.9	0.25%
Ps (kPa)	5000.0	5000.0		5000.0	5000.0		5000.0	5000.0		5000.0	5000.0	
	M58-28 Original & M58-28 Aged											
				Original Binder			**Aged Binder**					
				Batch 1 Av	**Batch 2 Av**	**Av**	**Batch 1 Av**	**Batch 2 Av**	**Av**			
Tc (^{o}C)				16.91	17.06	16.99	17.80	18.06	17.93			
T1 (^{o}C)				19.00	19.00	**Stdev**	19.00	19.00	**Stdev**			
T2 (^{o}C)				16.00	16.00	0.10	16.00	16.00	0.19			
P1 (kPa)				3533.3	3656.9	**COV**	4191.8	4323.7	**COV**			
P2 (kPa)				5822.6	5925.7	0.59%	6512.0	6885.7	1.04%			
Ps (kPa)				5000.0	5000.0		5000.0	5000.0				

Table 36 Continuous low PG construction of SBS LCY3710 modified asphalt binders

	LCY 3710 Stiffness-based								
	1%			**2%**			**3%**		
	Batch 1 Average	**Batch 2 Average**	**Average**	**Batch 1 Average**	**Batch 2 Average**	**Average**	**Batch 1 Average**	**Batch 2 Average**	**Average**
Tc stiffness (^oC)	-19.63	-19.19	-19.41	-19.79	-19.41	-19.60	-19.9	-20.1	-20.03
T1 (^oC)	-18.00	-18.00	**Stdev**	-18.00	-18.00	**Stdev**	-18.0	-18.0	**Stdev**
T2 (^oC)	-24	-24	0.31	-24	-24	0.26	-24	-24	0.12
P1 (kPa)	245	261	**COV**	241	253	**COV**	235	228	**COV**
P2 (kPa)	517	527	1.60%	503	522	1.35%	500	497	0.61%
Ps (kPa)	300	300		300	300		300	300	
	LCY 3710 m_value Based								
	1%			**2%**			**3%**		
	Batch 1 Average	**Batch 2 Average**	**Average**	**Batch 1 Average**	**Batch 2 Average**	**Average**	**Batch 1 Average**	**Batch 2 Average**	**Average**
Tc m-value (^oC)	-18.53	-19.00	-18.77	-20.53	-21.14	-20.83	-21.2	-21.5	-21.37
T1 (^oC)	-18.00	-18.00	**Stdev**	-18.00	-18.00	**Stdev**	-18.0	-18.0	**Stdev**
T2 (^oC)	-24	-24	0.33	-24	-24	0.44	-24	-24	0.18
P1 (kPa)	0.304	0.307	**COV**	0.324	0.333	**COV**	0.333	0.335	**COV**
P2 (kPa)	0.259	0.265	1.76%	0.267	0.27	2.09%	0.272	0.275	0.84%
Ps (kPa)	0.3	0.3		0.3	0.3		0.3	0.3	
Tc (^oC)	-18.77			-19.60			-20.03		

Table 37 Continuous low PG construction of DVR modified asphalt binders

	DVR Stiffness-based								
	3%			6%			9%		
	Batch 1 Average	Batch 2 Average	Average	Batch 1 Average	Batch 2 Average	Average	Batch 1 Average	Batch 2 Average	Average
Tc stiffness (^oC)	-18.81	-18.61	-18.71	-19.03	-19.02	-19.03	-19.7	-19.6	-19.66
T1 (^oC)	-18.00	-18.00	**Stdev**	-18.00	-18.00	**Stdev**	-18.0	-18.0	**Stdev**
T2 (^oC)	-24	-24	0.14	-24	-24	0.01	-24	-24	0.07
P1 (kPa)	268	276	**COV**	262	263	**COV**	238	240	**COV**
P2 (kPa)	616	625	0.76%	575	572	0.06%	537	550	0.33%
Ps (kPa)	300	300		300	300		300	300	
	DVR m_value-based								
	3%			**6%**			**9%**		
	Batch 1 Average	**Batch 2 Average**	**Average**	**Batch 1 Average**	**Batch 2 Average**	**Average**	**Batch 1 Average**	**Batch 2 Average**	**Average**
Tc m-value (^oC)	-18.77	-19.00	-18.88	-17.83	-18.41	-18.12	-18.6	-18.6	-18.60
T1 (^oC)	-18.00	-18.00	**Stdev**	-12.00	-18.00	**Stdev**	-18.0	-18.0	**Stdev**
T2 (^oC)	-24	-24	0.17	-18	-24	0.41	-24	-24	0.06
P1 (kPa)	0.306	0.308	**COV**	0.37	0.303	**COV**	0.304	0.306	**COV**
P2 (kPa)	0.259	0.26	0.88%	0.298	0.259	2.25%	0.261	0.25	0.32%
Ps (kPa)	0.3	0.3		0.3	0.3		0.3	0.3	
Tc (^oC)	-18.71			-18.12			-18.60		

Table 38 Continuous low PG construction of CR modified asphalt binders

	CR Stiffness-based								
	3%			6%			9%		
	Batch 1 Average	Batch 2 Average	Average	Batch 1 Average	Batch 2 Average	Average	Batch 1 Average	Batch 2 Average	Average
Tc stiffness (^oC)	-20.03	-19.91	-19.97	-21.45	-21.15	-21.30	-22.1	-22.7	-22.40
T1 (^oC)	-18.00	-18.00	**Stdev**	-18.00	-18.00	**Stdev**	-18.0	-18.0	**Stdev**
T2 (^oC)	-24	-24	0.08	-24	-24	0.21	-24	-24	0.39
P1 (kPa)	231	233	**COV**	191	199	**COV**	195	180	**COV**
P2 (kPa)	501	515	0.40%	419	435	0.99%	365	347	1.73%
Ps (kPa)	300	300		300	300		300	300	
	CR m_value-based								
	3%			**6%**			**9%**		
	Batch 1 Average	**Batch 2 Average**	**Average**	**Batch 1 Average**	**Batch 2 Average**	**Average**	**Batch 1 Average**	**Batch 2 Average**	**Average**
Tc m-value (^oC)	-20.65	-20.25	-20.45	-20.57	-20.75	-20.66	-19.8	-20.7	-20.24
T1 (^oC)	-18.00	-18.00	**Stdev**	-18.00	-18.00	**Stdev**	-18.0	-18.0	**Stdev**
T2 (^oC)	-24	-24	0.29	-24	-24	0.13	-24	-24	0.66
P1 (kPa)	0.323	0.318	**COV**	0.321	0.322	**COV**	0.313	0.318	**COV**
P2 (kPa)	0.271	0.27	1.40%	0.272	0.274	0.61%	0.269	0.278	3.24%
Ps (kPa)	0.3	0.3		0.3	0.3		0.3	0.3	
Tc (^oC)	-19.97			-20.66			-20.24		

Table 39 Continuous low PG construction of D1101AT + DVR modified asphalt binders

	D1101AT + DVR _Stiffness-based											
	1%SBS+3%DVR			1%SBS+6%DVR			2%SBS+3%DVR			2%SBS+6%DVR		
	Batch 1 Av	**Batch 2 Av**	**Av**	**Batch 1 Av**	**Batch 2 Av**	**Av**	**Batch 1 Av**	**Batch 2 Av**	**Av**	**Batch 1 Av**	**Batch 2 Av**	**Av**
Tc stiffness (^oC)	-19.30	-19.23	-19.26	-20.18	-19.90	-20.04	-20.51	-20.44	-20.47	-20.56	-20.29	-20.42
T1 (^oC)	-18.00	-18.00	**Stdev**	-18.00	-18.00	**Stdev**	-18.00	-18.00	**Stdev**	-18.00	-18.00	**Stdev**
T2 (^oC)	-24	-24	0.05	-24	-24	0.20	-24	-24	0.05	-24	-24	0.19
P1 (kPa)	260	262	**COV**	234	244	**COV**	227	229	**COV**	222	230	**COV**
P2 (kPa)	504	507	0.24%	464	469	0.99%	442	445	0.25%	450	461	0.91%
Ps (kPa)	300	300		300	300		300	300		300	300	
	D1101AT + DVR m_value-based											
	1%SBS+3%DVR			**1%SBS+6%DVR**			**2%SBS+3%DVR**			**2%SBS+6%DVR**		
	Batch 1 Av	**Batch 2 Av**	**Av**	**Batch 1 Av**	**Batch 2 Av**	**Av**	**Batch 1 Av**	**Batch 2 Av**	**Av**	**Batch 1 Av**	**Batch 2 Av**	**Av**
Tc m-value (^oC)	-19.43	-19.31	-19.37	-20.10	-20.94	-20.52	-20.79	-20.61	-20.70	-19.49	-20.87	-20.18
T1 (^oC)	-18.00	-18.00	**Stdev**	-18.00	-18.00	**Stdev**	-18.00	-18.00	**Stdev**	-18.00	-18.00	**Stdev**
T2 (^oC)	-24	-24	0.08	-24	-24	0.60	-24	-24	0.13	-24	-24	0.97
P1 (kPa)	0.316	0.314	**COV**	0.322	0.332	**COV**	0.331	0.333	**COV**	0.313	0.326	**COV**
P2 (kPa)	0.254	0.255	0.41%	0.263	0.27	2.91%	0.268	0.262	0.63%	0.264	0.274	4.82%
Ps (kPa)	0.3	0.3		0.3	0.3		0.3	0.3		0.3	0.3	
Tc (^oC)	-19.26			-20.04			-20.47			-20.18		

Table 40 Continuous low PG construction of D1101AT + CR modified asphalt binders

	D1101AT + CR _Stiffness-based											
	1%SBS+3%CR			1%SBS+6%CR			2%SBS+3%CR			2%SBS+6%CR		
	Batch 1 Av	**Batch 2 Av**	**Av**	**Batch 1 Av**	**Batch 2 Av**	**Av**	**Batch 1 Av**	**Batch 2 Av**	**Av**	**Batch 1 Av**	**Batch 2 Av**	**Av**
Tc stiffness (^{o}C)	-19.25	-19.17	-19.21	-21.40	-21.64	-21.52	-20.81	-20.52	-20.67	-21.50	-21.10	-21.30
T1 (^{o}C)	-18.00	-18.00	**Stdev**	-18.00	-18.00	**Stdev**	-18.00	-18.00	**Stdev**	-18.00	-18.00	**Stdev**
T2 (^{o}C)	-24	-24	0.06	-24	-24	0.17	-24	-24	0.21	-24	-24	0.28
P1 (kPa)	266	268	**COV**	210	200	**COV**	213	222	**COV**	196	209	**COV**
P2 (kPa)	473	477	0.29%	394	390	0.79%	442	455	1.02%	407	421	1.32%
Ps (kPa)	300	300		300	300		300	300		300	300	
	D1101AT + CR_m-based											
	1%SBS+3%CR			**1%SBS+6%CR**			**2%SBS+3%CR**			**2%SBS+6%CR**		
	Batch 1 Av	**Batch 2 Av**	**Av**	**Batch 1 Av**	**Batch 2 Av**	**Av**	**Batch 1 Av**	**Batch 2 Av**	**Av**	**Batch 1 Av**	**Batch 2 Av**	**Av**
Tc m-value (^{o}C)	-18.13	-18.31	-18.22	-18.49	-18.83	-18.66	-20.49	-20.90	-20.70	-18.94	-19.35	-19.15
T1 (^{o}C)	-18.00	-18.00	**Stdev**	-18.00	-18.00	**Stdev**	-18.00	-18.00	**Stdev**	-18.00	-18.00	**Stdev**
T2 (^{o}C)	-24	-24	0.13	-24	-24	0.24	-24	-24	0.29	-24	-24	0.29
P1 (kPa)	0.301	0.303	**COV**	0.303	0.307	**COV**	0.325	0.330	**COV**	0.307	0.309	**COV**
P2 (kPa)	0.259	0.25	0.69%	0.268	0.260	1.31%	0.268	0.271	1.41%	0.265	0.271	1.52%
Ps (kPa)	0.3	0.3		0.3	0.3		0.3	0.3		0.3	0.3	
Tc (^{o}C)	-18.22			-18.66			-20.67			-19.15		

Table 41 Continuous low PG construction of LCY3710+ DVR & LCY3710+ CR modified asphalt binders

	LCY3710 + DVR _Stiffness-based and m_value- based											
	1%SBS+3%DVR			**1%SBS+3%DVR**			**2%SBS+6%DVR**			**2%SBS+6%DVR**		
	Batch 1 Av	**Batch 2 Av**	**Av**	**Batch 1 Av**	**Batch 2 Av**	**Av**	**Batch 1 Av**	**Batch 2 Av**	**Av**	**Batch 1 Av**	**Batch 2 Av**	**Av**
Tc stiffness (^oC)	-19.44	-19.49	-19.47	-18.85	-19.02	-18.93	-21.00	-20.99	-20.99	-18.00	-18.16	-18.08
T1 (^oC)	-18.00	-18.00	**Stdev**	-18.00	-18.00	**Stdev**	-18.00	-18.00	**Stdev**	-18.00	-18.00	**Stdev**
T2 (^oC)	-24	-24	0.04	-24	-24	0.12	-24	-24	0.01	-24	-24	0.11
P1 (kPa)	252	250	**COV**	0.306	0.307	**COV**	208	208	**COV**	0.3	0.301	**COV**
P2 (kPa)	522	520	0.21%	0.266	0.268	0.64%	433	434	0.03%	0.257	0.265	0.61%
Ps (kPa)	300	300		0.3	0.3		300	300		0.3	0.3	
	-18.93						-18.08					
	LCY3710 + CR_Stiffness-based and m_value- based											
	1%SBS+3%CR			**1%SBS+3%CR**			**2%SBS+6%CR**			**2%SBS+6%CR**		
Tc m-value (^oC)	**Batch 1 Av**	**Batch 2 Av**	**Av**	**Batch 1 Av**	**Batch 2 Av**	**Av**	**Batch 1 Av**	**Batch 2 Av**	**Av**	**Batch 1 Av**	**Batch 2 Av**	**Av**
T1 (^oC)	-19.79	-19.81	-19.80	-18.23	-18.32	-18.27	-21.78	-21.68	-21.73	-18.32	-18.45	-18.38
T2 (^oC)	-18	-18	**Stdev**	-18	-18	**Stdev**	-18	-18	**Stdev**	-18	-18	**Stdev**
P1 (kPa)	-24	-24	0.0	-24	-24	0.07	-24	-24	0.07	-24	-24	0.09
P2 (kPa)	241	240	**COV**	0.306	0.302	**COV**	187	190	**COV**	0.302	0.303	**COV**
Ps (kPa)	503	502	0.10%	0.181	0.267	0.38%	396	400	0.32%	0.267	0.265	0.47%
Tc (^oC)	300	300		0.3	0.3		300	300		0.3	0.3	
Tc	-18.27						-18.38					

Table 42 Continuous low PG construction of original and Aged* asphalt binders

	M58-28 Original Binder											
	Original binder Stiffness-based			Original binder m-based			Aged Binder Stiffness-based			Aged Binder m -based		
	Batch 1 Av	**Batch 2 Av**	**Av**	**Batch 1 Av**	**Batch 2 Av**	**Average**	**Batch 1 Av**	**Batch 2 Av**	**Av**	**Batch 1 Av**	**Batch 2 Av**	**Average**
Tc_S&m- based	-19.02	-19.12	-19.07	-20.81	-20.32	-20.57	-18.35	-18.16	-18.25	-16.89	-16.85	-16.87
T1 (^oC)	-18.00	-18.00	**Stdev**	-18.00	-18.00	**Stdev**	-18.00	-18.00	**Stdev**	-18.00	-18.00	**Stdev**
T2 (^oC)	-24	-24	0.07	-24	-24	0.34	-24	-24	0.13	-24	-24	0.03
P1 (kPa)	264	260	**COV**	0.338	0.33	**COV**	287	294	**COV**	0.291	0.29	**COV**
P2 (kPa)	562	560	0.39%	0.262	0.258	1.67%	619	630	0.72%	0.247	0.243	0.15%
Ps (kPa)	300	300		0.3	0.3		300	300		0.3	0.3	
Tc (^oC)	-19.07						-16.87					

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CHAPTER 6 CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE WORK

6.1 SUMMARY

The scope of research work included investigation of interaction between recycled tire rubber, polymers and high amount reclaimed asphalt pavements by studying the binder rheology and hot mix asphalt performance. Recycled tire used was in the form of ground tire rubber (a.k.a. crumb rubber) and devulcanized rubber. This dissertation is organized in the format of articles which summarize the main findings of the doctoral study. This last section reinstates the conclusions found in the chapters of the dissertation and recommends areas where future research is needed.

6.2 CONCLUSIONS

The use of crumb rubber in asphalt pavement could be increased by using hybrid crumb rubber modified asphalt pavements without adversely affecting the asphalt pavement performance. By adding about 0.5% CR by weight of the mixture via dry process, the CR amount was doubled in CRHY mixtures compared to traditional CRTB mixtures. The performance tests conducted on both asphalt mixtures included rutting susceptibility, moisture damage, thermal cracking and fatigue resistance. Results indicated that both laboratory and field collected CRHY mixtures performed as well as CRTB mixtures at low temperature cracking and rutting susceptibility and better at fatigue cracking and moisture damage.

In the study, to investigate the feasibility of substituting crumb rubber modified asphalt mixture in lieu of grade bumped asphalt binders in mixtures that contain high percentage of RAP, the following conclusions were drawn:

- Linear viscoelastic characterization of the mixtures showed that at high temperatures/ low frequencies, CRTB mixtures became the stiffest one which was followed by Control and CRWET mixtures. CRDEV and PG58-34 possessed very similar stiffness values which were lower than rest of the mixtures. At low temperatures/ high frequencies, CRTB and Control maintained their place being the stiffest first two mixtures. Others had less stiffness values.
- Mixtures prepared by using CR modified asphalt binders performed as well or better than mixtures prepared with grade bumped PG 58-34 binder in terms of low temperature cracking susceptibility. This is an indication that CR modified binders may be used in lieu of grade bumping in colder climates.
- Fatigue cracking resistance of mixtures prepared by using rubberized binders generally provided better results than mixtures made with Control and PG58-34 binders. CRWET outperformed all other mixtures.

In the binder modifications study by using recycled tire rubber and elastomeric polymer SBS at individual and combined modifications, continuous-high,-intermediate, and -low performance grades, fatigue resistance, rutting susceptibility of the modified binders were studied. The following conclusions were drawn from this study:

- SBS with less vinyl content (AT) achieved better continuous high PGs for individual and combined modifications than SBS with high vinyl amount (LCY).
- While individual LCY modifications resulted in favourable continuous intermediate PGs, the difference between AT and LCY combined modifications were insignificant. This

conclusion also applies for continuous low PGs of individual and combined AT, LCY and RTR modifications.

- It was observed that every 3%DVR created the effect of 1%SBS in PG testing. Furthermore, the price of DVR is about 1/3rd of the SBS in current US market.
- Individual DVR and CR modifications accomplished similar or better results than SBS modifications at continuous PG tests.
- In general, combined modifications achieved similar continuous PGs compared to individual SBS modifications and better continuous PGs compared to individual RTR modifications.
- Fatigue resistance behaviour of individual SBS and DVR modifications were very similar. LAS results of DVR modification at 9% outperformed all other individual and combined modifications.
- Fatigue resistance of individual and combined DVR modifications performed better than corresponding CR modifications.
- Combined modifications of SBS and DVR did not perform better than individual modification whereas combined SBS and CR modifications enhanced the fatigue life of individual CR modifications.
- Even though MSCR test was performed at local and high PG temperatures, the main focus was given to local/regional temperature measurement results since recently published standard changes the testing temperature form high PG temperature to regional temperature, which is 58^{o}C for the testing location.
- MSCR test results showed that individual SBS and DVR modifications achieved the same traffic level at all corresponding percentages. While the addition of 1%SBS or

3%DVR brought the traffic level to “V-grade”, both 2%SBS-6%DVR and 3%SBS-9%DVR modifications reached “E-grade” traffic level.

- Combined modifications with SBS and RTR showed similar behaviour. Level-1 combined modifications achieved “V-grade” traffic level whereas Level-2 combinations achieved “E-grade” other than 1%SBS+6%DVR.
- Since MSCR standard does not apply to asphalt binders having discrete particles larger than 250 microns in size, concentric cylinder testing geometry was used to perform the MSCR test of CR modified asphalt binders in addition to parallel plate geometry. Test results obtained from CC and PP geometries were very similar.
- DVR alone did not pass the elastic response requirements at any level. However, combined modification with 2%SBS helped achieving the elasticity requirements.
- Although individual CR modification did not have enough elastic response up to 6%, higher percentage individual CR and SBS+CR combined modifications other than 1%SBS+3%CR passed the elastic response curve.

6.3 RECOMMENDATIONS FOR FUTURE WORK

Numerous topics related with asphalt binder and mixture modifications were covered thoroughly in the dissertation. However, enhancements can be made to the knowledge gained from this study.

Concerning the study covered in Chapter 3, dry CR addition was determined as 0.5%. This amount was accepted to reach the total CR amount close to traditional CRWET process which is around 20% by weight of binder. Performance properties of the CRHY and CRTB mixture were investigated. Future work can include addition of more dry CR particles into the mixture and performing rheological testing on the extracted binders. Moreover, various mesh size CR could have been researched to better understand the effect of CR size in CRHY mixtures. Since the ultimate goal of using recycled tire rubber in the roadways is to construct more sustainable and environmentally friendly roadways, it is recommended to prepare CRHY mixture by using one of the asphalt binders prepared and tested in Chapter 5. This can be done using either SBS+CR or SBS+DVR modified asphalt binders instead of using CRTB binder.

Chapter 4 included the investigation of the binder softening effect of recycled tire rubber modifiers for high percent reclaimed asphalt pavement mixtures. This study showed that it is possible to use CR modified asphalt binders instead of expensive grade bump softer binders. Testing in this chapter included linear viscoelastic characterization, push-pull fatigue and indirect tensile strength for thermal cracking of the mixtures. Further testing including moisture susceptibility and rutting potential could be performed even though rutting is not a potential problem for high RAP content mixtures. Future work could be extended to include binder rheological properties and investigate the relation between binder and mixture properties, especially for VECD model based LAS binder and PP fatigue mixtures testing. Mixture performances by using different percentage rubberized asphalt binders could be studied to find the optimum performance.

Finally, Chapter 5 included the combined effect of SBS and RTR modification on performance grade, fatigue cracking and rutting resistances. One of the biggest questions raised was to binder modifications process. Since there is not any specifications stating how to perform binder modification using elastomeric polymers or devulcanized rubber pellets, further research should focus on standardizing the asphalt binder modification. The ultimate results of modified binders will change according to the mixing temperate, revolution of the blending equipment and even type of milling device. Even milling SBS and DVR pellets together and separate affected the continuous performance grades. More research is definitely necessary to fully understand the effect of combined modifications. This could be done by using various asphalt binder sources, different elastomeric polymers, various mesh size crumb rubbers, different level devulcanized rubber, and changing amount and type of crosslinking agents.

All binder tests were conducted according to the standard specifications. However, some sections of the standards could have been enhanced to reflect the real behavior of asphalt binders and mixtures that occur in the field. LAS test is advised to be conducted at the intermediate pavement temperature determined from PG of the asphalt binder according to AASHTO M320. Moreover, some researchers suggested performing the LAS test at equi-stiffness temperatures. However, once the asphalt pavement with different sections constructed in the field, they will experience the same temperature. This study suggests using the same intermediate pavement temperature determined from PG of the base asphalt binder according to AASHTO M320. As long as modified base asphalt binder has the same PG, all modified asphalt binders, regardless of their PG values should be tested at the same temperature.

Another suggestion made is related with MSCR test. The percent non-recoverable creep compliance difference between two stress levels (0.1 kPa and 3.2 kPa) is limited at maximum 75%. This limitation was set to insure that the binder is not overreacting to unexpected heavy loads and high temperatures. However, heavily modified asphalt binders such as 20%CR or 14%DVR, non-recoverable creep compliance at 0.1kPa shear stress is very small value. This results in higher %J_{nr} difference values. Under the light of findings of this research study, enhancements could be done on %J_{nr} difference limitations.