**FELIPE FILIZZOLA CAMARGO**

# FIELD AND LABORATORY PERFORMANCE EVALUATION OF A FIELD-BLENDED RUBBER ASPHALT

A thesis submitted to the *Escola Politécnica da Universidade de São Paulo* in partial fulfillment of the requirements for the degree of Doctor in Transportation Engineering

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Research Area:
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Advisor:
**Prof. Dr**
**Liedi Légi Bariani Bernucci**

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## DEDICATION

To my wife, Carla, my son, Henrique, and my daughter, Gabriela, for your unconditional love and companionship. You give me the strength to go on.

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# RESUMO

No Brasil, o asfalto-borracha vem sendo utilizado desde meados de 2001. Dentre os processos de fabricação do asfalto-borracha, o mais utilizado no Brasil é o asfalto-borracha estocável ou *terminal blend*. Contudo, o asfalto-borracha do tipo não estocável (*field blend*) vem sendo bastante difundido nos Estados Unidos há décadas, principalmente no estado do Arizona. Este processo resulta em um asfalto-borracha de alta viscosidade, com alto desempenho, porém requer um equipamento de fabricação de asfalto-borracha específico, instalado no canteiro de obras, ou muito próximo à usina de asfaltos fornecedora da obra. Tendo em vista as possíveis vantagens tecnológicas do asfalto-borracha *field blend* e o conhecimento ainda pequeno sobre esta técnica no Brasil, há a necessidade de desenvolver estudos para a avaliação deste tipo de material frente às condições climáticas e de solicitação pelas cargas viárias em nosso país. Para tanto, foi realizado um estudo em laboratório para verificar as propriedades reológicas de um asfalto-borracha *field blend* e compará-las com as características de ligantes típicos empregados no Brasil (um CAP 30-45 e um ligante modificado por polímero elastomérico do tipo SBS). Elegeu-se o *Multiple Stress Creep and Recovery* (MSCR) para verificar a deformação permanente e o *Time Sweep* e *Linear Amplitude Sweep* (LAS) para verificar o comportamento na fadiga dos ligantes. Na sequência, determinou-se a deformação permanente e o comportamento à fadiga de uma mistura asfáltica descontínua (*gap-graded*) empregando o asfalto-borracha *field blend* em laboratório e no campo. A deformação permanente da mistura foi verificada por meio do simulador de tráfego LCPC, enquanto a vida de fadiga foi determinada utilizando o ensaio de flexão em viga (4 pontos). Por fim, foi construída uma seção teste após a conclusão da obra de restauração dos pavimentos da rodovia RJ-122, local onde se elegeu a utilização desta tecnologia pela primeira vez no país. O desempenho da mistura foi estudado *in loco* com o emprego de ensaios acelerados do pavimento utilizando-se o simulador de tráfego linear móvel em tamanho real. Os resultados obtidos foram utilizados para modelar o desempenho da estrutura com o revestimento asfáltico constituído pela mistura com o asfalto-borracha por meio dos modelos de trincamento e de deformação permanente do *Highway Development and Management Model* (HDM-4), podendo-se verificar o desempenho desta mistura calibrado para as condições locais. Os ensaios acelerados foram validados em campo por meio de campanhas de monitoramento periódicas realizadas ao longo de quatro anos na rodovia RJ-122. Pelos ensaios de laboratório no ligante e na mistura foi possível concluir que o asfalto-borracha *field blend* apresenta um bom desempenho quanto à deformação permanente e à fadiga, corroborando o que foi verificado no campo.

**Palavras-chave**: asfalto-borracha *field blend*, deformação permanente, fadiga, ensaios acelerados de pavimentos, HDM

# ABSTRACT

Rubber asphalt has been used in Brazil as early as 2001. Among the fabrication processes of rubber asphalt, the most widely used in Brazil is the terminal blend. However, the use of field-blended rubber asphalt has been around in the United States for decades, especially in the state of Arizona. This process results in a highly viscous material with enhanced engineering properties, but requires a specific equipment that is typically installed at the job site, or close to the supplying asphalt plant. Thus, keeping in mind the possible technological advantages of using a field-blended rubber asphalt mixture and the lack of information regarding this technique in Brazil, there is a necessity to develop studies to assess the performance of this type of material in our environmental conditions and axle loading configurations. Therefore, a laboratory study was conducted to determine the rheological properties of a field-blended rubber asphalt and compare them to those of typical binders used in Brazil (an AC 30/45 penetration grade and a binder modified with SBS, an elastomeric polymer). Binder permanent deformation was determined using the Multiple Stress Creep and Recovery (MSCR) test, whereas binder fatigue behavior was determined using the Time Sweep and Linear Amplitude Sweep (LAS) tests. Subsequently, the permanent deformation and fatigue behavior of a gap-graded mixture using the field-blended rubber asphalt were assessed in the laboratory and in the field. The permanent deformation of the mixture was determined in the laboratory using the LCPC wheel track test, whereas the fatigue behavior was determined using the four point bending flexural test. A test section was built after the rehabilitation job of highway RJ-122, where a field-blended rubber asphalt mixture was first used in the country. The mixture performance was studied in situ through accelerated pavement tests using a full scale, large mobile traffic simulator. The results were used to model the performance of the structure with the rubber asphalt mixture by means of the Highway Development and Management Model (HDM-4) cracking and permanent deformation models, calibrated to local conditions. Accelerated pavement tests were validated through periodic pavement monitoring campaigns conducted for four years in a test section in Highway RJ-122. The field-blended rubber asphalt showed a good performance in terms of permanent deformation and fatigue determined in the

laboratory at the binder and mixture levels, which confirms what was verified in the field.

**Keywords**: field-blended rubber asphalt, permanent deformation, fatigue, full scale accelerated pavement tests, HDM

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# NOMENCLATURE

| | |
|---|---|
| AASHTO | American association of state and highway transportation officials |
| ABCR | *Associação brasileira de concessionárias de rodovias* |
| ABNT | *Associação brasileira de normas técnicas* |
| AC | Asphalt cement |
| ADOT | Arizona department of transportation |
| ADT | Average daily traffic |
| ANOVA | Analysis of variance |
| ASTM | American society of testing and materials |
| BPR | Bureau of public roads |
| CALTRANS | California department of transportation |
| CNT | Confederação nacional do transporte |
| CRM | Crumb rubber modifier |
| CRERL | Cold regions engineering research laboratory |
| DCP | Dicyclopentadiene |
| DER-RJ | *Departamento de estradas de rodagem do estado do Rio de Janeiro* |
| DER-SP | *Departamento de estradas de rodagem do estado de São Paulo* |
| DNIT | *Departamento nacional de infraestrutura terrestre* |
| DOT | Department of transportation |
| DSR | Dynamic shear rheometer |
| ENB | Ethylidene norbornene |
| ESAL | Equivalent single axle loads |
| ETRA | European tyre recycling association |
| EVA | Ethylene-vinyl acetate |
| FHWA | Federal highway administration |
| f-sAPT | Full scale accelerated pavement testing |
| FSMTS | Full scale, mobile traffic simulator |
| FWD | Falling weight deflectometer |
| GT | Grip tester |
| HDM-4 | Highway development and management model |
| HDM-III | Highway design and maintenance standards |
| HMA | Hot-mix asphalt |
| HX | Trans1,4-hexadiene |
| IBP | *Instituto brasileiro de petróleo, gás e biocombustíveis* |
| IPR | *Instituto de pesquisas rodoviárias* |
| IRI | International roughness index |
| LAS | Linear amplitude sweep |
| LCMS | Laser crack measurement system |
| LCPC | *Laboratoire central des ponts et chaussées* |
| LEF | Load equivalency factor |
| MSCR | Multiple stress creep and recovery |

| | |
|---|---|
| NCHRP | National cooperative highway research program |
| PG | Performance grade |
| PIARC | World road association |
| RET | Reactive elastomeric terpolymers |
| RMSE | Root mean square error |
| RTFOT | Rolling thin film oven test |
| RUMAC | Rubber-modified asphalt concrete |
| SBR | Styrene-butadiene-styrene |
| SBS | Styrene-butadiene rubber |
| SHRP | Strategic highway research program |
| SUPERPAVE | Superior performing asphalt pavement system |
| TST | Time sweep test |
| USA | United States of America |
| USACE | United States Corps of Engineers |
| VECD | Viscoelastic continuum damage |

# LIST OF SYMBOLS

| | |
|---|---|
| # | Number |
| % | Percentage |
| ” | Inches |
| ± | More or less |
| ° C | Degrees celsius |
| AMAP | Asphalt mix on asphalt pavement |
| BPN | British pendulum number |
| C | Integrity |
| CDS | Construction defects index |
| cm | Centimeters |
| COMP | Relative compaction |
| cP | Centipoise |
| CRT | Crack retardation time due to maintenance |
| D | Diameter |
| $\delta$ | Phase angle |
| D | Damage intensity |
| $\Delta$RDPD | Incremental increase in plastic deformation |
| $\Delta$RDSTUC | Incremental rutting increase in structural deformation without cracking |
| E | Resilient modulus |
| E* | Dynamic complex modulus |
| $\varepsilon_t$ | Applied tensile strain |
| $\gamma$ | Strain level |
| G* | Complex shear modulus |
| \|G*\| | Dynamic shear modulus |
| GN | Grip number |
| HS | Total thickness of bituminous surfacing |
| HSNEW | Thickness of most recent surfacing |
| Hz | Hertz |
| ICA | Time to initiation of all structural cracks |
| $J_{nr}$ | Non-recoverable creep compliance |
| $J_{nr,\ diff}$ | Stress sensitivity parameter |
| $K_{cia}$ | Calibration factor for cracking initiation |
| $K_{cpa}$ | Calibration factor for all structural cracking progression |
| kg | Kilogram |
| km | Kilometer |
| km/h | Kilometer per hour |
| kN | Kilonewton |
| kPa | Kilopascal |
| $K_{rdw}$ | Calibration factor for surface wear |
| $k_{rid}$ | Calibration factor for initial densification |
| $k_{rpd}$ | Calibration factor for plastic deformation |

| | |
|---|---|
| $k_{rst}$ | Calibration factor for structural deformation |
| m | Meters |
| m/km | Meters per kilometer |
| min | Minutes |
| mm | Millimeters |
| mm | Micrometers |
| mm/year | Millimeters per year |
| $mm^3$ | Cubic millimeters |
| MPa | Megapascal |
| N | Number of load repetitions to failure |
| $N_{100}$ | Number of cycles at a strain level of 100 |
| $N_f$ | Fatigue life |
| PCRW | Area of wide cracking before latest reseal or overlay |
| R100 | Percent recovery for 0.1 kpa |
| R3200 | Percent recovery for 3.2 kpa |
| RPM | Rotations per minute |
| s | Seconds |
| Sh | Speed of heavy vehicles |
| SNC | Modified structural number |
| SNP | Adjusted structural number |
| $\sigma_t$ | Applied stress |
| VMA | Voids in mineral aggregate |
| YE4 | Annual equivalent standard axles |

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# 1. INTRODUCTION

The vast majority of roads in Brazil are composed of flexible pavements, in a country were more than 60% of the freight transport is through highway transportation (CNT, 2015). Even though highway transportation is the main transportation mode, there is still an urgent need for new technologies for the construction and rehabilitation of pavements. This lack of road network, associated with elevated traffic conditions carrying, at many times, excessive loads that have exponential effects on pavement degradation, implicates in undesirable pavement conditions.

In 2011 alone, approximately 26% of the federal highways exhibited pavements in regular or poor conditions in terms of pavement roughness (IRI, in m/km), which is a parameter directly proportional to the user ride quality. Similarly, approximately 37% of the federal highways showed pavements in similar conditions using the *Índice de Condição da Superfície* (Superficial Condition Index) (DNIT, 2011b). This pavement parameter is used to represent the pavement´s superficial condition, similar to the present serviceability index concept, and takes into account the number of distresses in the pavement along with the pavement roughness. A value of 5 is given to a pavement in excellent conditions, whereas a value of 1 is given to a pavement with poor conditions.

One way to extend the durability of pavements is by using binders that promote the enhancement of the mechanical properties of the asphalt mixtures. The mechanical properties of binders can be incremented, among others, by using polymers (RET, SBS, SBR, etc.) and crumb rubber modifier from tire wastes. Rubber modification techniques have been used worldwide in the past few decades (particularly in the North-American states of Texas, Arizona, Florida, and California) and nationally, for more than 10 years (Leite *et al.*, 2000; Oda & Fernandes Jr., 2000; Bertollo *et al*., 2000). According to the Brazilian Association of Highway Concessionaries (ABCR), rubber asphalt was used in approximately 1,867 kilometers of conceded highways (roughly 10% of the total extension granted) between the years of 2008 and 2012 (ABCR, 2013).

It is well known that rubber modification results in gains of the rheological properties of binders (Bahia & Davies, 1994; Moreno-Navarro *et al*., 2015), as well as an improved resistance to aging (Sainton, 1990), resulting in an asphalt mixture of high performance and a prolonged pavement service life. According to the California Department of Transportation, the advantages of using rubber asphalt in comparison to conventional binders are increased viscosity, which allows for greater film thickness in paving mixes without excessive drain down or bleeding, and increased elasticity at higher temperatures. In addition, rubber asphalt mixtures result in pavements with increased durability, increased resistance to surface initiated (top-down) and fatigue/reflection cracking, reduced temperature susceptibility, improved aging and oxidation resistance, improved resistance to rutting (permanent deformation), and lower pavement maintenance costs due to improved pavement durability and performance (CALTRANS, 2006).

Among the fabrication processes available for obtaining rubber asphalt, terminal blend is the most commonly used in Brazil. However, the use of field-blended rubber asphalt has been around in the United States for decades, especially in the state of Arizona. The field blending process for rubber asphalt results in a highly viscous material with enhanced engineering properties, but requires a specific equipment that is typically installed at the job site or close to the supplying asphalt plant.

The lack of information regarding this technique in Brazil, however, inhibits a more common use in pavement construction/rehabilitation. For instance, in a survey conducted in the United States in 2011, 42 of the 51 departments of transportation in the country answered a questionnaire regarding crumb rubber modified binders. As a result, forty-two percent of the state departments that have never used these types of binders have not done so because its performance is still uncertain (Bandini, 2011).

Thus, keeping in mind the possible technological advantages of using a field-blended rubber asphalt mixture and the lack of information regarding this technique in Brazil, there is a necessity to develop studies to assess the performance of this type of material in our environmental conditions and axle loading configurations. Moreover, pavement engineers need to know how to deal with pavements using field-blended rubber asphalt that will yield equal or better performance than pavements constructed with conventional asphalt mixtures.

### 1.1. Objectives

This study aims to evaluate the performance of a surface course using a field-blended rubber asphalt mixture by means of rheological binder testing, laboratory permanent deformation and fatigue tests, accelerated pavement tests in the field using a large scale, mobile traffic simulator, and periodic pavement monitoring campaigns of an in-service test section in order to study and predict performance.

First, the permanent deformation and fatigue behavior of a field-blended rubber asphalt were compared to those of typical binders used in Brazil (a neat binder of penetration grade 30/45 and a binder modified with SBS, an elastomeric polymer). The permanent deformation and fatigue behavior of a gap-graded mixture using the field-blended rubber asphalt were then evaluated for laboratory-compacted and specimens sawed from field slabs. The field performance of the mixture was studied through accelerated pavement tests using a full scale, large mobile traffic simulator in a test section at highway RJ-122.

The accelerated pavement test (APT) results were then used to model the performance of the pavement using the rubber asphalt mixture by means of the Highway Development and Management Model (HDM-4) cracking and permanent deformation models. The HDM performance models were used to predict the pavement performance with this type of mixture calibrated to local conditions. Finally, periodic pavement monitoring campaigns of a test section in RJ-122 were conducted up to 4 years of service in order to validate APT.

### 1.2. Thesis Outline

This thesis combines four technical papers, and is presented according to the style and format of the International Journal of Pavement Engineering, as well as the procedures in the *Escola Politécnica da Universidade de São Paulo* guide for presenting dissertations and thesis. Because of the structure used, some parts may be repeated along the document. The thesis is organized in seven sections as described next.

Chapter 1 presents an introduction that includes an overview of the subject, the research objectives, and the thesis outline.

Chapter 2 presents a literature review regarding the use of rubber asphalt mixtures in highway construction, including a brief history of usage, a description of the main components in rubber asphalt mixtures, the tire reclaiming process for obtaining the crumb rubber modifier, and the different methods for obtaining rubber asphalt mixtures.

Chapter 3 presents a technical paper regarding the permanent deformation and fatigue behavior of the field-blended rubber asphalt binder in respect to the neat binder and a polymer-modified binder (SBS). This paper was prepared for later submission. The authors of this paper are Felipe F. Camargo, Kamilla L. Vasconcelos, and Liedi L.B. Bernucci.

Chapter 4 presents a technical paper regarding the permanent deformation of the field-blended rubber asphalt binder in respect to the neat binder and verifies the mixture permanent deformation at different levels (binder, mixture, and field). This paper was prepared for later submission. The authors of this paper are Felipe F. Camargo, Kamilla L. Vasconcelos, and Liedi L.B. Bernucci.

Chapter 5 presents a technical paper regarding the fatigue behavior of the field-blended rubber asphalt binder in respect to the neat binder and verifies the mixture fatigue behavior at different levels (binder, mixture, and field). This paper was prepared for later submission. The authors of this paper are Felipe F. Camargo, Kamilla L. Vasconcelos, and Liedi L.B. Bernucci.

Chapter 6 presents a technical paper that is a case study of the RJ-122 pavement rehabilitation job, which was the first use of a field-blended rubber asphalt mixture in Brazil. This paper was submitted to the International Journal of Pavement

Engineering. The authors of this paper are Felipe F. Camargo and Liedi L.B. Bernucci.

Chapter 7 presents the summary and conclusions of the present thesis. In addition, suggestions for future studies are provided.

The process for obtaining the crumb rubber modifier and the field blending process at RJ-122 are shown in Appendix A.

# 2. LITERATURE REVIEW

In order to study the field-blended rubber asphalt mixture, it is important to provide an overview of asphalt mixtures modified with crumb rubber. In addition, an emphasis will be given to the different methods available for rubber modification, as well as a brief description of the constituents of rubber-asphalt. A relevant literature review may be found in each specific paper.

## 2.1. Asphalt Cement and Modifiers

Asphalt is a type of material widely used in road construction. In Brazil, asphalt concrete is present in the majority of the paved roads. Asphalt is defined as a solid material, or semi-solid at room temperature, black in color, and composed mainly of bitumen. It may be obtained by the distillation of petroleum, or may be found in natural deposits (IBP, 1999).

Asphalt is a complex mixture of organic molecules. It is composed of hydrocarbons in large quantities and complemented by small amounts of oxygen, nitrogen, and sulfur. The exact chemical composition of asphalt is difficult to be determined because it varies significantly depending on the origin of the crude oil. However, an analysis of asphalts fabricated from different crude oils resulted in the following proportions (SHELL, 2003): 82-88% of carbon, 8-11% hydrogen, 0-6% sulfur, 0-1.5% oxygen, and 0-1% nitrogen.

Different methods are available for analyzing the chemical composition of asphalt, while the composition varies according to the type of fractioning used. Asphalt is typically separated according to the Standard Test Method for Separation of Asphalt into Four Fractions (ASTM D4124-09), which separates the asphalt in the following fractions: saturates, naphthene aromatics, polar aromatics, and iso-octane insoluble asphaltenes. In this method, asphaltenes are first precipitated using n-heptane and the remaining constituents, which are soluble in n-heptane, are separated by adsorption chromatography (ASTM, 2009).

Asphalt cement is obtained or fabricated in order to exhibit adequate characteristics for use in flexible pavements. It may be obtained from crude oil by means of distillation processes (one, two, or three distillation processes) (Guarçoni, 1994). Asphalt cement obtained from distillation is called Petroleum Asphalt. On the other hand, asphalt obtained from natural deposits is called Natural Asphalt. At ambient temperature, these materials are semi-solid and, therefore, require heating to obtain the appropriate consistency to cover the aggregates. They exhibit flexibility, durability, agglutination, and impermeabilization characteristics, as well as a high resistance to most acids, salts and alkalis (Souza *et al.*, 1995).

Currently, Brazilian petroleum asphalt cements are graded according to their penetration at 25°C. The National Department of Transport Infrastructure (*Departamento Nacional de Infraestrutura Terrestre* – DNIT) specifies four types of asphalts in accordance with DNIT 095/2006-EM: AC 30-45, AC 50-70, AC 85-100, and AC 150-200 (DNIT, 2006a).

With the recent growth of traffic and the increasing axle-loads of commercial vehicles, it is often necessary to modify the properties of conventional asphalts to provide an adequate pavement service life. To increase strength, the asphalt cements may be modified with the following (Leite, 1999):

- Natural asphalts: gilsonite (USA), asphaltite (Argentina), and Trinidad Asphalt.
- Fillers: hydrated lime, Portland cement, and silica.
- Fibers: glass, asbestos, cellulose, and polymeric fibers.
- Sulfur.
- Polymers: Styrene-butadiene rubber (SBR), styrene-butadiene-styrene (SBS), ethylene-vinyl acetate (EVA), reactive elastomeric terpolymers (RET), and crumb rubber modifier from tires, among others.

One of the benefits of using modifiers is an increased resistance to permanent deformation at high temperatures, without affecting the properties of asphalt at other temperatures. In order to do so, the asphalt is hardened such that the response or total viscoelastic deformation is reduced, or by increasing the elastic component of asphalt (Lewandowski, 1994).

With the increasing use of these types of modifiers, DNIT has updated the standards in order to include modified binders, especially in the case of polymers and rubber obtained from tires. The following standards regarding the modification of asphalt cement are available:

- DNIT 111/2009-EM (DNIT, 2009a): *Pavimentação flexível - Cimento asfáltico modificado por borracha de pneus inservíveis pelo processo via úmida, do tipo* "Terminal Blending" (flexible pavements - asphalt cement modified with rubber from tires through wet process and by means of terminal blending).

- DNIT 129/2011-EM (DNIT, 2011a): *Cimento asfáltico de petróleo modificado por polímero elastomérico* (asphalt cement from petroleum modified with elastomeric polymer).

- DNIT 168/2013-EM (DNIT, 2013): *Cimento asfáltico de petróleo modificado por asfalto natural do tipo TLA* (Trinidad Lake Asphalt) (asphalt cement from petroleum modified with Trinidad Lake Natural Asphalt)

Reclaimed rubber from recycled tires is used for producing asphalt cement modified with rubber. The reclaimed rubber is currently referred to as crumb rubber modifier (CRM). A description of CRM and the methods available for producing CRM are provided next.

## 2.2. Crumb Rubber Modifier

### *2.2.1. Natural or Synthetic Rubber*

There are two types of rubber: natural and synthetic. Natural rubber comes from the sap of the Pará rubber tree, sharinga tree, or better known as simply the rubber tree (*Hevea brasiliensis*). Natural rubber is used in over 40,000 products in various industrial areas, such as products for medical use, adhesives, footwear, tire manufacturing, etc (Gonçalves & Fontes, 2009). In addition, natural rubber is widely used in the transportation industry and war products, because it is a material with good insulating properties and impermeability to air and water. Most of the world's natural rubber is intended for the tire industry (Galiani, 2010).

Natural rubber may also be found in latex, which is an emulsion comprising of approximately 37.5 % rubber and about 60 % of water. The latex is a colloidal dispersion consisting of polymeric particles in an aqueous medium. Natural rubber is the only natural polymer obtained by latex coagulation of some plant species. The latex undergoes a natural or spontaneous coagulation process a few hours after being collected, wherein the pH of the material is reduced from 6.5 to 5.0 on a biochemical process that occurs rapidly. This process forms a clot with approximately 60 % of dry rubber composed of cis1.4-polyisoprene (Oliveira, 2004).

To accelerate the natural clotting process, an acidic substance may be added to the latex, decreasing the coagulation process to only a few minutes (Galiani, 2010). The components of the newly collected latex are shown in Table 2.1, whereas the chemical composition of the dry natural rubber coagulated with acid are shown in Table 2.2.

**Table 2.1 -** Composition of recently-collected latex (Hwee & Tanaka, 1993)

| **Component** | **%** |
|---|---|
| Water | 58.6 |
| Rubber hydrocarbon | 36.0 |
| Proteins, amino acids, and nitrogenous compounds | 1.7 |
| Lipids | 1.6 |
| Ash | 0.5 |
| Inositols and carbohydrates | 1.6 |

**Table 2.2 -** Composition of natural rubber (Sekaran, 1988)

| **Component** | **%** |
|---|---|
| Hydrocarbons | 93.7 |
| Neutral lipids | 2.4 |
| Glycolipids and phospholipids | 1.0 |
| Proteins | 2.2 |
| Carbohydrates | 0.4 |
| Inorganic constituents | 0.2 |
| Others | 0.1 |

In Brazil, the specifications for natural rubber are given by the Brazilian Association of Technical Standards (ABNT). The specifications for natural rubber, according to standard NBR 11597 (ABNT, 1997), are summarized in Table 2.3. The synthetic rubbers, on the other hand, are produced in two stages mostly from petroleum byproducts: the first stage is the production of monomers (elongated particles and constituted by small units), followed by polymerization to form the rubber.

Synthetic rubbers have similar properties to natural rubbers. There are several types of synthetic rubber for various types of applications, such as the styrene butadiene rubber used in the modification of asphalt binders and the silicone rubber used for sealing (Blow, 1971).

**Table 2.3 -** Natural rubber specifications - NBR 11597

| Specification | Limits |
|---|---|
| Volatile matter (max. %) | 0.8 |
| Nitrogen (max. %) | 0.6 |
| Acetone extract (max. %) | 3.5 |
| Dirt retained (max %) | 0.1 |
| Ash content (max %) | 0.75 |
| Wallace rapid plasticity, P0 (min.) | 30 |
| Plasticity retention index, PRI (min. %) | 60 |

#### *2.2.1.1. Rubber Processing*

Rubber vulcanization is a process that consists in altering the properties of natural rubber. This process involves the separation of the crosslinked polymer by heat or other means, resulting in the degradation of the polymeric material. Consequently, the polymer chains absorb the molecules of the solvent, which causes swelling of the material. The higher the crosslink density, the less space is available for the penetration of solvent molecules and the lower the degree of swelling (Akcelrud, 2007).

There are two types of vulcanization: hot, used for most of the rubber products, including tires, and cold, for the manufacture of soft and thin products, such as surgical gloves, for example (Blow, 1971). Vulcanization may be accomplished by peroxide or sulfur, or by adding fillers, such as antioxidants and antiozonants (Blow &

Hepburn, 1982; Silva, 2007). The most commonly used process of vulcanization is with sulfur. In this process, disulfide bonds are formed to connect one molecule to another, reducing the mobility of the polymer chains. As a result, there is an increase in tensile strength and elasticity, preserving its extensibility (Marinho, 2005; Blow & Hepburn, 1982). The ratio of sulfur to rubber may range from 1:40 (soft products) to 1:1 for harder products (Shulman, 2000).

In peroxide vulcanization, the process consists of breaking the double bonds of the polymer chain and a three-dimensional network is achieved by the bonding between carbon molecules. Among the most commonly used peroxides for the vulcanization of rubber stand dicyclopentadiene (DCP), trans1,4-hexadiene (HX) and ethylidene norbornene (ENB) (Hofmann, 1989).

The incorporation of fillers in natural rubber contributes to an increase of hardness, abrasion resistance, and pigmentation. Different types of fillers are used, however, the most commonly used is the carbon black (Blow & Hepburn, 1982; Fried, 1995). Other widely used additives for natural rubber are the anti-degradants, which provide chemical protection to the elastomer against degradation by oxygen, ozone, and radiation (Blow & Hepburn, 1982; Fried, 1995; Akcelrud, 2007).

### *2.2.2. Tires*

Tires are all materials or products that function on compressed air. In the case of vehicles for transportation, the tires are rubber layers that are attached to a rim used for to buffer the contact between the vehicle and the road on which it travels. A typical composition of a tire is natural and synthetic rubber, carbon black, nylon, steel fibers and additives like paint, oils, etc. A cross-section view of an automobile tire along with its constituents is shown in Figure 2.1.

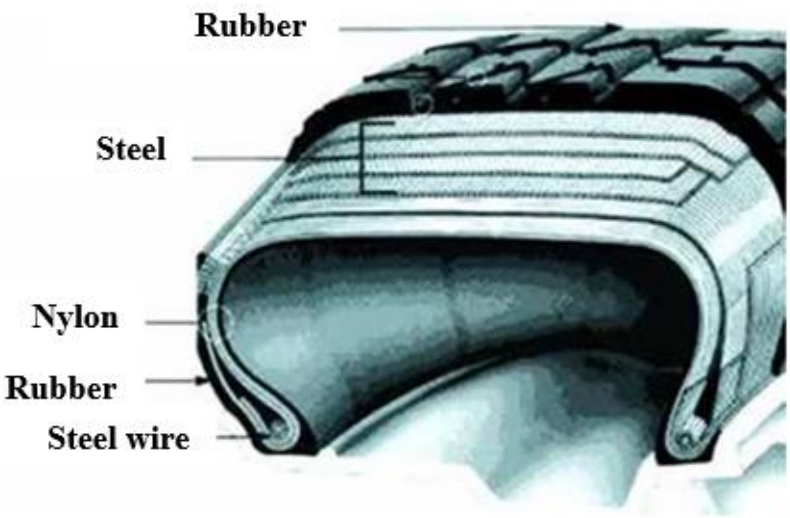



**Figure 2.1.** Cross-section view of an automobile tire (Andrietta, 2004)

In general, tires are composed of natural rubber and synthetic rubber. The ratio of these materials varies according to the tire size and use. The bigger the tire, the greater the concentration of natural rubber in relation to synthetic rubber. The second largest component of the tire is carbon black. The amount of carbon black may vary significantly depending on the type of manufacturer due to the various types available. Carbon black is used primarily to stiffen the tread to enhance traction, control abrasion, and reduce aquaplaning (Shulman, 2000). The third major component in terms of quantity in a tire is steel. The steel is used to provide rigidity and strength, as well as flexibility. In addition, most tires contain either nylon, rayon, or polyester.

According to the European Tyre Recycling Association (Shulman, 2008), the chemical composition of a light vehicle (passenger car) tire is, mainly, of hydrocarbons (Table 2.4). In addition, there is a significant difference in composition of light and heavy vehicle tires, as shown in Table 2.5.

**Table 2.4 -** Chemical composition of a light vehicle (passenger car) tire (Shulman, 2008)

| Element/Compound | Percentage (%) |
|---|---|
| Carbon | 70.0 |
| Hydrogen | 7.0 |
| Zinc oxide | 1.2 |
| Sulfur | 1.3 |
| Iron | 15.0 |
| Others | 5.5 |

**Table 2.5 -** Chemical composition of a light/ heavy vehicle tires (Shulman, 2008)

| Element/Compound | Percentage (%) | |
|---|---|---|
| | light | heavy |
| Rubbers | 48 | 45 |
| Carbon black | 22 | 22 |
| Steel | 15 | 25 |
| Nylon | 5 | 0 |
| Zinc oxide | 1 | 2 |
| Sulfur | 1 | 1 |
| Additives | 8 | 5 |

A study by Santos *et al*. (2002) showed with a thermogravimetric analysis of several brands of passenger cars tires that there are different polymer concentrations in tire rubber shavings, such as SBS, SBR, natural rubber, etc. Furthermore, the concentration of natural rubber varied from 22 to 39%, while the concentration of synthetic rubber was between 22 and 44%. The major difference between the rubber compositions between the passenger car and commercial tires occurred in the concentration of natural rubber, which is in greater presence in commercial vehicle tires.

### *2.2.3. Characteristics of Reclaimed Rubber*

Crumb rubber is the recycled rubber derived from light and heavy vehicles. The recycling process consists of removing the steel and fiber from scrap tire shavings. The different components of the tire after the recycling process are shown in Figure 2.2. The process of grinding the tire shavings, in order to fabricate finer grains, may be either ambient or cryogenic. The shavings are generally grinded and then sieved to fabricated the desired crumb rubber gradation according to specifications.

Currently, there are various types of crumb rubber gradations available. According to Bandini (2011), the crumb rubber may have a maximum diameter ranging from 2 mm (sieve no. 10) to 0.6 mm (sieve no. 30). Alternatively, a summary of crumb rubber use in the USA and Canada by CALTRANS (2005) showed a maximum diameter ranging from 4.75mm (sieve no. 4) and 0.18 mm (sieve no. 80). The gradation of crumb rubber has great influence on the physical properties of the modified binder (Neto *et al.*, 2006).


**Figure 2.2.** Crumb rubber (rubber 6mm, rubber 10 mesh, and rubber 20 mesh), fiber, and (steel) (RERUBBER, 2013)

In ambient grinding, the material is ground in a cracker mill at a temperature close to room temperature (Figure 2.3). During the process, which may include up to 3 grinding stages, the temperature increases significantly due to the friction between particles. The mill reduces the rubber particles because of the shearing process and sieves are coupled to the mill for obtaining the desired gradation. This process is typically necessary to promote elongated, irregularly shaped particles with high surface areas, which results in a better interaction with the asphalt cement (CALTRANS, 2005).

The cryogenic process uses liquid nitrogen to lower the temperature of the scrap tire significantly, until the particles are frozen and become brittle (Figure 2.4). The frozen scrap tires are then shattered by means of a hammer, which yields smooth particles with low specific area. Typically, the cryogenic process is used to reduce particle size prior to grinding at ambient temperatures (CALTRANS, 2005).

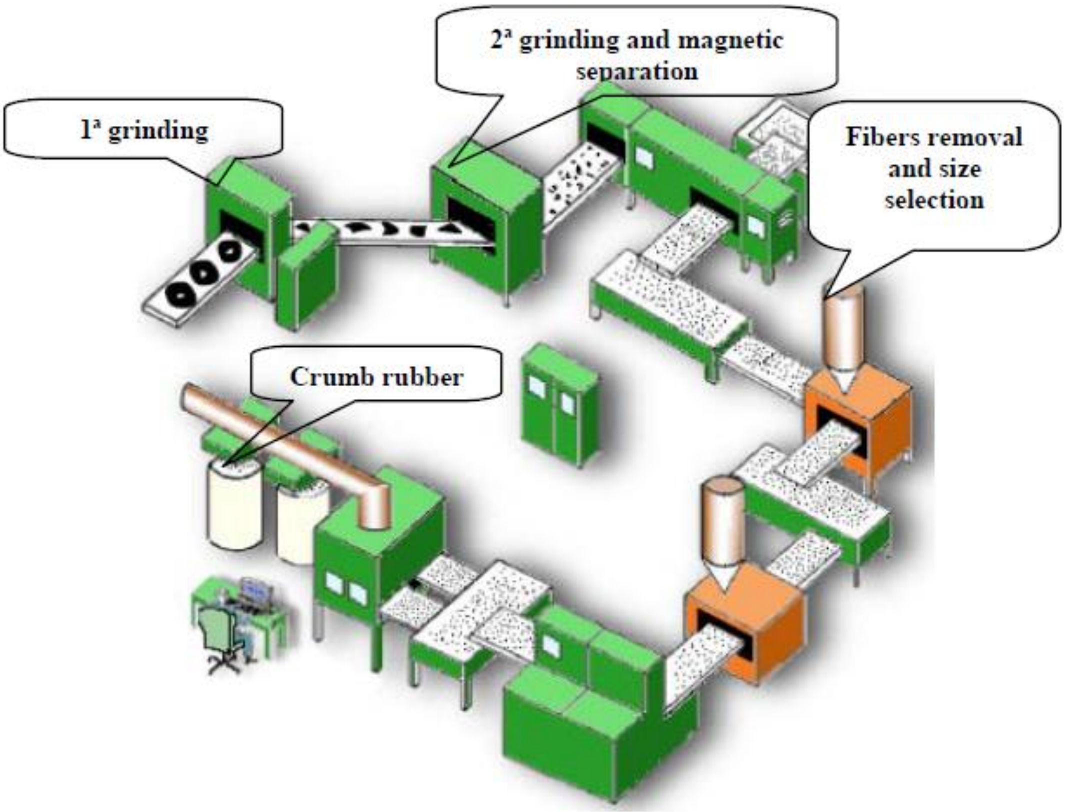



**Figure 2.3.** Ambient grinding process (Neto *et al.*, 2006)


**Figure 2.4.** Tire fragmentation (a), cryogenic tunnel (b), granulators (c), and the final step (d) (Neto *et al.*, 2006)

The crumb rubber particles obtained from the ambient and cryogenic grinding methods are shown in Figure 2.5. The particles from the ambient method (left), are characterized by an irregular shape, whereas the particles from the cryogenic method (right) are characterized by a smooth surface.

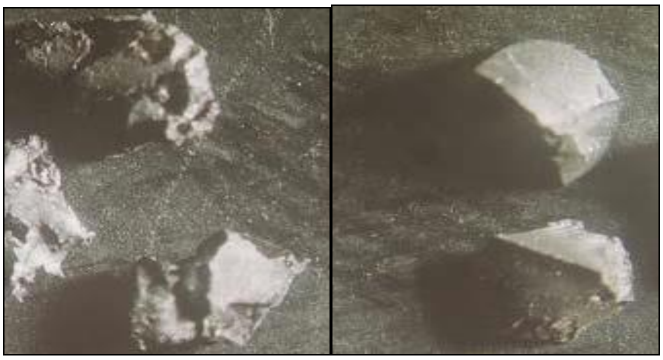


**Figure 2.5.** Crumb rubber particles obtained by ambient grinding (left) and cryogenic grinding (right) (Neto *et al.*, 2006)

## 2.3. Rubber Asphalt

Asphalt binders modified with crumb rubber may be referred to either rubberized asphalt or rubber asphalt, depending on the amount of crumb rubber used. Rubber asphalt is defined by ASTM as "a blend of asphalt cement, reclaimed tire rubber, and certain additives in which the rubber component is at least 15 percent by weight of the total blend and has reacted in the hot asphalt cement sufficiently to cause swelling of the rubber particles." Alternatively, rubberized asphalt contains up to 15% of crumb rubber.

### *2.3.1. History*

The first experiences with rubber asphalt date back to 1840 and involved the use of natural rubber to modify asphalt cement to enhance the mechanical properties. The process of modifying with natural and synthetic rubber was introduced in 1843 (Thompson & Hoiberg, 1979). The modification techniques were later improved in 1923 (Isacsson & Lu, 1999; Yildrim, 2007). According to Yildrim (2007), asphaltic materials with rubber modification were used in the 1930s for sealing, patching, and membranes. In 1950, the use of reclaimed tires in asphalt paving was reported (Hanson *et al*., 1994).

The engineers Lewis and Welborn from the Bureau of Public Roads (BPR) conducted an extensive study in 1950 to evaluate the effects of different types of rubber on the properties of asphalt. In this study, 14 different types of ground rubber were employed along with 3 asphalts. The results were published on the October of 1954 volume of the Public Roads journal, along with results from another contemporary study conducted by engineers Rex and Peck, also from BPR. The latter study focused on asphalt mixtures using different types of rubber modified binders (rubber from reclaimed tires, natural rubber, and SBR, among others). The mixtures were obtained using both the wet and dry processes. In March 1960, the Asphalt Institute hosted the first rubber asphalt symposium in Chicago, IL. The symposium included 5 presentations and discussions regarding the use of rubber asphalt in pavement construction (CALTRANS, 2006).

Also in 1960, the materials engineer Charles McDonald begun his studies of rubber modification of asphalts in Phoenix, Arizona. His studies in the 60s and 70s were essential for the development of the wet process for producing rubber asphalt, also known as the McDonald Process (CALTRANS, 2006). Charles McDonald found that after mixing the ground rubber with the asphalt binder and letting it interact for 45 to 60 minutes, the properties of the mixture were significantly different from those of the conventional mixture. This process resulted in the swelling of the particles at high temperatures, allowing for greater asphalt contents in the mixtures for paving (Huffman, 1980).

Charles McDonald was the first to routinely use rubber asphalt in patching and surface treatments for repair and maintenance of pavements in Phoenix, AZ. This was the city´s main maintenance strategy for its arterial streets for almost 20 years. In 1978, the first pavement with asphalt concrete modified with rubber by dry process was constructed on SR 50 in Meyers Flat, California. The rubber asphalt had 1% by weight rubber powder added to the dry aggregate prior to mixing with the asphalt binder, and its performance at the time was rated as good (CALTRANS, 2006).

In Brazil, the first studies of rubber asphalt were conducted in 2000 (Bertollo *et al*., 2000; Oda & Fernandes Jr, 2000; Leite *et al.,* 2000). Since then, terminal blend rubber asphalt has been widely used in the country, including jobs for the federal highway, the departments of transportation of São Paulo, Paraná, Santa Catarina, Minas Gerais and the cities of São Paulo, Curitiba, Rio de Janeiro, Florianópolis, and Belo Horizonte. In addition, terminal blend rubber asphalt has been used regularly in road concessionaries (GRECA, 2013). The ABCR annual report informed the use of rubber asphalt in paving jobs of approximately 1,867 kilometers of conceded highways between 2008 and 2012 (ABCR, 2013).

### *2.3.2. Methods for Obtaining Rubber Asphalt*

There are two methods for obtaining asphalt cement with crumb rubber modifier: the wet, or McDonald, process and the dry process (CALTRANS, 2006). In the wet process, the crumb rubber acts as a modifying agent and is added directly in the pre-heated binder. In the dry process, however, the crumb rubber is added to the mixture as part of the aggregate and not as a part of the asphalt binder.

#### *2.3.2.1. Dry Process*

Rubber-modified asphalt concrete, or RUMAC, was developed in Sweden in the 1960’s and is commonly referred to as Rubit in Europe. In the USA, it was patented as Plusride in 1978 (Heitzman, 1991). According to CALTRANS, the dry process uses approximately 1 to 3% of crumb rubber modifier as a substitute of the aggregate in asphalt concrete mixture, changing the mixture gradation. It is important to highlight that, even though some interaction may occur during the dry process, the binder is not considered modified by this methodology (CALTRANS, 2006). There are three types of dry process:

- Plusride: a gap-graded mixture (not continuous for all size fractions) with up to 3% of substitution of aggregate with crumb rubber. The asphalt content is typically around 7-9% (Heitzman, 1992). This system uses gap-graded coarse aggregates to provide space for the rubber particles.

- Generic technology or TAK System: developed in the 80's by Barry Takallou to produce mostly dense-graded mixtures, in which the size of the crumb rubber is typically smaller (by one sieve size) relative to the gap created in the mineral aggregate. The CRM used in the generic system are finer than that for the PlusRide system. The crumb rubber gradation is such to obtain a better interaction with the binder. The particle sizes may vary from 2 mm to 180 μm in diameter and contents vary up to 3% of the dry aggregate weight in the mixture (Takallou & Hicks, 1988).

- *Chunk Rubber*: developed by the United States Corps of Engineers (USACE) in its cold regions engineering research laboratory (CRREL), this method has particles with diameters ranging from 4.75 mm to 9.5 mm. The percentages range between 3, 6, and 12% of the aggregate weight and the optimum asphalt content may vary between 6.5 and 9.5% depending on the gradation used (Heitzman, 1992).

#### *2.3.2.2. Wet Process*

As mentioned earlier, the wet process was developed by Charles McDonald and refers to the modification of asphalt cement with crumb rubber modifier. This process requires that the crumb rubber be added to asphalt binder at elevated temperatures (176 ºC to 226 ºC) and resulting material be kept at high temperatures (150 ºC to 218 ºC) for a given time (typically from 45 to 60 min) to allow for the interaction between the rubber and binder. The interaction, commonly referred to as simply the reaction, is the physical exchange between the asphalt cement and the crumb rubber that takes place at high temperatures, which includes the swelling of rubber particles and change in the physical properties of the blend (binder and crumb rubber). Even though it is referred to as a reaction, the interaction is not a chemical reaction, but a physical interaction in which the aromatic oils and small volatile or active molecules from the asphalt cement are absorbed by the crumb rubber. Similar oils used in rubber compounding are released into the asphalt cement. A more appropriate term to describe this interaction is polymer swelling (CALTRANS, 2005).

The interaction depends on several variables, such as the temperature of the blend, the time the blend is kept at the designated temperature, the type and how long agitation is applied, and the type of crumb rubber used, as well as its size and specific area. During the interaction, the polymeric chains in the natural and synthetic rubbers absorb the aromatic fraction of the binder (Heitzman, 1991). This absorption results in a viscous gel and, consequently, an increase in viscosity of the material. If the temperature is too high or the interaction time is too long, the absorption will continue until the rubber particles are totally dispersed in the medium (Lewandowski, 1994; Abdelrahman, 1996).

According to Abdelrahman & Carpenter (1999), the reaction occurs due to two mechanisms: devulcanization and depolymerization (swelling and degradation, respectively). Both mechanisms are chemical reactions that reduce the molecular weight of the rubber and, consequently, brake down the chemical bonds. In the process of devulcanization, the breakdown of sulfur-sulfur or carbon-sulfur bonds that are formed in the production of tires occur. These two processes are critical to the production of a binder with the storage stability (Billiter, 1997). The depolymerization, however, is a process by which a polymer is broken down into the pure monomer from which it was originally obtained (António, 2007).

According to Heitzman (1991), there are three types of wet processes:

- Batch blending: this process consists of modifying the binder in the pug mill during production;
- Continuous or field blend: in this process, the rubber asphalt is produced with a specific equipment at the job and must be applied immediately after because of its poor storage stability.
- Terminal blending: the crumb rubber is blended with the asphalt cement at a refinery or at a distribution terminal, which results in a stable binder that is relatively homogeneous and may be stocked for later use. In this case, special attention must be given to the amount of rubber used, the correct temperature and the process of homogenization of the material to assure that the reaction occurs evenly in the blend.

To obtain a rubber asphalt with storage stability, the blend must be processed at high temperatures by mixing with high shear, thus, resulting in the depolymerization and devulcanization of the rubber and allowing the reaction of the devulcanized and depolymerized rubber molecules of the binder. This process results in a final product with lower viscosity. In the case of rubber asphalt with no storage stability, there is a swelling of the rubber on the surface of the maltenes in the binder, allowing the use of rubber particles with larger diameter, which results in a material with high viscosity. In this case, neither the depolymerization or the devulcanization occur and low shear agitation is used (Bernucci *et al.*, 2010). A rubber-asphalt obtained by the continuous (left) and another by the terminal blend (right) methods are shown in Figure 2.6, in which the first exhibits a much higher viscosity than the latter.

**Figure 2.6.** Continuous blend rubber asphalt (left) and terminal blend rubber asphalt (right) (Fontes *et al.*, 2006)

The terminal blend process is shown in Figure 2.7. The waste tires (1), after proper processing, result in a crumb rubber powder with a specific gradation, which is then stored in big bags of crumb rubber (2). The crumb rubber is then mixed with the asphalt at high temperatures (3) and, subsequently, mixed with the aggregates (6). The aggregates are stored in silos (4) and are previously heated in the drum (5) before being mixed with the binder. The asphalt mixture is placed in silos (7) and the trucks are loaded to carry the material to the jobsite (8).

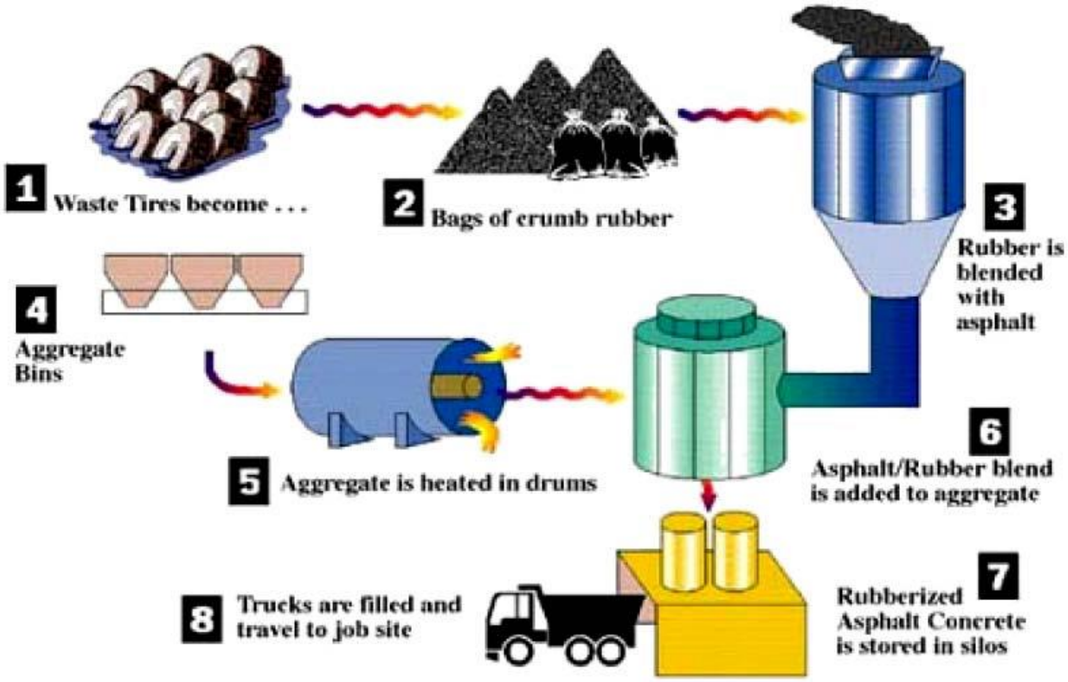



**Figure 2.7.** Terminal blend process (Utah Department of Transportation, 2003)

The continuous or field blend process is shown in Figure 2.8. The equipment to feed and mix the asphalt with the rubber may vary according with the manufacturer, but the processes are all similar. The preheated asphalt binder and the crumb rubber, along with extender oil, when applicable, are fed to a mixing tank (blender) in their proper proportions and mixed to obtain a homogeneous mixture. This mixture is subsequently pumped to a heated tank (reaction vessel), where the interaction occurs between the rubber particles and the asphalt binder for the desired time.

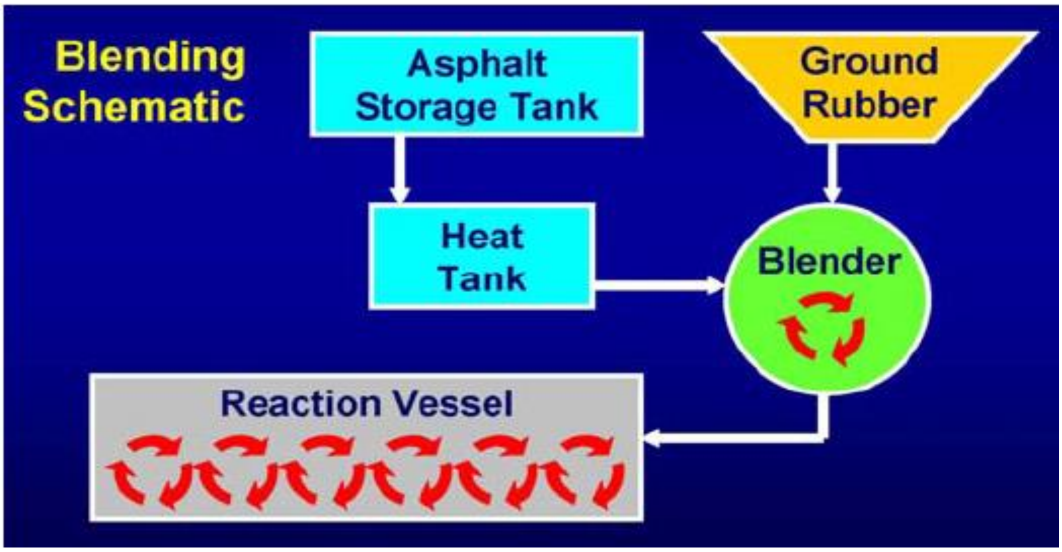



**Figure 2.8.** Continuous blend or field blend process (CALTRANS, 2006)

# 3. COMPARISON OF PERMANENT DEFORMATION AND FATIGUE BEHAVIOR OF NEAT, POLYMER, AND RUBBER ASPHALT BINDERS

## 3.1. Introduction

It is well known that fatigue cracking and rutting are among the major types of distresses to be considered in flexible pavement design (Huang, 1993). In that context, the choice of the asphalt binder for a mixture plays a major role in both the permanent deformation and the fatigue behavior of the final mixture. An increase in traffic and loading conditions experienced in the past years has resulted in a growing interest in improving binder characteristics in order to improve the performance of Brazilian pavements.

As a result, Brazilian agencies and concessionaries have relied on modification of conventional asphalt binders to improve pavement performance. Among the different methods of improving the rheological properties of binders, elastomeric polymer and rubber modification are the most commonly used in Brazil. However, the use of rubber asphalt in Brazil has been restricted to terminal blends until 2011, when the first Brazilian experience with field blending occurred at RJ-122, a heavy traffic highway in the state of Rio de Janeiro.

The rubber asphalt produced by the field blending process results in a highly viscous material with enhanced engineering properties, but requires a specific equipment that is typically installed at the job site, or close to the supplying asphalt plant. However, the lack of information regarding this technique and the performance of the resulting field-blended mixtures in Brazil inhibits a more common use in pavement construction/rehabilitation.

The main purpose of the present study is to assess the permanent deformation and the fatigue behavior of the field-blended rubber asphalt used in RJ-122. Because this is a relatively new technique in Brazil, the characteristics of the rubber asphalt binder will be compared with the ones for typical binders used in Brazil. Thus, the neat binder used for rubber modification (AC 30/45 penetration grade) was used for comparison. In addition, the neat binder was polymer-modified using styrene-butadiene-styrene (SBS), in accordance with the Brazilian specification DNIT 129/2011-EM (DNIT, 2011a).

The binders were evaluated using the multiple stress creep and recovery test (MSCR), which is the latest procedure for assessing binder permanent deformation. Similarly, the Time Sweep and Linear Amplitude Sweep tests were conducted in order to characterize the fatigue behavior of the binders.

## 3.2. Materials

The neat asphalt is a binder of penetration grade 30/45 (AC 30/45), typically used in the Southeastern region of Brazil. The neat binder was modified to obtain a rubber asphalt and a polymer-modified asphalt. The rubber asphalt was obtained by adding 19% of ambient ground crumb rubber modifier (CRM) meeting the Arizona Department of Transportation Type B crumb (ADOT, 2000) and with 100% passing the #16 sieve (1.18 mm). The rubber asphalt binder was obtained directly at the job plant using a combination blender from D&H Equipment after the rehabilitation job of highway RJ-122 in Brazil. A 60-minute reaction time was used for modifying the binder.

The polymer-modified asphalt was obtained by adding 4.2% of a styrene-butadiene-styrene (SBS) polymer. The same neat binder was used for rubber and polymer modification for comparison. The polymer modification was done in the laboratory using a low-shear process, where the binder was kept under a controlled temperature of 170°C for 1 hour. The SBS binder meets the Brazilian specifications for a 65/90-E type polymer, according to the DNIT 129/2011- EM standard for polymer modification (DNIT, 2011a). This specification guarantees a minimum softening point of 65°C and a minimum elastic recovery of 90%.

The physical properties of the neat and modified binders are summarized in Table 3.1. The rubber and polymer modification of the neat binder resulted in an increase in softening point, elastic recovery, and viscosity, as expected. Similarly, the penetration of the neat binder was slightly increased with the addition of the SBS polymer (from 38 to 40), but was decreased for the rubber asphalt (32). In addition, the Brookfield viscosities of the rubber asphalt are significantly larger than the ones exhibited by the SBS binder (4 to 6 times higher depending on the temperature).

**Table 3.1 -** Physical properties of the neat, rubber asphalt and SBS binders

| Property | Units | AC 30/45 | SBS | CRM |
|---|---|---|---|---|
| Penetration | 0.1 mm | 38 | 40 | 32 |
| Softening Point | ºC | 53 | 83 | 70 |
| Elastic recovery | % | - | 91.0 | 86.2 |
| Brookfield Viscosity* | | | | |
| - @ 135°C | cP | 435 (20 rpm) | 1,366 (20 rpm) | 6,637 (20 rpm) |
| - @ 150°C | cP | 215 (50 rpm) | 642 (50 rpm) | 3,887 (20 rpm) |
| - @ 177°C | cP | 77 (100 rpm) | 494 (100 rpm) | 1,975 (20 rpm) |

* Spindle 21 was used for the neat binder and 27 for the modified binders

Frequency and temperature sweep tests were conducted on the three binders using a Dynamic Shear Rheometer (DSR) from TA Instruments Discovery Model HR-3 at a 0.1% strain. Frequency ranged from 0.159 to 15.9 Hz (1.0 to 100.0 rad/s), and temperature ranged from 46°C to 82°C, with an interval of 6°C. Testing at lower temperatures was not possible because the cooling system was not functioning properly.

Rheological characterization was conducted on virgin binders and short-term aging using the rolling thin film oven test (RTFOT). The RTFOT was performed according to ASTM D2872-12 at a temperature of 163°C (ASTM, 2012). However, the asphalt rubber binder did not roll inside the glass bottles during the test. The same was also reported in NCHRP Project 9-10 for modified binders because of their high viscosity (Bahia *et al*. 2001). Thus, the RTFOT was also carried out at 175°C in order to allow for binder rolling and to create the thin film for proper aging. At this temperature, which is closer to the temperature used for mixing the rubber asphalt in the field, the film was achieved in the glass bottle.

Master curves were developed for the dynamic shear modulus (|G*|) and phase angle ($\delta$) for a 46°C reference temperature. Typically, a 25-mm diameter parallel plate geometry with 1-mm gap setting is used for rheological binder testing. However, because of the influence of the crumb rubber particles in the binder, the rubber asphalt binder was tested using a 2-mm gap setting. A higher gap setting (2 mm) for rubber-modified binders has also been used in another study for the same reason (Subhy *et al.*, 2015).The neat and SBS binders were tested using the typical 1-mm gap setting.

The master curves for the dynamic shear modulus (|G*|) and phase angle ($\delta$) are shown in Figure 3.1 for the neat and modified binders at a 46°C reference temperature. Data obtained for the higher temperatures were ignored because there was a lot of variation, possibly because the binders became more fluid at these temperatures.

As expected, the |G*| of the rubber asphalt and SBS are higher than the neat binder for all frequencies tested. However, the increase in dynamic shear modulus for the modified binders, in comparison with the neat binder, is more prominent at lower frequencies. When comparing the modified binders, the rubber asphalt shows higher |G*| than the SBS at lower frequencies. At higher frequencies, the SBS exhibits slightly higher |G*| than the rubber asphalt. The addition of the modifiers result in a reduction of dependency of |G*| on frequency.

In terms of phase angle, the addition of rubber and SBS caused a reduction of phase angle for all the frequencies in relation to the neat binder. The phase angle varied from approximately 59 to 68° for the rubber asphalt and from approximately 68 to 72° for the SBS binder, whereas the neat binder showed phase angles varying from 78 to 87° for the neat binder. The SBS shows higher phase angles than the rubber asphalt for the range of frequencies tested.

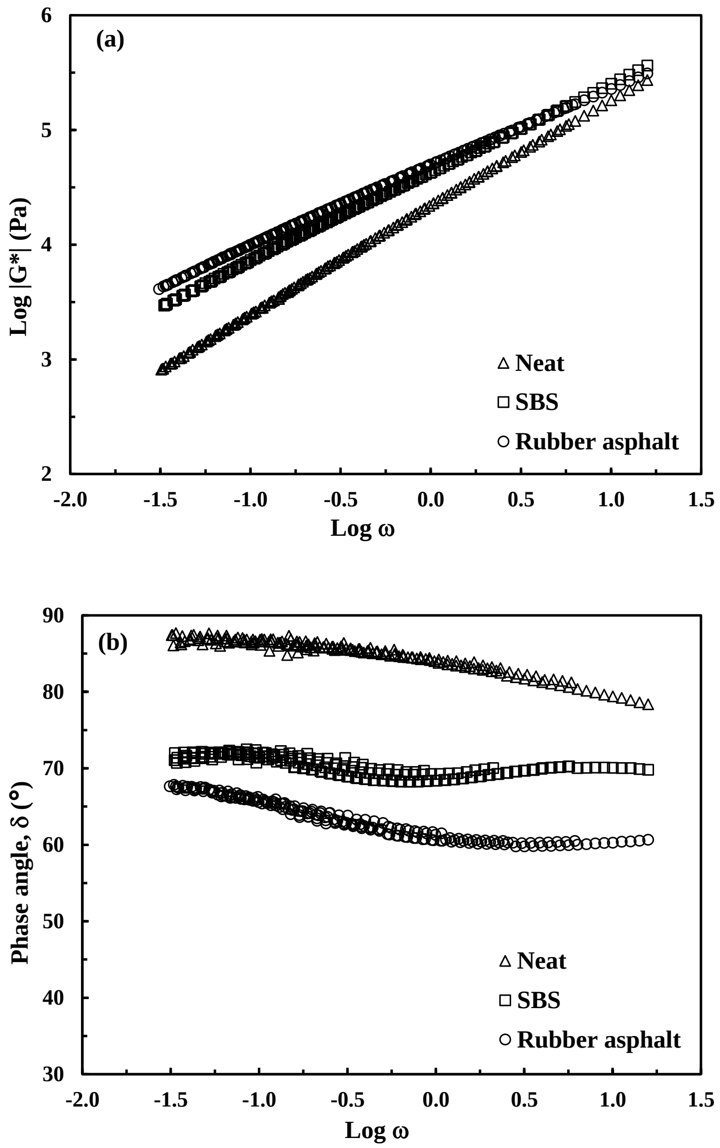



**Figure 3.1.** Master curves for dynamic shear modulus, |G*|, (a) and phase angle, δ, (b) at 46°C

The master curves for the dynamic shear modulus (|G*|) and phase angle (δ) are shown in Figure 3.2 for the neat and modified binders after RTFOT aging (ASTM D2872-12) at a 46°C reference temperature. Short-term aging (in the RTFOT) resulted in higher |G*| for all binders when compared to the virgin condition. However, the master curves after short-term aging show a more prominent difference among the three binders. The decrease in slope for the rubber asphalt is more evident and the rubber asphalt shows higher |G*| at low frequencies, but lower |G*| at high frequencies.

In terms of phase angle, the rubber asphalt exhibited the lowest phase angles for all frequencies, followed by the SBS binder. In general, aging resulted in a decrease in |G*| slope and reduction of phase angle for all binders, but this tendency was stronger for the rubber asphalt. This is an indicative that the rubber asphalt and SBS binders will result in a better rutting resistance and a decreased temperature sensitivity when compared to the neat binder. However, when comparing the modified binders, the rubber asphalt should provide a better resistance to rutting than the SBS binder. Wang *et al*. (2015) observed similar trends when comparing rubber asphalt and SBS binders.

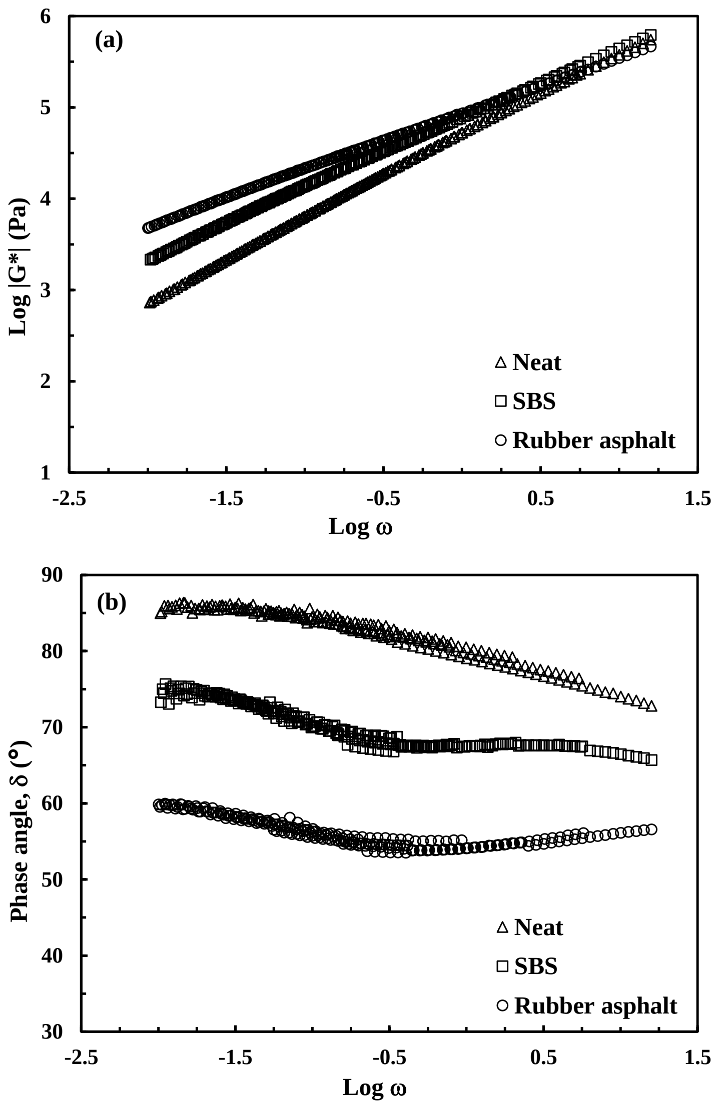



**Figure 3.2.** Master curves for dynamic shear modulus, |G*|, (a) and phase angle, $\delta$, (b) at 46°C after RTFOT aging

## 3.3. Methods

### *3.3.1. Permanent Deformation*

The current Superior Performing Asphalt Pavement System (Superpave) mixture design process, developed under the Strategic Highway Research Program (SHRP) as an initiative to improve mixture design, uses the performance grade (PG) system to select asphalt binders based on the expected pavement temperatures. The approach originally proposed in the grading system based the control of binder rutting on the viscoelastic parameter $|G^*|/\sin\delta$ (Anderson *et al*., 1994; Harrigan *et al*., 1994). Several studies, however, have shown that this parameter is not effective in predicting the true rutting resistance of asphalt binders, especially when evaluating modified binders (Bahia *et al*., 2001; D'Angelo & Dongre, 2002).

The Multiple Stress Creep and Recovery (MSCR) test is an improvement in terms of binder rutting performance prediction (FHWA, 2011). The test is used in the current Superpave PG grading system under the AASHTO M332 test protocol (AASHTO, 2014a). The test is performed according to the AASHTO T350-14 (AASHTO, 2014b) and provides a new high temperature binder specification that predicts rutting performance more accurately, especially in the case of modified binders. Another benefit of using the MSCR test is that the need for additional tests to indicate binder modification are no longer required, such as elastic recovery, toughness and tenacity, and ductility. Furthermore, studies have shown that the non-recoverable creep compliance ($J_{nr}$) parameter from the MSCR test is a better indicator of field rutting performance (D' Angelo, 2009).

The binder resistance to rutting was assessed by determining the $J_{nr}$ parameter using a Dynamic Shear Rheometer (DSR) from TA Instruments Discovery HR-3. The procedure was used in order to rank the binders in terms of permanent deformation, but the applicability of the Jnr parameter as an indicator of field performance for the climate and loading conditions in Brazil needs to be verified by a long-term pavement performance program. Typically, a 25-mm diameter parallel plate geometry with a 1-mm gap setting is used for the MSCR. Because of the influence of crumb rubber particles in the binder, however, the rubber asphalt binder was tested using a 2-mm gap setting. Higher gap settings for rubber-modified binders have also been used in other studies for this reason (Subhy *et al*., 2015). The neat and SBS binders were tested using the typical 1-mm gap setting.

The MSCR tests were performed in accordance with AASHTO T350-14 (AASHTO, 2014b). Sample preparation and apparatus, in accordance with AASHTO T315-12 (AASHTO, 2012), were used for the MSCR test and binders were RTFOT aged. The samples were loaded at a constant stress for 1 s and then allowed to recover for 9 s. Ten creep and recovery cycles were run at 0.1 kPa creep stress followed by ten cycles at 3.2 kPa creep stress. MSCR tests were carried out on binders aged at 163 and 175°C temperatures in the RTFOT. Tests were conducted on 3 replicates for each binder at 58, 64, and 70°C.

### *3.3.2. Fatigue Behavior*

In terms of fatigue behavior, the approach originally proposed by SHRP in the grading system to control binder fatigue is based on the viscoelastic parameter $|G^*|\cdot\sin\delta$ (Bahia *et al*., 2001). However, the NCHRP Project 9-10, which was focused on validating the use of the $|G^*|\cdot\sin\delta$ parameter for characterizing the fatigue behavior of modified binders, found a poor correlation between $|G^*|\cdot\sin\delta$ and the actual pavement resistance to fatigue (Bahia *et al*., 2001). In addition, later research showed that this parameter did not relate well with the accumulation of fatigue damage of mixtures measured through a beam fatigue test under strain-controlled conditions (Shenoy, 2002). The main reason why the $|G^*|\cdot\sin\delta$ parameter does not correlate well with fatigue behavior of mixtures is that the test is conducted in the linear viscoelastic range (low deformation levels), whereas fatigue typically occurs in the non-linear range (high deformations levels) (Bahia *et al*., 1999).

Currently, there are two additional test procedures commonly used for fatigue testing of asphalt binders: the time sweep test and the Linear Amplitude Sweep (LAS) test, both conducted using a Dynamic Shear Rheometer (DSR). The time sweep test was developed during the NCHRP 9-10 project in an attempt to better characterize the fatigue behavior of binders (Bahia *et al*., 2001) and account for non-linear viscoelastic behavior. The applied load amplitude is user-defined, which allows for considering the pavement structure and the expected strain levels in the field. The time sweep test has shown good correlation with the fatigue life of asphalt mixtures when proper conditions are applied (Anderson *et al*., 2001, Martono *et al*., 2007). However, the test may take several days in order to achieve failure.

Later work by Martono & Bahia (2008) suggested that an amplitude sweep procedure was a promising indicative of fatigue performance of asphalt binders and an accelerated procedure for fatigue characterization of asphalt binders was developed (Johnson, 2010). The LAS procedure was later modified in order to include very small increments in loading amplitude every cycle (linearly increasing loads) for smoother crack growth rate. The fatigue behavior was analyzed using the continuum damage approach (Hintz, 2012). The test is divided into two stages: the sample is first tested in shear using a frequency sweep to determine the binder rheological properties using a Dynamic Shear Rheometer (DSR). The sample is then tested using a series of oscillatory load cycles at linearly increasing strain amplitudes at a constant frequency to cause accelerated fatigue damage.

The fatigue law of the following form (Eq. 3.1) is derived from the LAS test.

$$N_f = A\, \gamma_{max}^{-B} \qquad \text{(Eq. 3.1)}$$

where $N_f$ is the number of cycles to failure, $\gamma$ is the applied shear strain and A, B are constants (material dependent).

In this study, binder fatigue resistance was assessed by determining the time sweep and the LAS tests using the DSR from TA Instruments Discovery Model HR-3. The binders were first characterized in terms of their linear viscoelastic behavior and later subjected to the fatigue tests. The fatigue resistance of the binders was conducted using an 8-mm diameter parallel plate geometry with a 2-mm gap setting. The LAS test was conducted in accordance with AASHTO TP101-14 (AASHTO, 2014c). Both the time sweep and the LAS tests were conducted at a temperature of 20°C and a loading frequency of 10 Hz. This configuration was used in order to reduce the duration of the time sweep tests.

Strain sweep tests were previously performed in accordance with AASHTO T315-12 (AASHTO, 2012) in order to determine the linear viscoelastic region for each binder at 20°C. The neat binder exhibited a linear viscoelastic behavior up to 2.2% strain, whereas the viscoelastic limit for the SBS and rubber asphalt binders was 2.6 and 2.7%, respectively. The strain levels for the time sweep tests were determined such that the lowest strain level for all the binders was still in the linear viscoelastic region (2%, 3%, and 4% strain levels). At least 2 samples were tested at each strain level. The tests were interrupted when the initial |G*| was reduced by more than 50%. The 50% reduction in initial stiffness is typically used as the failure criterion for fatigue testing of mixtures in Brazil.

## 3.4. Results and Discussion

### *3.4.1. Permanent Deformation*

The variation of the percent recovery for the 3.2 kPa stress level with change in temperature is shown Figure 3.3 for the three binders. The percent recovery for all binders decreased with increasing temperature. As expected, rubber or polymer modification resulted in an increase in percent recovery in comparison with the neat binder. However, the percent recovery for the rubber asphalt is the highest among the three binders at any given temperature. In addition, the reduction in percent recovery with increasing temperature is the smallest for the rubber asphalt (approximately 30%) when compared with the SBS binder (approximately 60%), which indicates that the rubber asphalt is less sensitive to temperature for the range of temperatures tested.

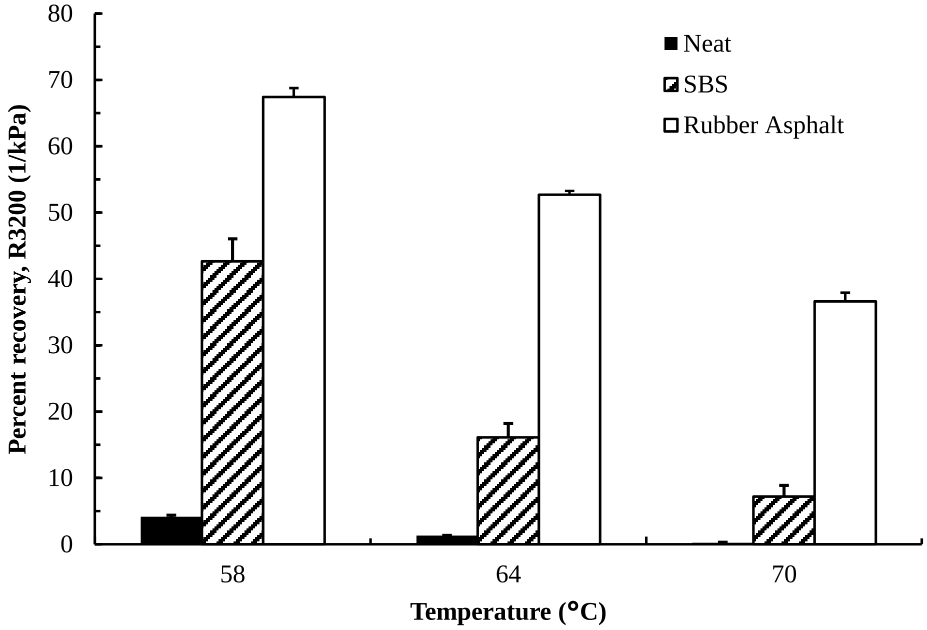



**Figure 3.3.** Change in R3200 with temperature for the neat, rubber asphalt, and SBS binders

The non-recoverable creep compliances ($J_{nr,3200}$) for all binders increase with increasing temperature (Figure 3.4). For any given temperature, the non-recoverable creep compliance for the modified binders is lower than the one exhibited by the neat binder for both stress levels. This shows that the modification of the neat binder with either rubber or polymer results in a binder with a better resistance to rutting.

However, the $J_{nr,3200}$ for the rubber asphalt is significantly lower (from approximately 38 to 55% lower, depending on the temperature) than the ones for the SBS binder, which shows that the rubber asphalt has a better potential to resist rutting among the three binders. The decrease of non-recoverable creep compliance because of rubber modification has also been reported in the literature (Giuliani & Merusi, 2010; Subhy *et al*., 2015). In addition, a larger decrease in $J_{nr,3200}$ for rubber asphalt in comparison to polymer-modified (SBS) and the respective neat binders has also been observed in other studies for similar temperatures (64 to 88°C) (Santagata *et al*. 2015).

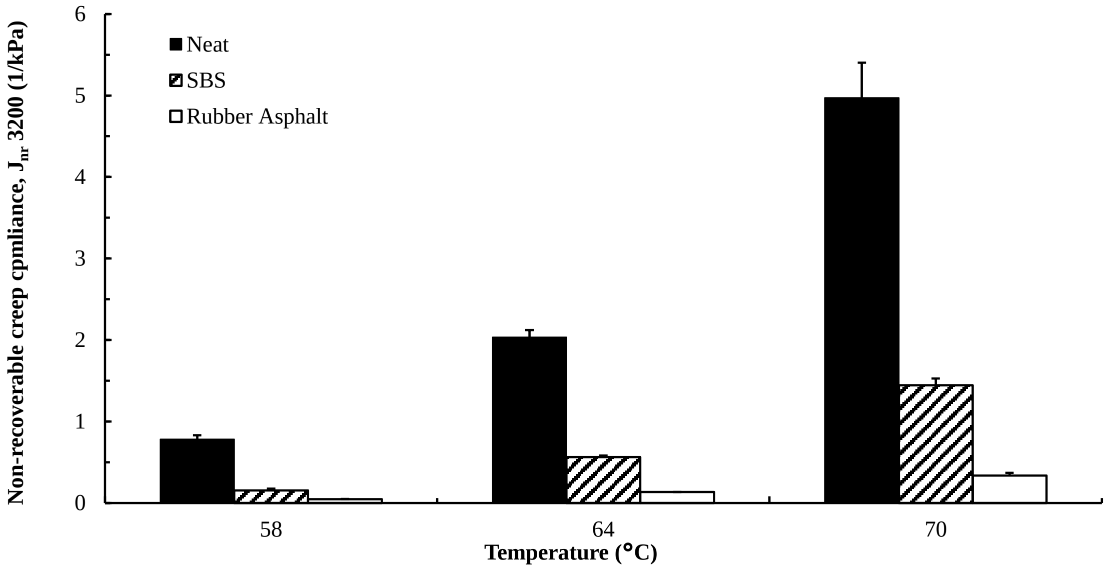



**Figure 3.4.** Change in $J_{nr,3200}$ with temperature for the neat, rubber asphalt, and SBS binders

The MSCR stress sensitivity parameter ($J_{nr,diff}$) for the binders are summarized in Table 3.2 for the three temperatures tested. The neat and SBS binders exhibited adequate sensitivity to stress in all temperatures tested, since the $J_{nr,diff}$ was lower than the 75% limit. The rubber asphalt binder, however, exhibited $J_{nr,diff}$ over the 75% limit for all temperatures tested, which indicates that the binder is stress dependent. Other studies have also observed a stress dependent behavior of rubber asphalt binders by means of the MSCR test (Martins *et al.*, 2011; Santagata *et al.*, 2015; Willis *et al.*, 2012). Even though the binder showed a stress dependent behavior, the $J_{nr}$ values are very small (ranging from 0.05 to 0.34 1/kPa for the higher stress level), indicating that the mixture should have a small rutting potential in the field.

**Table 3.2** - Stress sensitivity parameter ($J_{nr, diff}$) for the neat, rubber asphalt, and SBS binders

| **Temperature (°C)** | **$J_{nr, diff}$ Neat** | | **$J_{nr, diff}$ SBS** | | **$J_{nr, diff}$ Rubber Asphalt** | |
|---|---|---|---|---|---|---|
| | **Avg.** | **CV** | **Avg.** | **CV** | **Avg.** | **CV** |
| 58 | 4.7% | 29% | 64.1% | 18% | 83.2% | 14% |
| 64 | 8.2% | 7% | 31.7% | 32% | 159.7% | 5% |
| 70 | 11.5% | 17% | 28.9% | 23% | 346.2% | 14% |

**where "avg." is the average of three tests and "cv" is the coefficient of variation**

### 3.4.2. *Binder Fatigue*

The fatigue life from the time sweep tests (TST) are shown in Figure 3.5 for the strain-controlled tests performed on the neat, rubber asphalt, and SBS binders at 20°C and at a frequency of 10 Hz (strains varying from 2 to 4%). The fatigue life ($N_f$) for the modified binders, determined as the number of cycles for a 50% reduction of the initial dynamic shear modulus, are higher than the neat binder for the range of strains tested. However, the fatigue life of the rubber asphalt is the highest among the three binders.

Micaelo *et al*. (2015) also observed a higher fatigue life for an SBS-modified binder (4%) in relation to the neat binder in the time sweep tests with a 50% reduction of |G*| failure criterion. The tests were conducted for strain levels between 1.2-2.0%. Similarly, another research has reported a higher fatigue life of the rubber modified binder in relation to the neat binder using the time sweep tests (Giuliani & Merusi, 2010). Nuñez (2013) compared the fatigue life of a rubber (14%) and a SBS-modified (4%) binder, among others, through time sweep tests at different temperature and test conditions. The rubber asphalt also exhibited a higher fatigue life in time sweep tests in comparison with the SBS binder irrespective of the testing condition.

Fatigue laws were developed based on the results of replicate specimens for the neat and modified binders (Table 3.3). The coefficients of determination for the fatigue laws in this study varied from 0.980 (rubber asphalt) to 0.999 (neat). The rubber asphalt exhibited a fatigue life of approximately 500,000 cycles for the lowest strain level (2 %), whereas the fatigue life of the SBS binder was around 61,000 cycles for the same strain level. The neat binder showed the lowest fatigue life of the three binders, as expected, with just below 50,000 cycles. Similar trends were observed for the remaining strain levels.

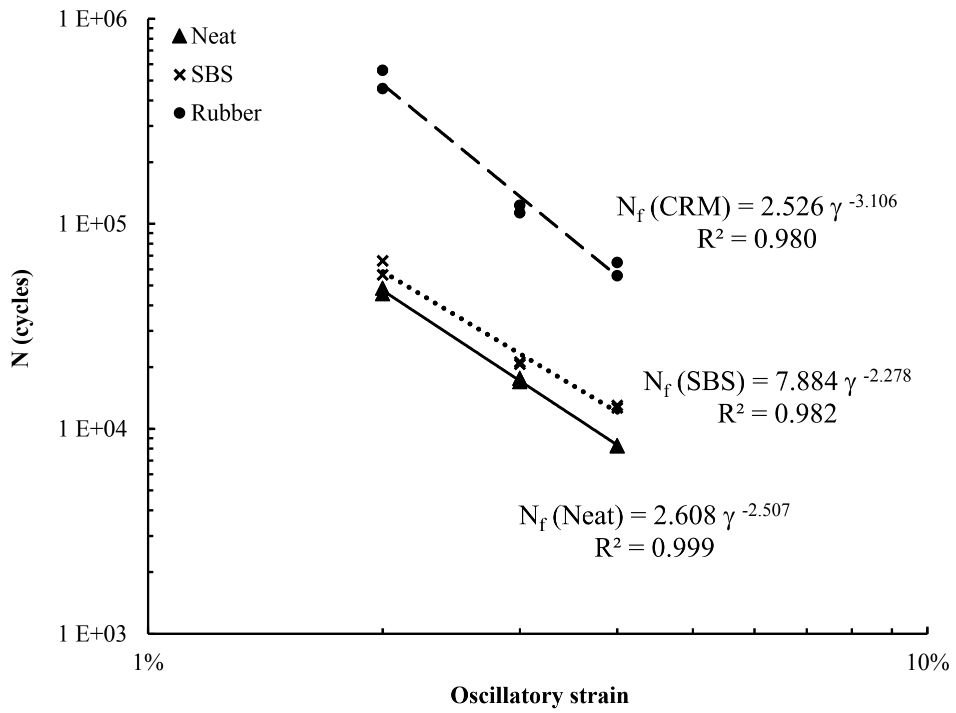



**Figure 3.5.** Fatigue laws for the neat, rubber asphalt, and SBS binders derived from the TST

**Table 3.3** - Results of the Time Sweep Tests for the neat, rubber asphalt, and SBS binders at 20°C and 10 Hz

| Binder | Strain (%) | $N_f$ (Cycles) | | a | b | $R^2$ |
|---|---|---|---|---|---|---|
| | | Mean | Coefficient of variation | | | |
| Neat | 2 | 47,104 | 4.5% | 2.608 | -2.507 | 0.999 |
| | 3 | 17,333 | 2.7% | | | |
| | 4 | 8,268 | 0.5% | | | |
| Rubber asphalt | 2 | 509,675 | 14.6% | 2.527 | -3.106 | 0.980 |
| | 3 | 118,391 | 5.9% | | | |
| | 4 | 60,434 | 10.5% | | | |
| SBS | 2 | 61,227 | 11.0% | 7.884 | -2.278 | 0.982 |
| | 3 | 20,931 | 1.6% | | | |
| | 4 | 12,811 | 1.9% | | | |

The change in shear stress as a function of the applied shear strain during the Linear Amplitude Sweep (LAS) test is shown in Figure 3.6 for sample A of the neat and modified binders. The neat binder shows a fast increase in shear stress with increasing strain level, reaching a peak value of 900 kPa at approximately 8% strain. The shear stress then falls sharply until the 15% strain level and then slowly approaches zero until approximately 25% strain. The behavior exhibited by the SBS binder is similar to the one shown by the neat binder, with the exception that the peak strain was lowered to approximately 800 kPa and the strain falls more gradually after reaching its peak. In other words, the stress-strain curve was slightly flattened.

The rubber asphalt, on the other hand, shows a different stress-strain behavior than the neat and the SBS binders. The peak value of 450 kPa (approximately 50 and 56% lower than the neat and SBS binders, respectively) is slowly reached at an approximate 12% strain level and the shear stress decreases is slowly dissipated until the end of the test (30% strain level). In addition, the shear stresses are still at about 200 kPa at the high strain level. Similar stress-strain behavior was also observed by (Micaelo *et al*., 2015) when comparing neat and modified binders.

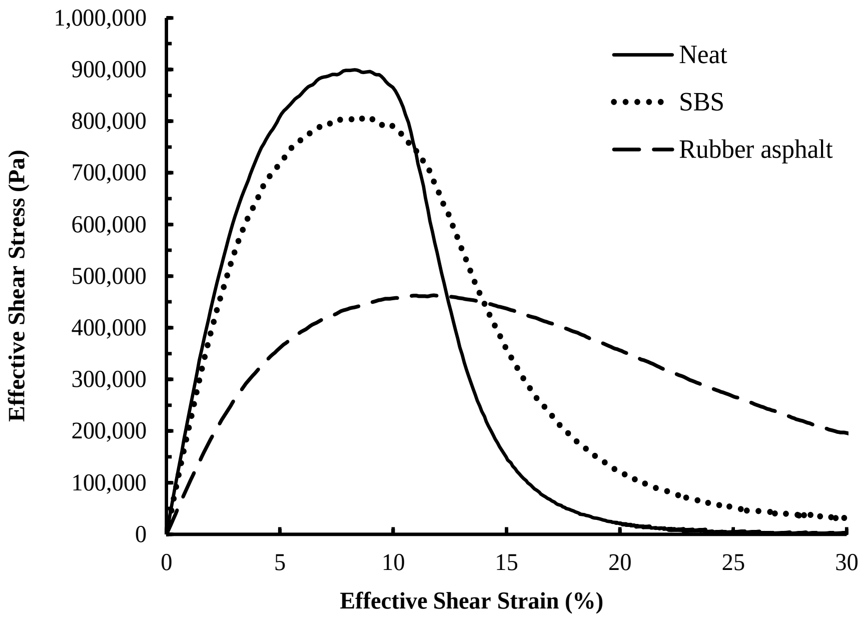



**Figure 3.6.** Stress-strain curves for the neat and modified binders in the LAS test

In order to determine the damage characteristic curve for the binder in the LAS test, the $|G^*|\cdot\sin\delta$ was taken as the integrity parameter, C, and compared to the development of the damage intensity, D. The damage characteristic curves for sample A of the three binders are shown in Figure 3.7. As expected, the integrity of all binders decreases for increasing damage intensity. However, the decrease in integrity is the fastest for the neat binder when compared to the modified binders.

The SBS binder shows a similar decrease in integrity until a value of 0.3, after which the decrease in integrity is slower than the one for the neat binder.

The neat and SBS binders reach zero integrity at a damage intensity of 250 and 325, respectively, whereas the rubber asphalt still exhibits some integrity (roughly 0.1) at the end of the test at a damage intensity of about 450. Thus, the rubber asphalt exhibited a better tolerance to fatigue than the other binders, since higher integrities were retained for higher damage levels. The rubber asphalt failed at a damage intensity of 94, whereas the neat and SBS binders failed at damage intensities of 58 and 62, respectively.

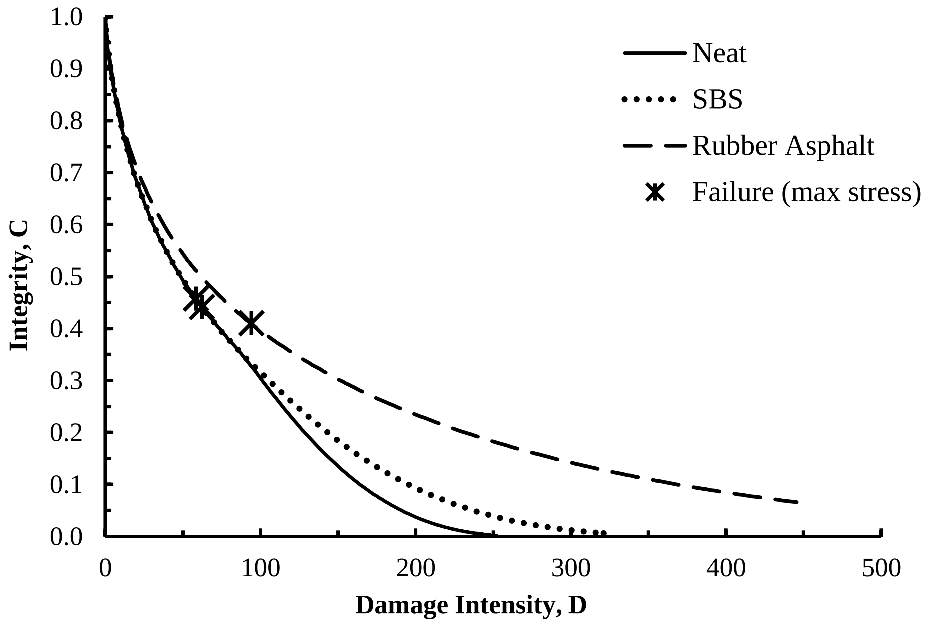



**Figure 3.7.** Integrity versus damage intensity (damage characteristic curve) for the binders in the LAS test

The model inputs for the LAS test are summarized in Table 3.4 for the neat and modified binders along with fatigue life for two strain levels (2 and 4% strain). The model parameters A and B for the LAS are similar for replicate specimens of the same binder. As expected, the A parameters for the modified binders are larger than the ones obtained for the neat binder. Typically, an increase in A parameter indicates an increase in fatigue resistance. The addition of SBS resulted in an average 2.4 and 1.8-fold increase of fatigue life for 2 and 4% strain levels, respectively. The addition of rubber, however, resulted in larger increases in fatigue life for the 2% (average 11.4-fold increase) and 4% strain levels (average 7.6-fold increase).

An increase in parameter A and a decrease in parameter B due to rubber and SBS modification was also observed by Nuñez (2013) in the LAS test for different testing conditions (aging, temperature, and failure criterion). In addition, the increase on A and decrease in B was higher for the rubber asphalt in comparison with the SBS-modified binder, resulting in higher fatigue lives for the rubber asphalt.

**Table 3.4 -** Model inputs for the LAS test for the neat and modified binders

| Binder | Sample | A | B | $R^2$ | 2% $N_f$ | 4% $N_f$ |
|---|---|---|---|---|---|---|
| Neat | A | 1.621E+05 | -2.845 | 0.993 | 22,568 | 3,141 |
| | B | 1.629E+05 | -2.765 | 0.991 | 23,961 | 3,525 |
| SBS | A | 4.805E+05 | -3.126 | 0.982 | 55,052 | 5,734 |
| | B | 5.266E+05 | -3.260 | 0.974 | 54,948 | 6,308 |
| Rubber asphalt | A | 2.893E+06 | -3.346 | 0.980 | 284,526 | 27,979 |
| | B | 2.713E+06 | -3.452 | 0.981 | 247,936 | 22,660 |

The average increase in fatigue life due to the presence of the modifiers is shown in Figure 3.8 in the LAS test (up to 30% strain level). The average increase in fatigue life due to the rubber is highest at lower strain levels, ranging from 17.2-fold (1% strain) to 2.3-fold (30% strain). The SBS-modified binder, on other hand, showed increases in fatigue life ranging from 2.9 to 1.0 times for 1 and 30% strain levels, respectively.

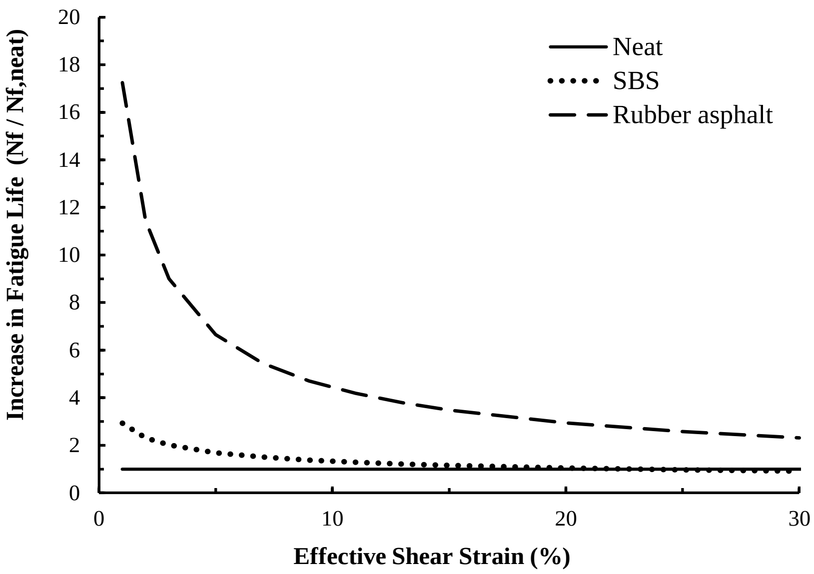



**Figure 3.8.** Average increase in fatigue parameter, $N_f$ (normalized to 1 million ESALs), as a function of the applied shear strain in the LAS test

The average number of cycles to failure in the TST and LAS tests are shown in Figure 3.9 for the neat and modified binders. In general, the TST resulted in higher fatigue lives for all strain levels. The average number of cycles to failure in the TST was approximately 2.3 times higher than the number of cycles in the LAS test for the neat binder, 2.0 times higher than the number of cycles in the LAS test for the rubber asphalt, and an average 1.5-increase in fatigue life for the TST for the SBS binder. It is important to note that different criteria were used for determining failure in the TST and LAS tests, which may account for the observed differences in fatigue life. However, regardless of the methodology used to assess the fatigue life of the binders, the rubber asphalt exhibited a superior resistance to fatigue than the neat and SBS binders for the range of strain levels tested.

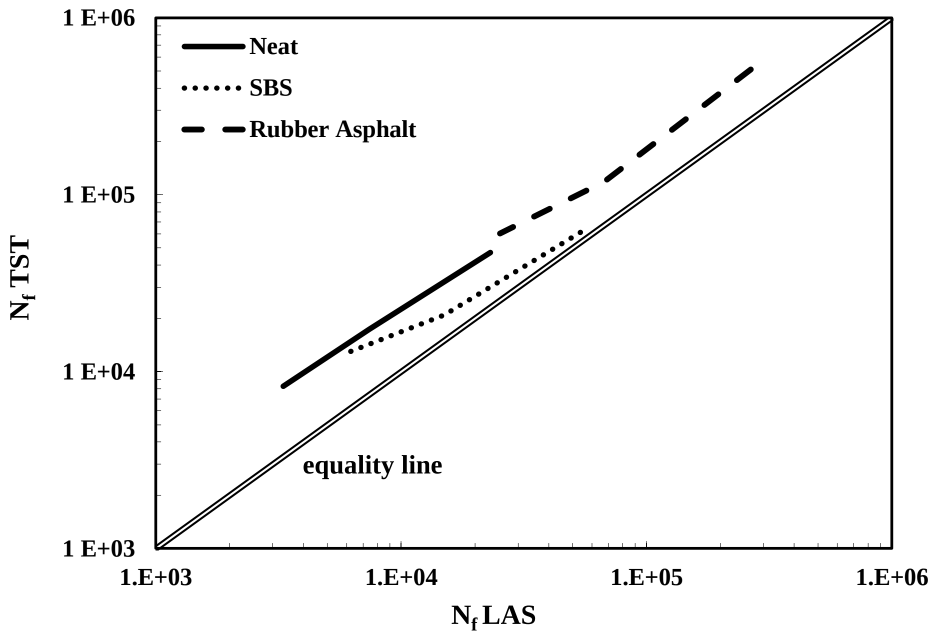



**Figure 3.9.** Comparison of fatigue life obtained in TST and LAS tests

## 3.5. Conclusions

This study was conducted in order to evaluate the permanent deformation and fatigue behavior of a neat binder, a field-blended rubber asphalt, and a SBS-modified binder. The binders were evaluated using the MSCR procedure for permanent deformation and the TST and LAS procedures for fatigue behavior. The following conclusions can be drawn:

- There was a difference in binder aging procedure, since the rubber asphalt was aged in the RTFOT at 175°C for creating the film for proper aging.
- Modification of the neat binder (AC 30/45 penetration grade) resulted in an increase in percent recovery in the MSCR test and the percent recovery for rubber asphalt was the highest among the three binders at any given temperature.
- The non-recoverable creep compliances ($J_{nr,3200}$) for the modified binders are lower than the one exhibited by the neat binder for both stress levels (0.1 and 3.2 kPa) at any given temperature, which shows the modification of the neat binder with either rubber or polymer results in a binder with a better resistance to rutting.
- The $J_{nr,3200}$ for the rubber asphalt is significantly lower (from approximately 38 to 55% lower, depending on the temperature) than the ones for the SBS binder.
- The neat and SBS binders showed adequate sensitivity to stress in all temperatures tested, since the $J_{nr,\ diff}$ sensitivity parameter was lower than the 75% limit. The rubber asphalt binder, however, exhibited $J_{nr,\ diff}$ over the 75% limit for all temperatures tested. Even though the binder showed a stress dependent behavior, the $J_{nr}$ values are very small (ranging from 0.05 to 0.34 1/kPa for the higher stress level), indicating a small rutting potential in the field.
- Binder modification resulted in a better fatigue behavior than the neat binder according to the time sweep and linear amplitude sweep test. In addition, rubber modification resulted in the best fatigue behavior among the binders tested.
- Rubber modification resulted in a binder with better stress-strain behavior and the damage characteristic curves for the binders show that the rubber asphalt has a better response to damage than the neat and SBS binders (higher integrity for the same damage level).
- Fatigue life in the TST was consistently higher than fatigue life in the LAS test, probably because different criteria were used for determining failure in each test; however, ranking of the three binders remained constant.

- Results showed that the field-blended rubber asphalt has a better resistance to fatigue and permanent deformation than the neat and SBS-modified binders.

# 4. FIELD AND LABORATORY PERMANENT DEFORMATION OF A RUBBER ASPHALT MIXTURE

## 4.1. Introduction

The comprehension of pavement distresses, their respective causes, and the ability to predict them are essential tools for pavement management and design. Permanent deformation or rutting is one of the most important forms of distress in asphalt pavements and has been designated as a primary distress mechanism and one of the most important criteria in flexible pavement design (Parker & Brown, 1993). In asphalt mixtures, permanent deformation is the unrecoverable, cumulative deformation that occurs in the wheel paths due to repeated traffic loading, principally, at high temperatures.

The factors affecting rutting of asphalt mixtures are the following (adapated from Sousa *et* al.,1991):

- Aggregates: gradation, shape, surface texture, size, and abrasion.
- Binder: binder stiffness and binder modification.
- Mixture: binder content, air void content, VMA, and method of compaction (field or laboratory).
- Test/field conditions: temperature, state of stress/strain, load repetitions, and water (in case of moisture sensitive mixtures).

Asphalt binders are a major component in the development of mixture rutting due to their temperature and time dependent behavior, especially at high in-service temperatures and under heavy vehicles moving slowly (Delgadillo & Bahia, 2010). In Brazil, a substantial increase in traffic in terms of repetitions (ESALS of over 100 million for a ten-year design period are common in main highways) and loads (new vehicle configurations with up to 9 axles and loads of up to 80 tons) has occurred in the past years. These elevated traffic conditions, along with the high temperatures in Brazil, result in a critical condition for pavements in terms of permanent deformation.

There is a growing interest in improving binder and, consequently, mixture performance in order to mitigate permanent deformation in Brazilian pavements. The use of rubber asphalt mixtures presents itself as an alternative to conventional asphalt mixtures due to improved resistance to rutting and lower pavement maintenance costs (CALTRANS, 2006). In particular, wet process-high viscosity binders, with up to 20% of crumb rubber modifier (CRM) contents to increase the binder viscosity, can be used in gap-graded gradations for improved performance (Zeiada *et al*., 2014).

This study aims to assess the permanent deformation of a field-blended rubber asphalt, recently used in a gap-graded mixture in Brazil for the first time. The permanent deformation of the field-blended rubber asphalt binder was first evaluated and compared to the neat binder (AC 30/45 penetration grade). Then, the asphalt mixture was tested in the laboratory and in the field, in order to characterize the permanent deformation at different levels (binder, mixture, and field).

The binders (neat and rubber asphalt) were evaluated using the Superpave grading system, in terms of permanent deformation control, through the viscoelastic parameter $|G^*|/\sin\delta$ and the Multiple Stress Creep and Recovery (MSCR) test. The permanent deformation of the rubber asphalt mixture was assessed by performing a LCPC wheel tracking test on specimens compacted in the laboratory as well as field-collected specimens. Field performance of the rubber asphalt mixture was investigated by means of periodic monitoring of a test section and by employing a heavy vehicle traffic simulator for predicting future performance.

### 4.2. Materials

A neat asphalt binder of penetration grade 30/45 (AC 30/45), typically used in Brazil, was modified using 19% of crumb rubber modifier (ambient grinding) meeting the Arizona Department of Transportation Type B crumb (ADOT, 2000) and with 100% passing the #16 sieve (1.18 mm). The rubber asphalt binder was obtained directly at the job plant using a combination blender from D&H Equipment after the rehabilitation job of highway RJ-122 in Brazil. A 60-minute reaction time was used for modifying the binder with agitation inside the tank to ensure the material is reacting uniformly and produces a consistent blend. The physical properties of the neat and rubber asphalt binder are summarized in Table 4.1. The binder is classified as a PG

88-22 and was well above the specifications of a PG 58-10 for the RJ-122 project (PG 64-10 with grade bumping) (Balaguer, 2012).

**Table 4.1 -** Physical properties of the neat and CRM binders

| Property | Units | Neat | CRM |
|---|---|---|---|
| Penetration | 0.1 mm | 38 | 32 |
| Softening Point | ºC | 53 | 70 |
| Elastic recovery | % | - | 86.2 |
| Brookfield Viscosity* | | | |
| - @ 135°C | cP | 435 (20 rpm) | 6,637 (20 rpm) |
| - @ 150°C | cP | 215 (50 rpm) | 3,887 (20 rpm) |
| - @ 177°C | cP | 77 (100 rpm) | 1,975 (20 rpm) |

* Spindle 21 was used for the neat binder and 27 for the rubber asphalt

The mix gradation is a gap-graded similar to that specified by the Arizona Department of Transportation using granite aggregate. The mean grain size distribution curve is shown in Table 4.2 along with ADOT specifications. The gap-graded mixture was collected in the field straight from the combination blender after the rehabilitation job in RJ-122 and brought to the laboratory for testing. Field slabs from a section of highway RJ-122 that was not subjected to traffic were also collected for laboratory testing.

**Table 4.2 –** Grain size distribution curve for the RJ-122 test section

| Sieve | Diameter (mm) | Mean Curve (%passing) | ADOT Specifications | |
|---|---|---|---|---|
| | | | Lower limit | Upper limit |
| 3/4" | 19.000 | 100 | 100 | 100 |
| 1/2" | 12.500 | 87.4 | 80 | 100 |
| 3/8" | 9.520 | 69.8 | 65 | 80 |
| Nº 4 | 4.800 | 22.7 | 28 | 42 |
| Nº 8 | 2.360 | 13.0 | 14 | 22 |
| Nº 200 | 0.075 | 2.1 | 0 | 2.5 |

The volumetric properties of Marshall-compacted samples and the optimum binder content for the field-blended rubber asphalt are presented in Table 4.3 for a 75-blow energy. The mixture has an asphalt content of 8.0 % and air voids of 5.6%. Because there isn´t a Brazilian specification for this mixture, the results are compared with the Brazilian specification DNIT 112/2009 for a rubber asphalt terminal blend or “wet process” (DNIT, 2011a).

**Table 4.3 –** Volumetric properties of the field-blended rubber asphalt mixture

| Property | Mixture | Limits (DNIT 112/2009) |
|---|---|---|
| Air Voids (%) | 5.6 | 4-6 |
| Optimum asphalt content (%) | 8.0 | 5-8 |
| Voids in Mineral Aggregate (%) | 23.1 | >14 |
| Stability (kN) | 7.28 | >7.0 |

## 4.3. Methods

A series of laboratory and field tests were conducted in order to assess the permanent deformation of the field-blended rubber asphalt used in the gap-graded mixture. The idea was to compare the permanent deformation of a field-blended rubber asphalt binder in relation to the neat binder and then investigate the mixture performance in the laboratory and in the field, in order to characterize the permanent deformation at different levels, as described in the following items.

### *4.3.1. Asphalt Binder*

The current Superior Performing Asphalt Pavement System (Superpave) mixture design process, developed under the Strategic Highway Research Program (SHRP) as an initiative to improve mixture design, uses the performance grade (PG) system to select asphalt binders based on the expected pavement temperatures. The approach originally proposed in the grading system based the control of binder rutting on the viscoelastic parameter $|G^*|/\sin\delta$ (Anderson *et al*., 1994; Harrigan *et al*., 1994). Several studies, however, have shown that this parameter is not effective in predicting the true rutting resistance of asphalt binders, especially when evaluating modified binders (Bahia *et al*., 2001; D’Angelo & Dongre, 2002).

The Multiple Stress Creep and Recovery (MSCR) test is an improvement in terms of binder rutting performance prediction (FHWA, 2011). The test is used in the current Superpave PG grading system under the AASHTO M332 test protocol (AASHTO, 2014a). The test is performed according to the AASHTO T350-14 (AASHTO, 2014b) and provides a new high temperature binder specification that predicts rutting performance more accurately, especially in the case of modified binders. Another benefit of using the MSCR test is that the need for additional tests to indicate binder modification are no longer required, such as elastic recovery, toughness and tenacity, and ductility. Furthermore, studies have shown that the non-recoverable creep compliance ($J_{nr}$) parameter from the MSCR test is a better indicator of field rutting performance (D' Angelo, 2009).

Thus, binder resistance to rutting was assessed in this study by determining both the $|G^*|/\sin\delta$ and $J_{nr}$ parameters using a Dynamic Shear Rheometer (DSR) from TA Instruments Discovery Model HR-3. Typically, 25-mm diameter parallel plate geometry with 1-mm gap setting is used for rheological binder testing. However, because of the influence of the crumb rubber particles in the binder, the rubber asphalt binder was tested using a 2-mm gap setting. A higher gap setting (2 mm) for rubber-modified binders has also been used in another study for the same reason (Subhy *et al*., 2015). The neat binder was tested using the typical 1-mm gap setting.

Frequency and temperature sweep tests were conducted on the neat and rubber asphalt binders using the DSR at a 0.1% strain. Frequencies ranged from 0.159 to 15.9 Hz (1.0 to 100.0 rad/s) and temperatures ranged from 46°C to 82°C, with an interval of 6°C. Testing at lower temperatures was not possible because the cooling system was not functioning properly. Binders were tested under virgin and rolling thin film oven test (RTFOT) aged conditions. The RTFOT was performed according to ASTM D2872-12 at a temperature of 163°C (ASTM, 2012).

However, the rubber asphalt binder did not roll inside the glass bottles during the test. The same was also reported in NCHRP Project 9-10 for modified binders because of their high viscosity (Bahia *et al*., 2001). Thus, the RTFOT was also carried out at 175°C in order to allow for binder rolling and to create the thin film for aging. At this temperature, which is closer to the temperature used for mixing the rubber asphalt in the field, the film was achieved in the glass bottle. Master curves were developed for the dynamic shear modulus ($|G^*|$) and the phase angle ($\delta$) for a 46°C reference temperature. The $|G^*|/\sin\delta$ parameter was calculated for a 1.59 Hz (10 rad/s) angle.

The MSCR tests were performed in accordance with AASHTO T350-14 (AASHTO, 2014b). Sample preparation and apparatus, in accordance with AASHTO T315-12 (AASHTO, 2012), were used for the MSCR test and binders were RTFOT aged. The samples were loaded at a constant stress for 1 s and then allowed to recover for 9 s. Ten creep and recovery cycles were run at 0.1 kPa creep stress followed by ten cycles at 3.2 kPa creep stress. MSCR tests were carried out on binders aged at 163 and 175°C temperatures in the RTFOT.

### *4.3.2. Asphalt Mixture*

The purpose of performing laboratory tests on asphalt mixtures is to attempt to predict the field behavior. The closer the conditions are in the laboratory (stress state, climatic conditions, moisture, compaction, etc.), the closer the results will be in relation to the expected field performance. Among the tests available for assessing the permanent deformation of asphalt mixtures, are the fundamental tests (unconfined uniaxial or confined triaxial stress tests with repeated loading) or tests that attempt to simulate field conditions by means of a wheel-track test (SHELL, 2003), which can be performed on slabs or actual pavement cross-sections.

The wheel-track test, first developed by the *Laboratoire Central des Ponts et Chaussées* (LCPC), is used to study permanent deformation of asphalt mixtures by measuring rutting of prismatic specimens subjected to repeated wheel loading for up to 100,000 cycles at high temperatures. According to the French specifications (LPC, 2007), the permanent deformation of an asphalt mixture cannot exceed 5% to 10% (depending on gradation, type of mixture, and application) after 30,000 cycles at a 60°C test temperature in order to guarantee good rut resistance in the field.

A study in Brazil showed that 5% of permanent deformation at 30,000 cycles is more adequate for heavy traffic in the Brazilian highways (Moura, 2010). A Colorado DOT study of thirty-three sites with different permanent deformation performances, as well as traffic and climate conditions, showed that the LCPC wheel-track test may be used to distinguish between good and poor field rutting performance (Aschenbrener, 1992).

A review of U.S. loaded wheel testers (Cooley *et al*., 2000) has shown, among other conclusions, that:

- Results obtained from wheel-tracking devices correlate reasonably well to actual field performance when the in-service loading and environmental conditions of that location are considered.
- Wheel tracking devices can reasonably differentiate between binder performance grades.
- Wheel tracking devices, when properly correlated to a specific site's traffic and environmental conditions have the potential to allow the user agency the option of a pass/fail or "go/no go" criteria.

The permanent deformation of the gap-graded asphalt mixture obtained using the rubber asphalt binder was evaluated by means of the LCPC wheel-track test. Specimens were prepared in the laboratory for the LCPC wheel-track test according to the European specification EN 12697-33 (CEN, 2007b). Prismatic specimens were compacted using a metal frame that resulted in 180 mm wide, 500 mm in length and 50 mm thick specimens. The mixture was short-term aged for 2 hours at 135°C before compaction, than cured for 2 to 3 days prior to testing. The pavement slabs collected after the rehabilitation job of RJ-122 were sawed to the aforementioned dimensions before testing.

Testing was conducted in accordance with European specification EN12697-22 at 60°C (CEN, 2007a). Two specimens, placed in each side of the equipment, were tested simultaneously. A 1-Hz load cycle was applied in each direction and permanent deformation was measured at different locations at 0, 100, 300, 1,000, 3,000, 10,000, 20,000, and 30,000 cycles.

### *4.3.3. Field Performance*

As mentioned earlier, extensive binder and mixture testing are typically performed in the laboratory in an attempt to predict performance under conditions as close as possible to the ones expected in the field in order to predict pavement performanceEven though laboratory testing provides us with good estimates of field behavior, several factors may affect in situ performance, such as climatic and loading conditions.

Long-term pavement performance is one way to monitor field behavior and assess performance under local conditions. Even though monitoring field performance eliminates possible discrepancies between lab and field, many years of data collection are necessary in order to gather enough information to draw any conclusions. One way to provide this link between laboratory evaluation of materials used in pavement layers and the field behavior of these materials in pavement structures is via full-scale accelerated pavement tests (f-sAPT) (Steyn, 2012).

According to Metcalf (1996), accelerated pavement test can be defined as “the controlled application of a prototype wheel loading, at or above the appropriate legal load limit to a prototype or actual, layered, structural pavement system to determine pavement response and performance under a controlled, accelerated, accumulation of damage in a compressed time period.” Thus, heavy vehicle simulators can be used for evaluating new road concepts and maintenance strategies and provide the option of performing a life cycle cost analysis, which, in turn, can decrease both costs and environmental impacts (Saevarsdottir *et al*., 2014). Among other applications, f-sAPT can be used to assess the permanent deformation of in-service pavements. Studies using f-sAPT have been conducted in order to understand the permanent deformation behavior of paving materials (White *et al*., 1999; Ahmed & Erlingsson, 2013).

Field permanent deformation of the field-blended rubber asphalt mixture was determined through periodical pavement monitoring along with accelerated pavement tests using a heavy vehicle simulator. The section selected for further evaluation and monitoring of highway RJ-122 was between the 34.5 and 35.0-kilometer marks. A Pavement Scanner was used to determine the maximum permanent deformation in the wheel paths after 3 and 4 years of service life.

The Pavement Scanner is a vehicle mounted with a Laser Crack Measurement System (LCMS) that allows the automatic detection of cracks and the evaluation of rutting, macrotexture, and other road surface features. Detailed information about the

Pavement Scanner can be found elsewhere (Fialho, 2015). Rutting was determined using 4,000 points to determine the maximum rut depth in a lane at every 1 m in each wheel track. Rut depth was defined as the maximum rut between the two wheel paths for each direction.

A representative test section was chosen for f-sAPT using a full scale, mobile traffic simulator (FSMTS) from *Simular Tecnologia do Pavimento Ltda*. The FSMTS was positioned at the northbound 35.035 kilometer mark. The FSMTS has a total length of 25 meters and width of 3 meters, weighing approximately 50,000 kg. The equipment consists of a dual-wheel half-axle that can apply loads up to 100 kN, which corresponds to an equivalent dual-wheel single axle load of 200 kN. The half-axle moves along an axis either directional (8,000 cycles/day) or bidirectional (16,000 cycles/day) with longitudinal displacements of up to 8.6 meters and transversal displacements of up to 1 meter. In addition, the FSMTS is equipped with sensors to automatically record air humidity, ambient temperature (inside the FSMTS and outside), pavement temperature, load applied, and number of passages.

Tests were conducted using a bidirectional 78.1 kN load, which is approximately 45% higher than the allowable load of 53.8 kN for this type of axle according to the Brazilian legislation at the time. Full scale accelerated pavement tests were conducted by applying 150,000 cycles of a 78.1 kN (156.2 kN on a single axle) load. The load applications yielded 2.47 million equivalent single axle loads (ESALS) according to the American Association of State and Highway Transportation Officials (AASHTO). The f-sAPT simulated approximately 7 years of traffic for RJ-122, which is the typical time interval for overlaying existing pavements in Brazil (7 to 8 years).

For determining pavement rutting under the FSMTS, a rut-depth measurement device was fabricated using a similar concept to that of a straightedge. Rut depths were measured at 0.1 m intervals and the maximum rut-depth for the 7-meter long test section was determined.

## 4.4. Results and Discussion

### *4.4.1. Asphalt Binder*

The master curves for the dynamic shear modulus (|G*|) and phase angle (δ) are shown in Figure 4.1 (a) and (b), respectively, for the neat and rubber asphalt binders at a 46°C reference temperature. Master curves are shown both for the virgin binder as well as for the binder after RTFOT. Data obtained for the higher temperatures were ignored because there was a lot of variation, possibly because the binders became more fluid at these temperatures. As expected, the aged binder has a higher |G*| than the virgin binder for the neat and rubber asphalt binders (Figure 4.1a). On the other hand, the aged binder has a lower δ than the virgin binders (Figure 4.1b). The higher |G*| and lower δ for the aged binders is a result of the hardening process by the RTFOT, which simulates short-term aging of the asphalt binders during mixing and placement.

The |G*| of the rubber asphalt is higher than the neat binder for all frequencies tested. However, the increase in dynamic shear modulus for the rubber asphalt, in comparison with the neat binder, is more prominent at lower frequencies. In addition, there is a decrease in the slope of the rubber asphalt master curve, or less frequency/temperature dependency, as a result of the rubber modification. Thus, the addition of the rubber modifier results in a reduction of dependency of |G*| on frequency. The same behavior is also observed in polymer modified asphalt binders (Lu *et al*., 1999).

In terms of phase angle, the addition of rubber caused a reduction of phase angle for all the frequencies in relation to the neat binder. The phase angle varied from approximately 59 to 68° for the rubber asphalt and from approximately 78 to 88° for the neat binder. This reduction in phase angle indicates the elasticity increase of the rubber asphalt binder with respect to the neat binder. Typically, modified asphalt binders show a higher |G*| and a lower δ, which means they are, in general, stiffer and more elastic than neat binders.

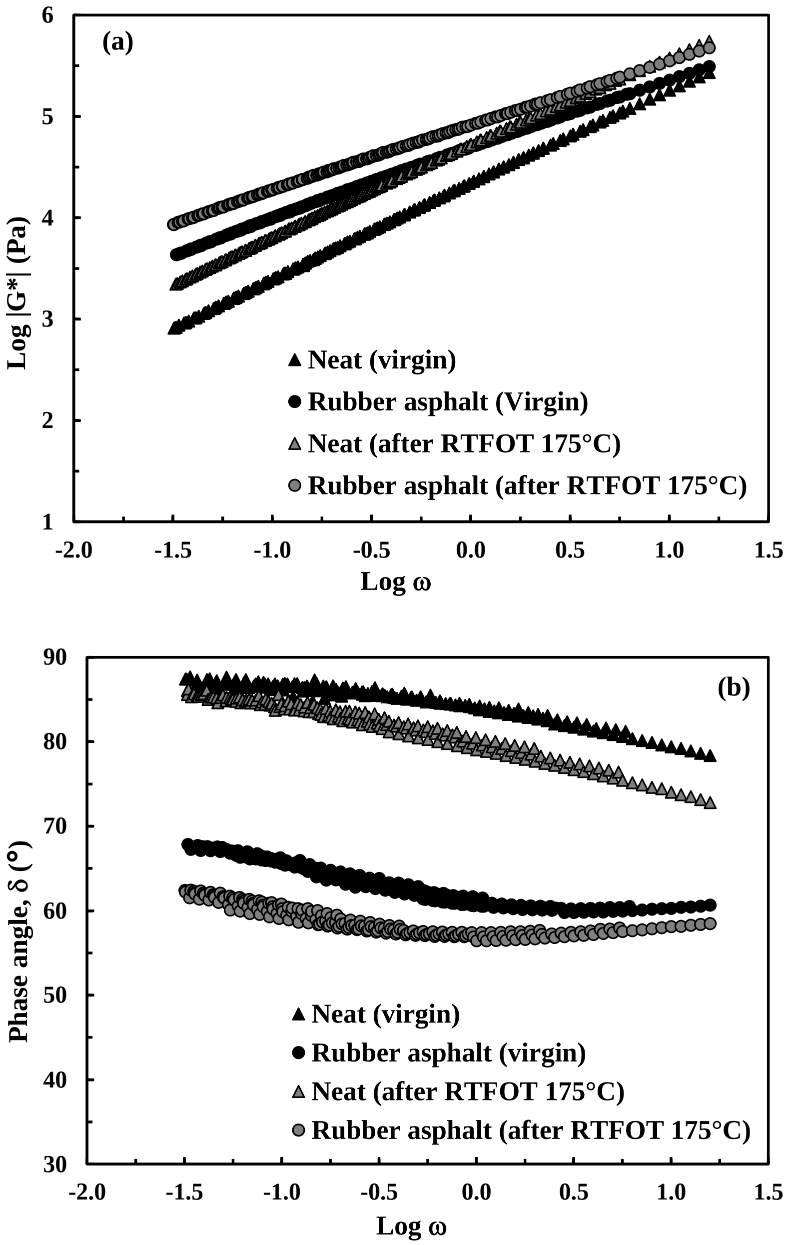



**Figure 4.1.** Master curves for (a) dynamic shear modulus (|G*|) and (b) phase angle (δ) at 46°C for the neat and rubber asphalt binders at virgin and after RTFOT conditions

The effects of RTFOT aging at 163°C and 175°C on the rubber asphalt binder are shown in Figure 4.2. The test was performed at 175°C to allow for proper aging, since the binder did not roll inside the glass bottle at 163°C, and to better simulate field conditions. Aging the binder at a higher temperature resulted in a decrease of the phase angle and an increase in the dynamic shear modulus at lower frequencies.

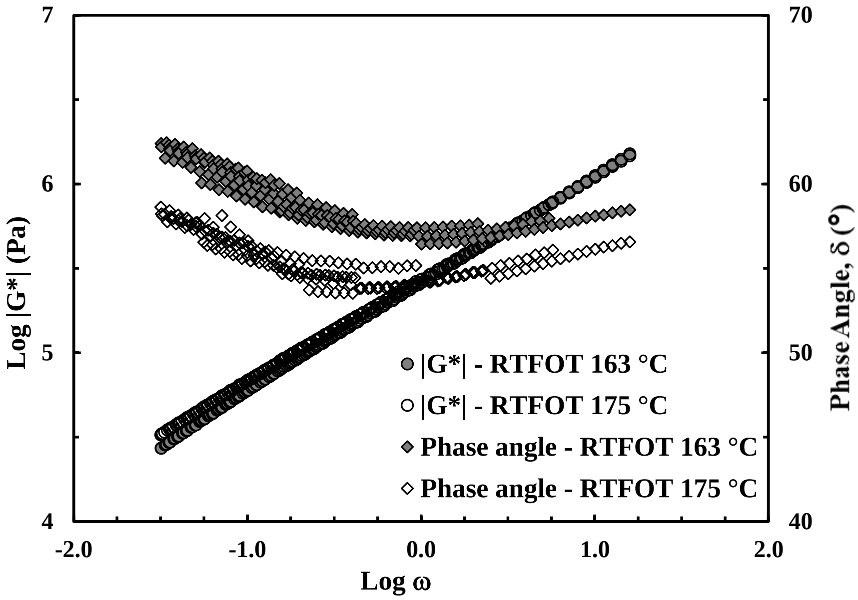
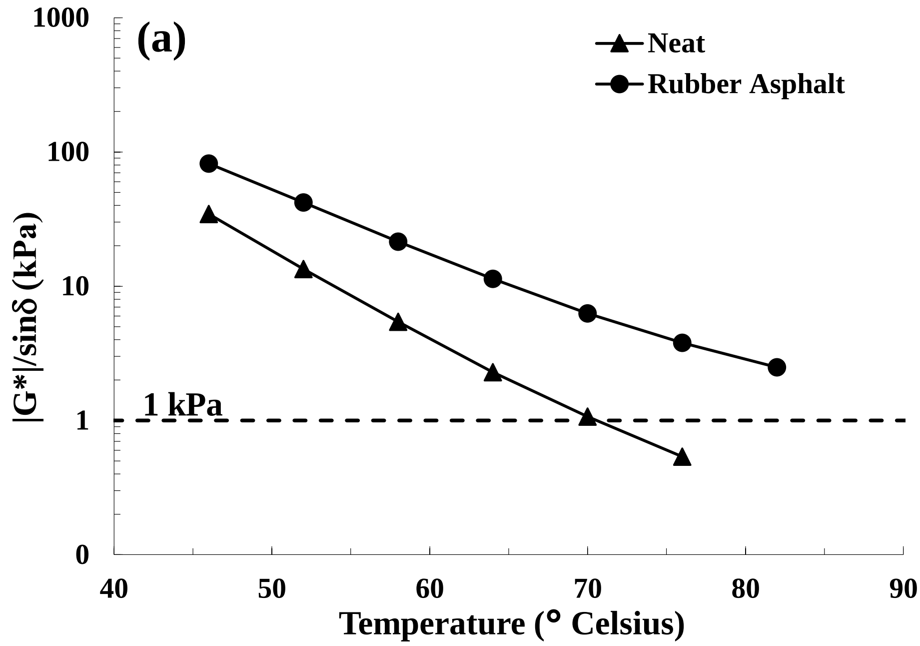



**Figure 4.2.** Dynamic shear modulus (|G*|) and phase angle (δ) master curves at 46°C for rubber asphalt after RTFOT aging at 163°C and 175°C

The change in G*/sinδ as a function of temperature are shown in Figure 4.3 (a) and (b) for the virgin and aged binder (163 and 175°C), respectively, for the neat and rubber modified binders. The rubber asphalt binder showed a higher |G*|/sinδ than the neat binder for the range of temperatures tested (46 to 82°C), which indicates that the rubber asphalt has a better potential to resist rutting than the neat binder. Moreover, the |G*|/sinδ for the binder aged at 175°C is higher than the one for 163°C, which shows that the binder aged in a temperature closer to the field mixing resulted in a stiffer binder and, consequently, more resistant to rutting.

A study by Giuliani & Merusi (2010) has also shown higher |G*|/sinδ for rubber asphalt in comparison with the neat binder. In addition, the rubber asphalt meets the |G*|/sinδ criteria (minimum 1.0 kPa for the virgin condition and 2.2 kPa for the RTFOT aged binder) for all the temperatures tested, whereas the neat binder meets the criteria up to 70°C. Both binders are suitable for the RJ-122 job, since a PG 64-10 binder was specified in the design (PG 58-10 with grade bumping due to slow traffic).



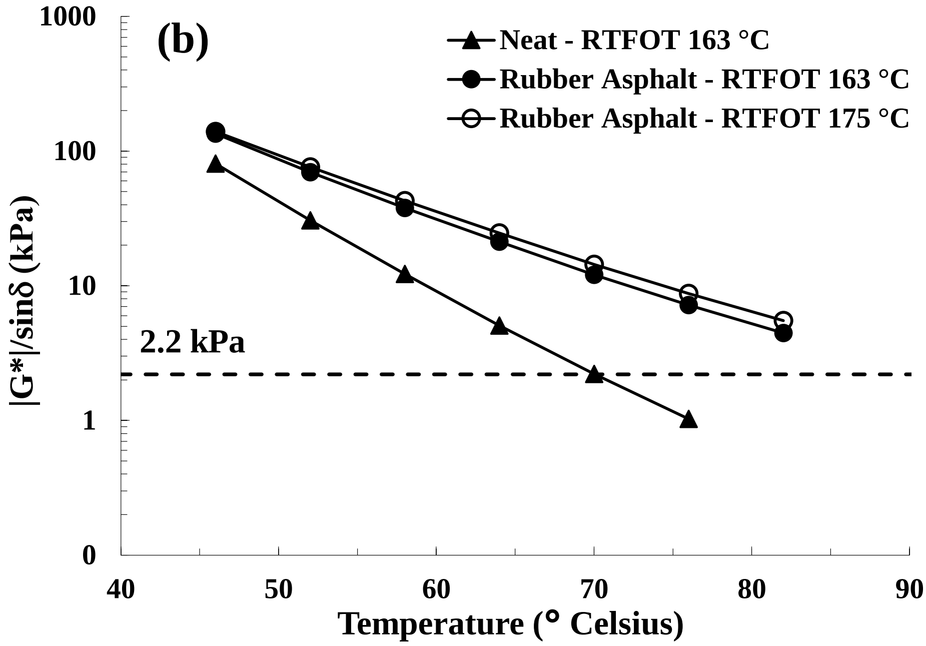



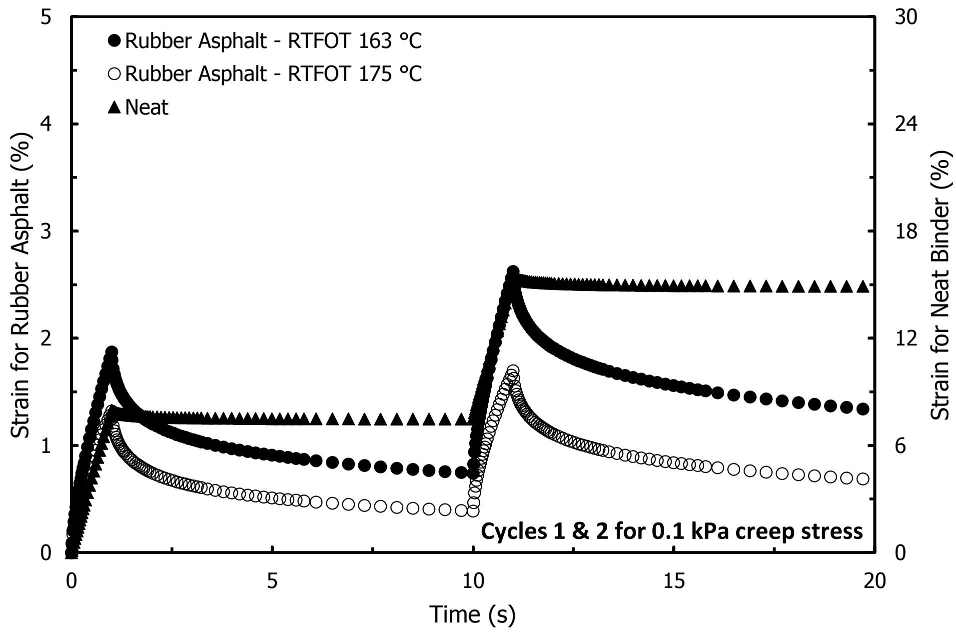

**Figure 4.3.** |G*|/sinδ as a function of temperature for the (a) virgin and (b) aged binders

Regarding the MSCR tests (at 58°C), the first two complete cycles (1 s of creep loading and 9 s at rest) of the MSCR tests are shown in Figure 4.4 (a) and (b) for the 0.1 and 3.2 kPa creep stresses, respectively. The scale for the y-axes in both figures was adjusted so that the stress-strain curves could be compared, since the strain in the neat binder is much larger than the one in the rubber asphalt (6 times for both stress levels).

It is important to note that the MSCR are always performed on binders after RTFOT aging at 163°C. However, an additional test at 175°C was performed for the rubber asphalt binder for proper aging. Thus, results of the MSCR tests for the rubber asphalt are presented for both situations (163°C and 175°C), whereas the neat binder was tested only under typical conditions (163°C).

The load-unload behavior in both binders are very distinct (Figure 4.4). The neat binder has a low elastic recovery, whereas the rubber asphalt shows a more pronounced elastic recovery. In addition, the neat binder accumulates much more strain than the rubber asphalt binder does. On the other hand, the rubber asphalt aged at 175°C exhibited an elastic recovery curve with a similar shape as the one aged at 163°C, but the rubber asphalt aged at 163°C accumulated more strain. The neat binder showed a percent recovery varying from 2.8 to 6.8% for the 0.1 kPa stress level (R100) and varying from 0.2 to 4.1% for the 3.2 kPa stress level (R3200), depending on the temperature (Figure 4.5). The rubber asphalt aged at 163°C, on the other hand, exhibited a R100 varying from 75.5 to 77.6% and a R3200 varying from 26.9 to 56.3%. Furthermore, aging at a higher temperature (175°C) resulted in an increase of percent recovery at all temperatures for both stress levels.

The increase in percent recovery after RTFOT aging is probably due to further interaction of the rubber particles with the asphalt binder. The rate and extent of swelling greatly depend on the temperature of the interaction process. When temperature is increased, the rate of swelling also increases. On the other hand, the extent of swelling decreases with the increase in temperature (Green & Tolonen, 1977). Lalwani *et al*. (1982) reported that the rubber-asphalt mix is not homogenous and has a higher viscosity at lower temperatures. However, when the temperature is increased beyond 200°C, depolymerisation occurs causing undesirable hardening of the binder. The depolymerisation process occurs slowly when the temperature is between 150°C and 200°C and proceeds rapidly when the temperature is greater than 200°C (Takallou, 1991). At higher temperatures, (225°C) the viscosity reaches a maximum value within 5 min and decreases rapidly thereafter (Sun & Li, 2010). According to Peralta *et al*. (2009), the rubber particles in asphalt rubber continue to swell during the aging process because they are not totally saturated after the rubber asphalt production and the. In addition, they state that elastic recovery is the binder property less affected in aging and typically increases after RTFOT.



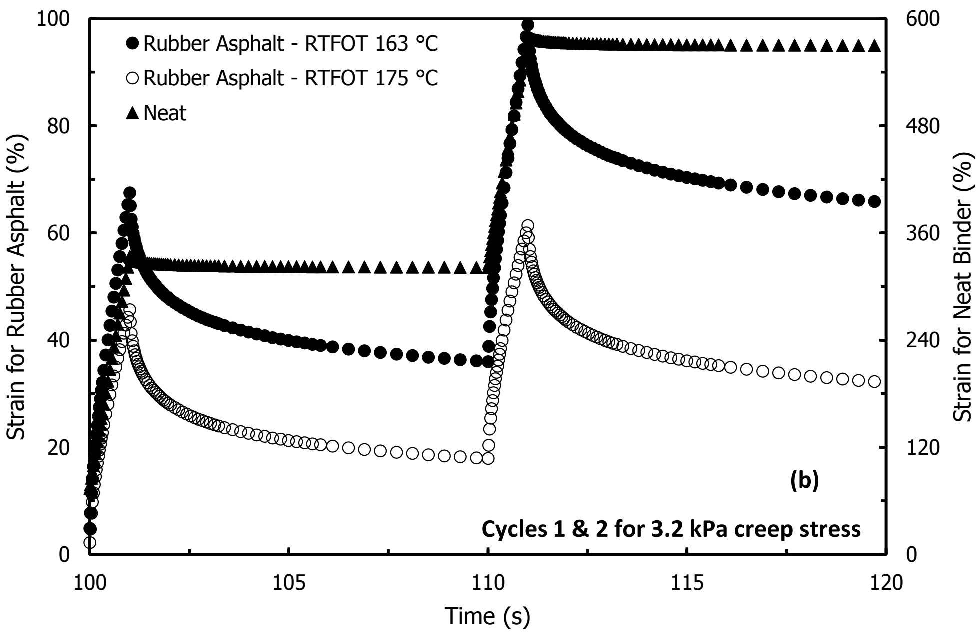




**Figure 4.4.** First 2 cycles of the MSCR test at (a) 0.1 kPa and (b) 3.2 kPa creep stresses

The variation of the percent recovery for the 3.2 kPa stress level with change in temperature is shown in Figure 4.5. The percent recovery for both binders decreased with increasing temperature, as expected. However, the percent recovery for rubber asphalt is higher than the recovery for the neat binder at any given temperature. The increased in percent recovery of the rubber asphalt binder validates the presence of the rubber modifier.

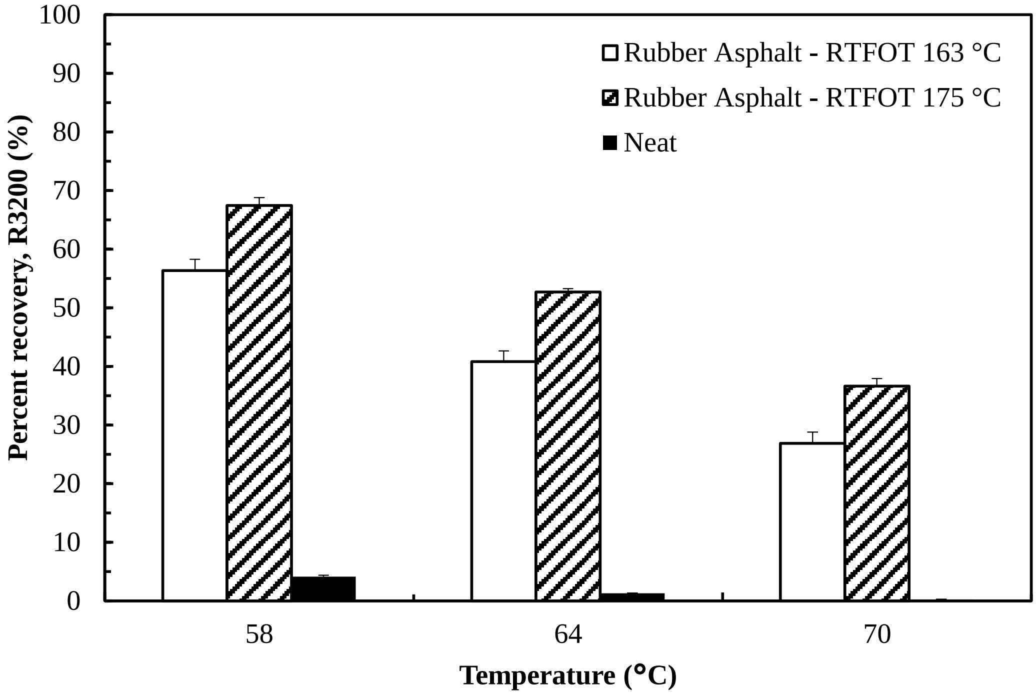



**Figure 4.5.** Change in R3200 with temperature for the neat and rubber asphalt binders

The neat binder (aged at 163°C) showed a non-recoverable creep compliance varying from 0.75 to 4.46 1/kPa for the 0.1 kPa stress level ($J_{nr,100}$) and from 0.78 to 4.97 1/kPa for the 3.2 kPa stress level ($J_{nr,3200}$). However, the rubber asphalt (aged at 163°C) exhibited a $J_{nr}$ 100 varying from 0.04 to 0.13 1/kPa and a $J_{nr,3200}$ varying from 0.08 to 0.53 1/kPa. Furthermore, aging the rubber asphalt at a temperature close to the field (175°C) resulted in a decrease of $J_{nr}$ of approximately 40% for all temperatures and stress levels when compared to the rubber asphalt aged at 163°C. The variation of non-recoverable creep compliance for the 3.2 kPa stress level with change in temperature is shown in Figure 4.6.

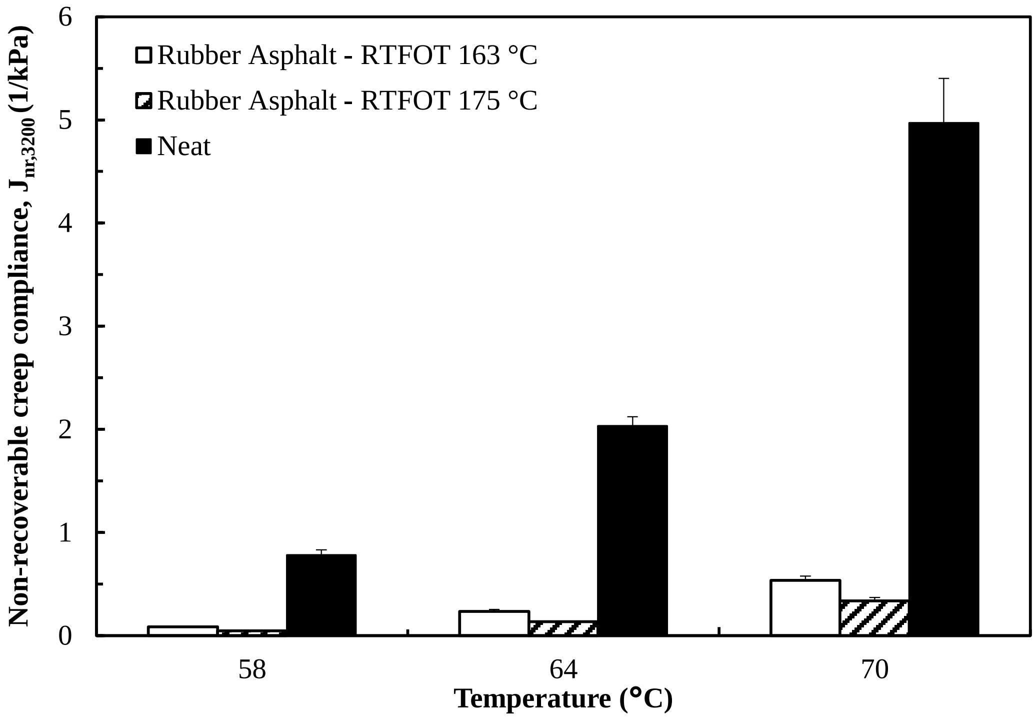



**Figure 4.6.** Change in $J_{nr,3200}$ with temperature for the neat and rubber asphalt binders

The non-recoverable creep compliances for both binders increase with increasing temperature. For any given temperature, the non-recoverable creep compliance for the rubber asphalt is lower than the one exhibited by the neat binder for both stress levels. This is an indicative that the rubber asphalt binder has a better resistance to rutting than the neat binder. These results are similar to the ones obtained for the $|G^*|/\sin\delta$ parameter. The decrease of non-recoverable creep compliance because of rubber modification has also been reported in the literature (Giuliani & Merusi, 2010; Subhy *et al*., 2015). In addition, aging the asphalt binder at a temperature close to the one expected in the field resulted in an even lower rutting potential of the rubber asphalt.

In terms of the MSCR stress sensitivity parameter ($J_{nr,diff}$), the rubber asphalt binder exhibited $J_{nr,diff}$ over the 75% limit for all temperatures tested (aged at 163°C and at 175°C), which indicates that the binder is stress dependent. Other studies have also observed the stress dependent behavior of rubber asphalt binders by means of the MSCR test (Martins *et al*., 2011; Santagata *et al*., 2015; Willis *et al*., 2012). Even though the binder showed a stress dependent behavior, the $J_{nr}$ values are very small (ranging from 0.08 to 0.53 1/kPa and from 0.05 to 0.34 1/kPa for the highest stress level for the RTFOT at 163°C and 175°C, respectively), indicating that the mixture should have a small rutting potential in the field.

In addition, it is important to note that the MSCR test is performed based on strains measured at the end of the recovery phase for each cycle. However, the rubber asphalt shows a delayed elastic response (Figure 4.4 (a) and (b)) in both stress levels, but more prominent in the lower stress level, since no horizontal plateau has been observed. This behavior shows that the delayed elastic deformation may have influenced the results, since the strain recorded at the end of each cycle was affected by the residual strain recorded at the end of the previous one. This behavior has also been reported elsewhere (Santagata *et al*., 2015).

### *4.4.2. Asphalt Mixture*

Permanent deformation of the field-blended rubber asphalt mixture under the LCPC wheel-track test are shown in Figure 4.7 for laboratory prepared and field cored specimens. The specimens fabricated in the laboratory using the rubber asphalt mixture collected after the RJ-122 rehabilitation job resulted in a permanent deformation of 4.2% after 30,000 cycles. The results are in accordance with French specifications and below the 5% limit for Brazilian conditions. This is an indicative that the rubber asphalt mixture (gap-graded gradation) has adequate permanent deformation resistance. These results are in agreement with the ones obtained for the binder permanent deformation.

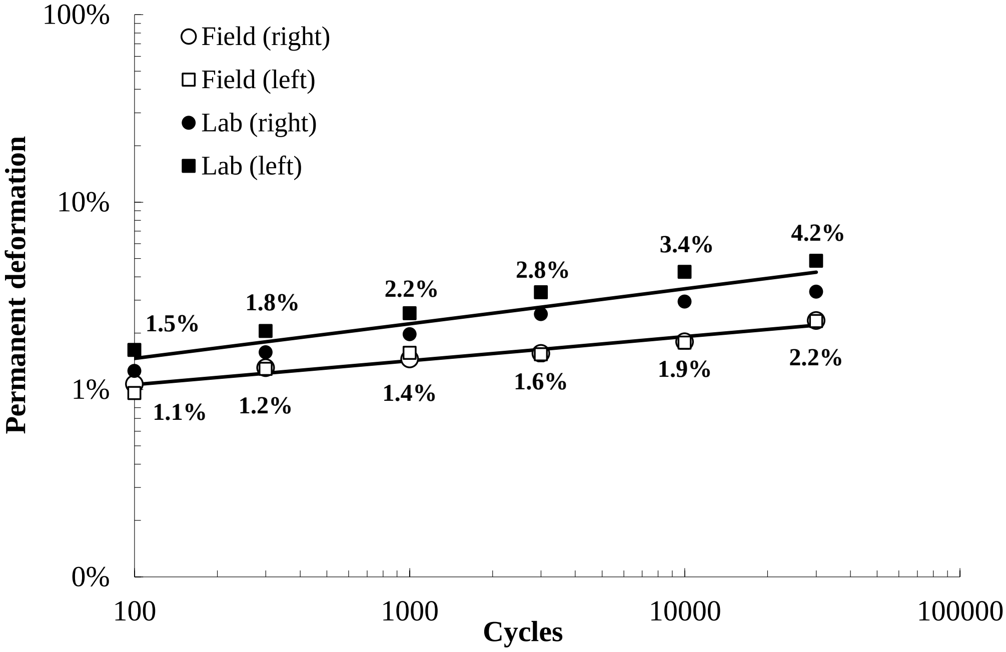



**Figure 4.7.** Permanent deformation of the field-blended rubber asphalt under the LCPC wheel-track test

In Brazil, other gap-graded mixtures using a terminal-blended rubber asphalt have been tested for permanent deformation using the LCPC wheel track test. Moura (2010) studied a gap-graded mixture with 8.2% of one terminal-blended rubber asphalt. The gap-graded mixture was in accordance with CALTRANS gradation specifications and met the Brazilian specifications for terminal-blended rubber asphalt. The permanent deformation in the LCPC wheel track test was 12.7% for laboratory specimens. Similarly, Negrão (2012) also performed LCPC wheel track tests on a similar gap-graded mixture, but with 6.0% of another terminal-blended rubber asphalt. The permanent deformation in the LCPC wheel track test was lower than the one obtained by Moura (2010), but was still above the 5% limit for adequate performance for heavy traffic in Brazil (5.5%).

The LCPC wheel-track test results for the RJ-122 field specimens also showed adequate permanent deformation resistance for the mixture (<5%). However, the permanent deformation of the field specimens after 30,000 cycles (2.2%) were approximately 48% lower than the one for the laboratory specimens. The lower permanent deformation obtained for the field specimens may be attributed to several factors, such as better compaction and homogeneity in the field or even possible differences in the constituents (aggregate, binder, etc.), since the mixture was collected after the rehabilitation job. Regardless, both results show adequate permanent deformation.

The field and laboratory specimens were used for binder extraction according to ASTM D2172-01 (ASTM, 2001) and further analysis. Enough binder was extracted from 15 kg of the field and laboratory mixtures in order to perform the rheological characterization. Even though the use of solvents for binder extraction may affect the binder rheological properties, both the field and laboratory extracted binders were subjected to the same chemical process. Binders were recovered in accordance with ASTM D1856-09 (ASTM, 2009). Thus, the binders should provide similar results. The master curves for the binders extracted from the field and laboratory mixtures are shown in Figure 4.8 for temperatures varying from 46 to 82°C. The masters curves were determined as the average of two samples for each binder, which yielded similar results.

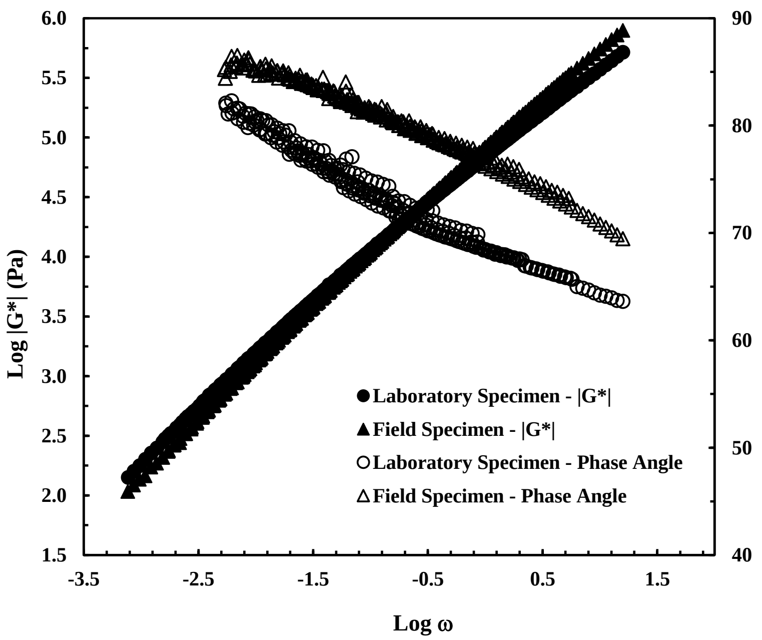



**Figure 4.8.** Dynamic shear modulus (|G*|) and phase angle (δ) master curves at 46°C for the extracted binders

The |G*| for the binder extracted from the laboratory mixture exhibited a smaller slope than the one for the binder extracted from the field mixture. Thus, the |G*| for the binder extracted from the laboratory mixture has a lower thermal susceptibility than the one extracted from the field mixture. In addition, the binder extracted from the laboratory mixture showed a more elastic behavior for practically all the frequencies. In general, the binder extracted from the laboratory mixture showed a better behavior: lower |G*| for the higher frequencies and higher |G*| for the lower frequencies. Furthermore, the binder from the field mixture consistently exhibited higher phase angles than the binder from the laboratory.

Thus, the rheological characterization performed on the extracted binders could not explain the lower permanent deformation exhibited by the field specimens, possibly because of the chemical process used to extract the binders. The lower permanent deformation is probably due to a better compaction, resulting in low air voids and better homogeneity in the field, resulting in a mixture with higher stiffness.

### *4.4.3. Field Performance*

The average rut-depths measured under the f-sAPT test section are summarized in Table 4.4 for the 7 years of traffic simulated. The average rut-depth observed in the f-sAPT test section was 4.2 mm for the 7 years of simulated traffic. Even though permanent deformation was observed, the limit of 7 mm used in toll roads in Brazil was not achieved during the 7-year simulation.

**Table 4.4 -** Average rut-depths under the f-sAPT test section

| Cycles | ESALS | Year | Average Maximum Rutting, mm |
|---|---|---|---|
| 0 | 0.00E+00 | 0 | 0.0 ± 0.0 |
| 14,573 | 2.40E+05 | 1 | 1.5 ± 2.2 |
| 35,621 | 5.86E+05 | 2 | 2.1 ± 2.5 |
| 60,577 | 9.97E+05 | 3 | 2.3 ± 2.6 |
| 81,392 | 1.34E+06 | 4 | 3.9 ± 1.5 |
| 105,185 | 1.73E+06 | 5 | 4.1 ± 1.3 |
| 118,166 | 1.94E+06 | 6 | 4.1 ± 1.3 |
| 150,000 | 2.47E+06 | 7 | 4.2 ± 1.2 |

The average maximum rut-depths after 3 and 4 years of service life for the 500-m test section in RJ-122 are shown in Figure 4.9 for each direction of traffic. The average maximum rut-depth are higher for the northbound direction in both monitoring surveys, due to trucks carrying more loads in that direction. The average rut-depth after 3 years of service life was 3.0 mm and 2.2 for the northbound and southbound directions, respectively.

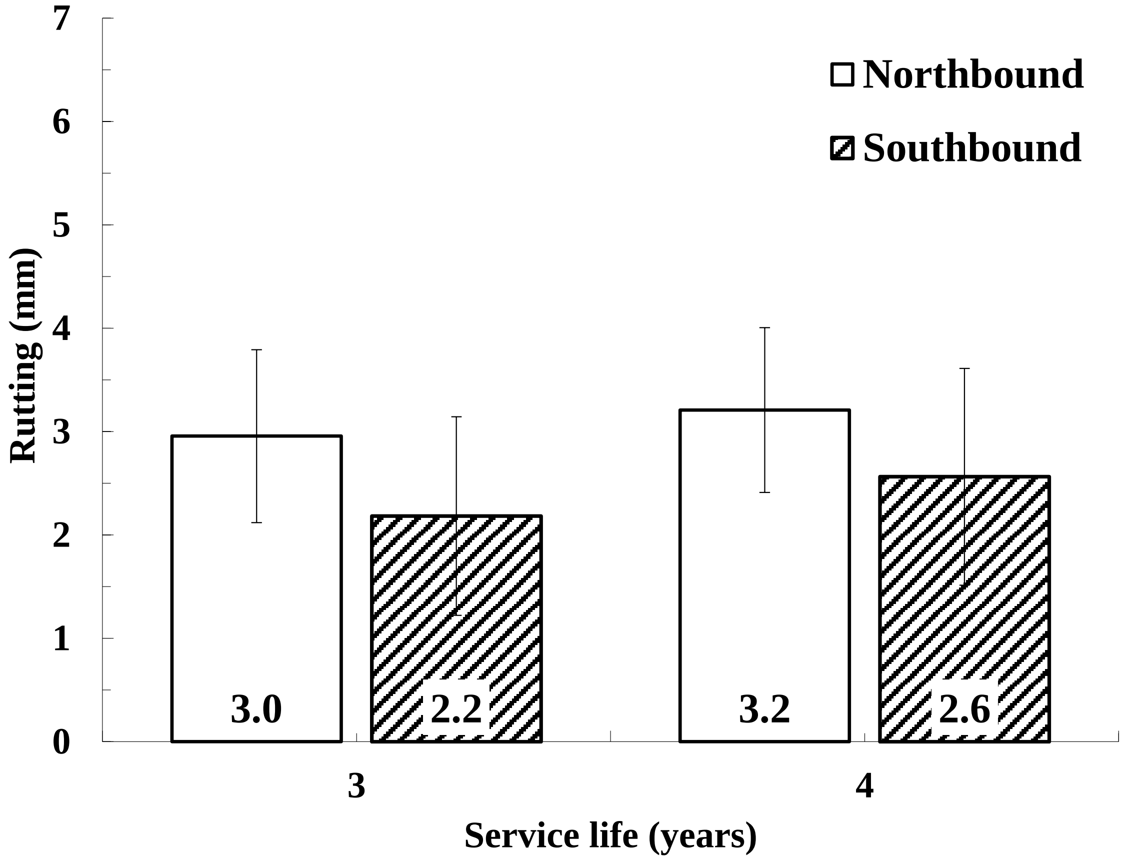



**Figure 4.9.** Maximum rut-depth after 3 and 4 years of service life for each direction of traffic in RJ-122

An 8.5% increase (3.2 mm) was observed for the northbound direction after another year of service life (4 years total), whereas the increase was 17.4% (2.6 mm) for the southbound direction. The observed rut-depths after 4 years of service life are well below the limit used for toll roads in Brazil (7 mm). These results are in agreement with previous binder and mixture testing, which showed a good resistance to permanent deformation at both levels. The maximum rut-depth observed in the f-sAPT simulation are shown in Figure 4.10 along with the ones observed in the pavement monitoring surveys for the northbound direction (highest rut-depths).

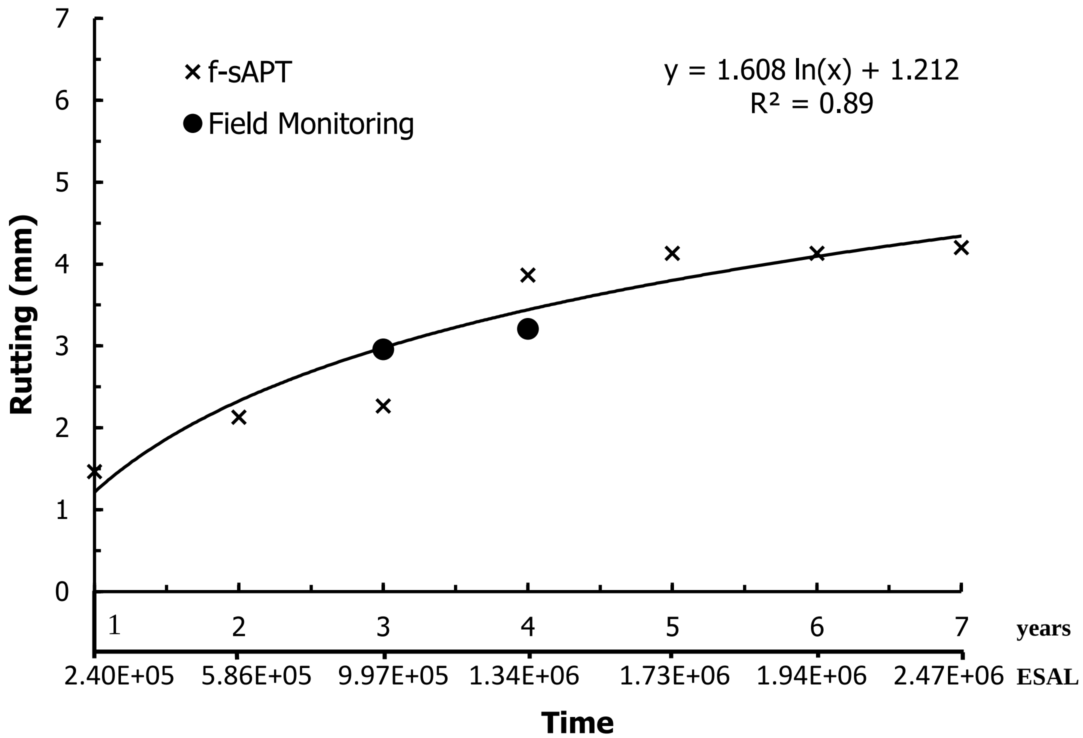



**Figure 4.10.** Average rut-depths measured under the f-sAPT test section

The rut-depths obtained during the field monitoring surveys after 3 and 4 years of service life are in the same range as the ones obtained for f-sAPT. It is important to note that field and f-sAPT data were obtained using different methodologies (LCMS vs visual inspection with a rut-measurement device), which may lead to differences in readings. In addition, the rut-depths are very small and any variations, including equipment or human error, may lead to the observed differences. Even though there is a slight variation between the actual rut-depths obtained in the field monitoring surveys in relation to the ones for f-sAPT, the field rut-depths for the 3$^{rd}$ and 4$^{th}$ years are very similar to the ones for the best fit line using the f-sAPT data. The results obtained for the pavement monitoring and f-sAPT are an indicative that traffic was successfully simulated. In addition, the permanent deformation observed in the field corroborated the results obtained both at the binder and at mixture level performed in the laboratory.

### 4.5. Conclusions

This study was conducted in order to evaluate the permanent deformation of a field-blended rubber asphalt recently used in a gap-graded mixture in Brazil for the first time. The permanent deformation of the field-blended rubber asphalt binder was first evaluated and compared to the neat binder. Then, the asphalt mixture was assessed in the laboratory and in the field, in order to characterize the permanent deformation at different levels. The following conclusions are warranted in this study:

- The |G*| of the rubber asphalt is higher than the neat binder for all frequencies tested and the increase in |G*| for the rubber asphalt is more prominent at lower frequencies for the range of temperatures evaluated (46 to 82°C).
- The addition of crumb rubber modifier resulted in a reduction of phase angle, for all the frequencies, in relation to the neat binder.
- The rubber asphalt binder did not roll inside the glass during the RTFOT test at 163°C and additional tests were carried out at a higher temperature (175°C) that is also close to the temperature expected during field mixing.
- The rubber asphalt binder showed a higher |G*|/sinδ than the neat binder for all temperatures tested both before and after aging, which indicates that the rubber asphalt has a better potential to resist rutting than the neat binder.
- The percent recovery for rubber asphalt determined in the MSCR test is higher than the recovery for the neat binder at any given temperature. The increased in percent recovery of the rubber asphalt binder validates the presence of the rubber modifier.
- For any given temperature, the non-recoverable creep compliance ($J_{nr}$) for the rubber asphalt is lower than the one exhibited by the neat binder for both stress levels. This is an indicative that the rubber asphalt binder has a better resistance to rutting than the neat binder.
- The stress sensitivity parameter ($J_{nr,diff}$) for the rubber asphalt binder exhibited was over the 75% limit for all temperatures tested, which indicates that the binder is stress dependent. Even though the binder showed a stress dependent behavior, the $J_{nr}$ values are very small (ranging from 0.08 to 0.53 1/kPa for the higher stress level), indicating that the mixture should have a small rutting potential in the field.
- The |G*|/sinδ parameter alone would have provided enough information to show that the rubber asphalt has a better rutting resistance than the neat binder.

- The permanent deformation after 30,000 cycles of the LCPC wheel-track test for the rubber asphalt mixture collected after the RJ-122 rehabilitation job was 4.2%. The results are in accordance with French specifications and below the 5% limit for Brazilian conditions. The wheel track test results are an indicative that the rubber asphalt mixture in a gap-graded gradation has adequate permanent deformation resistance, which confirms the results obtained in the MSCR tests.
- The LCPC wheel-track test results for the RJ-122 field specimens also showed adequate permanent deformation resistance for the mixture (<5%). However, the permanent deformation of the field specimens after 30,000 cycles (2.2%) were approximately 48% lower than the one for the laboratory specimens.
- The observed rut-depths of the test section after 3 and 4 years of service life (3.0 and 3.2 mm, respectively) are well below the limit used for toll roads in Brazil (7 mm). The adequate resistance to rutting shown by the rubber asphalt mixture confirmed the results obtained in the MSCR and the wheel track test.
- The average rut-depth observed in the f-sAPT test section was 4.2 mm for the 7 years of simulated traffic. The results obtained for the pavement monitoring and f-sAPT were similar, which is an indicative that traffic was successfully simulated.
- The permanent deformation observed in the field confirm the results obtained both at the binder and at mixture level performed in the laboratory, even though the asphalt binder showed a stress dependent behavior in the MSCR test.

# 5. FIELD AND LABORATORY FATIGUE BEHAVIOR OF A RUBBER ASPHALT MIXTURE

## 5.1. Introduction

Current practice in pavement design in Brazil is shifting from more traditional, empirical pavement design methods to mechanistic–empirical ones. In mechanistic-empirical methods, a series of failure criteria must be established in order to account for each type of distress. As a result, the ability to assess the fundamental characteristics of pavement materials is becoming increasingly more important.

In terms of flexible pavement design, fatigue cracking and rutting are among the major types of distress to be considered (Huang, 1993). According to ASTM E1823-13 (ASTM, 2013), fatigue is defined as "the process of progressive localized permanent structural change occurring in material subjected to conditions which produce fluctuating stresses and strains at some point or points and which may culminate in crack or complete fracture after a sufficient number of fluctuations." Among the factors affecting fatigue resistance of asphalt mixtures, the following play a major role: asphalt characteristics, asphalt content, aggregate type and gradation, air void content, and temperature (Epps & Monismith, 1972).

In terms of fatigue cracking in flexible pavements, the focus is on the horizontal tensile strain that occurs at the bottom of the asphalt layer. The tensile strain is typically associated with the maximum allowable number of load repetitions through laboratory fatigue tests and shift factors or transfer functions are used to account for differences in geometric and loading conditions for the actual pavements in relation to laboratory tests. The ability to predict the fatigue behavior of the asphalt mixture in the field based on the fatigue properties determined in the laboratory is a key element for pavement performance.

In terms of fatigue behavior, there is a growing interest in improving binder and, consequently, mixture performance in order to improve the fatigue resistance in Brazilian pavements. The use of rubber asphalt mixtures is an alternative to conventional asphalt mixtures due to improved fatigue resistance and lower pavement maintenance costs (CALTRANS, 2006). It is well known that rubber modification results in gains on the binder rheological properties (Bahia & Davies 1994; Moreno-Navarro *et al*., 2015), as well as an improved resistance to aging (Sainton, 1990), resulting in an asphalt mixture of high performance and a longer pavement service life. In particular, wet process-high viscosity binders, with up to 20% of crumb rubber modifier (CRM) contents may be used in a gap-graded gradation for improved performance (Zeiada *et al*., 2014).

Among the fabrication processes available for rubber asphalt production, terminal blend is the most commonly used in Brazil. Alternatively, rubber asphalt may be produced by a field blending process, which results in a highly viscous material with enhanced engineering properties, but requires specific equipment that is typically installed at the job site or close to the supplying asphalt plant. However, the lack of information regarding this technique and the performance of the resulting field-blended mixtures in Brazil inhibits a more extensive use in pavement construction/rehabilitation.

Thus, this study aims to assess the fatigue behavior of a field-blended rubber asphalt, recently used in a gap-graded mixture in Brazil for the first time. The fatigue behavior of the field-blended rubber asphalt binder was first evaluated and compared to the neat binder (AC 30/45 penetration grade). Then, the asphalt mixture was tested in the laboratory and in the field, in order to characterize the fatigue behavior at different levels (binder, mixture, and field).

The binders (neat and rubber asphalt) were evaluated using the Time Sweep and the Linear Amplitude Sweep (LAS) tests in order to characterize fatigue behavior. The fatigue of the rubber asphalt mixture was determined by performing four-point bending tests on specimens prepared in the laboratory and on field-collected specimens. Field performance of the rubber asphalt mixture was investigated by means of periodic monitoring of a test section and by employing a heavy vehicle traffic simulator for predicting future performance.

## 5.2. Materials

A neat asphalt binder of penetration grade 30/45 (AC 30/45), typically used in Brazil, was modified using 19% of crumb rubber modifier (ambient grinding) meeting the Arizona Department of Transportation Type B crumb (ADOT, 2000) and with 100% passing the #16 sieve (1.18 mm). The rubber asphalt binder was obtained directly at the job plant using a combination blender from D&H Equipment after the rehabilitation job of highway RJ-122 in Brazil. A 60-minute reaction time was used for modifying the binder with agitation inside the tank to ensure the material is reacting uniformly and produces a consistent blend. The physical properties of the neat and rubber asphalt binder are summarized in Table 5.1. The binder is classified as a PG 88-22 and was well above the specifications of a PG 58-10 for the RJ-122 project (PG 64-10 with grade bumping) (Balaguer, 2012).

**Table 5.1 -** Physical properties of the neat and CRM binders

| Property | Units | Neat | CRM |
|---|---|---|---|
| Penetration | 0.1 mm | 38 | 32 |
| Softening Point | ºC | 53 | 70 |
| Elastic recovery | % | - | 86.2 |
| Brookfield Viscosity* | | | |
| - @ 135°C | cP | 435 (20 rpm) | 6,637 (20 rpm) |
| - @ 150°C | cP | 215 (50 rpm) | 3,887 (20 rpm) |
| - @ 177°C | cP | 77 (100 rpm) | 1,975 (20 rpm) |

* Spindle 21 was used for the neat binder and 27 for the rubber asphalt

The mix gradation, with granite aggregate, is a gap-graded similar to that specified by the Arizona Department of Transportation. The mean grain size distribution curve is shown in Table 5.2 along with ADOT specifications. The gap-graded mixture was collected in the field straight from the combination blender after the rehabilitation job in RJ-122 and brought to the laboratory for testing. Field slabs from a section of highway RJ-122 that was not subjected to traffic were also collected for laboratory testing.

**Table 5.2 -** Grain size distribution curve for the RJ-122 test section

| Sieve | Diameter (mm) | Mean Curve (%passing) | ADOT Specifications | |
|---|---|---|---|---|
| | | | Lower limit | Upper limit |
| 3/4" | 19.000 | 100 | 100 | 100 |
| 1/2" | 12.500 | 87.4 | 80 | 100 |
| 3/8" | 9.520 | 69.8 | 65 | 80 |
| Nº 4 | 4.800 | 22.7 | 28 | 42 |
| Nº 8 | 2.360 | 13.0 | 14 | 22 |
| Nº 200 | 0.075 | 2.1 | 0 | 2.5 |

The volumetric properties of Marshall-compacted samples and the optimum binder content for the field-blended rubber asphalt mixture are presented in Table 5.3 for a 75-blow energy. The mixture has an asphalt content of 8.0 % and air voids of 5.6%. Because there isn´t a Brazilian specification for this mixture, the results are compared with the Brazilian specification DNIT ES 112/2009 for a rubber asphalt terminal blend or “wet process” (DNIT, 2011a).

**Table 5.3** – Volumetric properties of the field-blended rubber asphalt mixture

| Property | Mixture | Limits (DNIT 112/2009) |
|---|---|---|
| Air Voids (%) | 5.6 | 4-6 |
| Optimum asphalt content (%) | 8.0 | 5-8 |
| Voids in Mineral Aggregate (%) | 23.1 | >14 |
| Stability (kN) | 7.28 | >7.0 |

## 5.3. Methods

A series of laboratory and field tests were conducted in order to assess the fatigue behavior of the field-blended rubber asphalt used in the gap-graded mixture. The objective was to verify the fatigue of the field-blended rubber asphalt binder in relation to the neat binder and then investigate the mixture performance in the laboratory and in the field, in order to characterize the fatigue behavior at different levels. Therefore, laboratory and field tests were used as described next.

### *5.3.1. Asphalt Binder*

The current Superior Performing Asphalt Pavement System (Superpave) mixture design process, developed under the Strategic Highway Research Program (SHRP) as an initiative to improve mixture design, uses the performance grade (PG) system to select asphalt binders based on the expected pavement temperatures. The approach originally proposed in the grading system to control binder fatigue is based on the viscoelastic parameter $|G^*|\cdot\sin\delta$ (Bahia *et al*., 2001).

However, the NCHRP Project 9-10, which was focused on validating the use of the $|G^*|\cdot\sin\delta$ parameter for characterizing the fatigue behavior of modified binders, found a poor correlation between $|G^*|\cdot\sin\delta$ and the actual pavement resistance to fatigue (Bahia *et al*., 2001). In addition, later research showed that this parameter did not relate well with the accumulation of fatigue damage of mixtures measured through a beam fatigue test under strain-controlled conditions (Shenoy, 2002). The main reason the $|G^*|\cdot\sin\delta$ parameter does not correlate well with fatigue behavior of mixtures is that the test is conducted in the linear viscoelastic range (low deformation levels), whereas fatigue typically occurs in the non-linear range (high deformations levels) (Bahia *et al*., 1999).

Currently, there are two test procedures commonly used for fatigue testing of asphalt binders: the time sweep test and the Linear Amplitude Sweep (LAS) test, both conducted using a Dynamic Shear Rheometer (DSR). The time sweep test was developed during the NCHRP 9-10 project in an attempt to better characterize the fatigue behavior of binders (Bahia *et al*., 2001) and account for non-linear viscoelastic behavior. The applied load amplitude is user-defined, which allows for considering the pavement structure and the expected strain levels in the field. The time sweep test has shown good correlation with the fatigue life of asphalt mixtures when proper conditions are applied (Anderson *et al*., 2001, Martono *et al*., 2007). However, the test may take several days in order to achieve failure.

Later work by Martono & Bahia (2008) suggested that an amplitude sweep procedure was a promising indicative of fatigue performance of asphalt binders and an accelerated procedure for fatigue characterization of asphalt binders was developed (Johnson, 2010). The LAS procedure was later modified in order to include very small increments in loading amplitude every cycle (linearly increasing loads) for smoother crack growth rate. The fatigue behavior was analyzed using the continuum damage approach (Hintz, 2012). The test is divided into two stages: the sample is first tested in shear using a frequency sweep to determine the binder rheological properties using a Dynamic Shear Rheometer (DSR). The sample is then tested using a series of oscillatory load cycles at linearly increasing strain amplitudes at a constant frequency to cause accelerated fatigue damage. The fatigue law of the following form (Eq. 5.1) is derived from the LAS test.

$$N_f = A\,(\gamma_{max})^{-B} \qquad \text{(Eq. 5.1)}$$

where $N_f$ is the number of cycles to failure, $\gamma$ is the applied shear strain and A, B are constants (material dependent).

In this study, binder fatigue resistance was assessed by determining the time sweep and the LAS tests using the DSR from TA Instruments Discovery Model HR-3. The binders were first characterized in terms of their linear viscoelastic behavior and later subjected to the fatigue tests. Typically, a 25-mm diameter parallel plate geometry with a 1-mm gap setting is used for rheological binder testing on virgin and RTFOT aged samples. However, because of the influence of the crumb rubber particles in the binder, the rubber asphalt binder was tested using a 25-mm diameter parallel plate geometry with a 2-mm gap setting. A higher gap setting (2 mm) for rubber-modified binders has also been used in another study for the same reason (Subhy *et al*., 2015). The neat binder was tested using the typical 1-mm gap setting.

Frequency and temperature sweep tests were conducted on the neat and rubber asphalt binders using the DSR at a 0.1% strain using the geometries aforementioned. Frequency ranged from 0.159 to 15.9 Hz (1.0 to 100.0 rad/s), and temperature ranged from 46°C to 82°C, with an interval of 6°C. Testing at lower temperatures was not possible because the cooling system was not functioning properly. Master curves were developed for the dynamic shear modulus (|G*|) and phase angle ($\delta$) for a 46°C reference temperature. Binders were tested under virgin and rolling thin film oven test (RTFOT) aged conditions. The RTFOT was performed according to ASTM D2872-12 at a temperature of 163°C (ASTM, 2012). However, the rubber asphalt binder did not roll inside the glass bottles during the test at this temperature. The same was also reported in NCHRP Project 9-10 for modified binders because of their high viscosity (Bahia *et al*., 2001). Thus, the RTFOT was also carried out at 175°C in order to allow for binder rolling and to create the thin film for aging. At this temperature, which is closer to the temperature used for mixing the rubber asphalt in the field, the film was properly achieved in the glass bottle.

The fatigue resistance of the binders was conducted using an 8-mm diameter parallel plate geometry with a 2-mm gap setting for all binders. The LAS test was conducted in accordance with AASHTO TP101-14 (AASHTO, 2014c). Both the time sweep and the linear amplitude sweep tests were conducted at a temperature of 20°C and a loading frequency of 10 Hz. This configuration was used in order to be consistent with fatigue testing of the gap-graded mixture, as well as reduce the duration of the time sweep tests. Strain sweep tests were previously performed in accordance with AASHTO T315-12 (AASHTO, 2012) in order to determine the linear viscoelastic region for each binder at 20°C. The neat binder exhibited a linear viscoelastic behavior up to 2.2% strain, whereas the linear viscoelastic limit for rubber asphalt was 2.7%. The strain levels for the time sweep tests were determined such that the lowest strain level for both binders was still in the linear viscoelastic region. Thus, the binders were tested at 2%, 3%, and 4% strain. At least 2 samples were tested at each strain level. Even though there are several fatigue failure criteria available, the present study used the 50% reduction in initial |G*| approach to be consistent with the mixture fatigue tests described in the following item.

### *5.3.2. Asphalt Mixture*

Even though the asphalt binder plays a major role in the fatigue resistance of asphalt mixtures, it is still necessary to verify the fatigue resistance of the mixtures. This occurs because other parameters, such as aggregate gradation, for example, have an influence on the fatigue behavior of asphalt mixtures. There are, currently, several tests available for determining the fatigue behavior of asphalt mixtures, such as flexural fatigue tests (three-point or four-point), tensile fatigue test (diametral or uniaxial tension compression), fracture mechanics approach, and tensile strength and stiffness. Tayebali *et al*. (1994) conducted an evaluation of the different methods for defining the fatigue response of asphalt agreggate mixes and, among the tests available, the repeated flexural bending tests in constant strain mode at 10 Hz, along with direct tension tests, were the most recommended.

In laboratory, fatigue testing may be carried out either in strain-controlled or stress-controlled modes (Monismith *et al*., 1971). In the constant stress mode, the stress is kept constant and the strain increases with the number of load repetitions. In the constant strain mode, the strain is kept constant, and the load or stress is decreased with the number of load repetitions. According to Huang (1993), the constant stress test is more appropriate for thicker pavements (HMA larger than 152 mm thick), where the HMA is the main load-carrying element and the strain increases with the number of repetitions as the HMA weakens. The constant strain test, on the other hand, is more appropriate for thin pavements (HMA less than 51 mm thick) because the strain in the asphalt layer is governed by the underlying layers and is not affected by the decrease in stiffness of the HMA (Huang, 1993). For intermediate thicknesses (from 51 to 152 mm), a combination of constant stress and constant strain occurs. However, the researchers in the SHRP A003-A project recommended the use of constant strain tests for all pavement loading conditions. If fatigue evaluations are made using the expected strains at the bottom of a given pavement structure, for example, then constant stress and constant strain tests result in similar rankings (Tayebali *et al*., 1993; Tayebali *et al*., 1994; Harvey & Tsai, 1996).

Typically, the fatigue behavior of a given asphalt mixture is determined by conducting tests at different stress/strain levels and relating it to the number of cycles to failure. The fatigue laws for asphalt mixtures are normally based on the relationship proposed by Monismith *et al*. (1971) and shown in Equation 5.2.

$$N = k_1 \left(\frac{1}{\varepsilon_t \; or \; \sigma_t}\right)^{k_2} \qquad \text{(Eq. 5.2)}$$

Where N is the number of load repetitions to failure, $\varepsilon_t$ is the applied tensile strain, $\sigma_t$ is the applied stress, and $k_1$, $k_2$ are coefficients determined experimentally. In the constant strain test, where specimens are not tested until rupture, there are several ways to define failure and to determine the fatigue life of asphalt mixtures. The conventional approach consists of performing the test until the initial sample stiffness (typically taken at the 50th cycle) is reduced by either 40 or 50%. Alternatively, other researchers have used methods such as the energy ratio, ratio of dissipated energy change, and the viscoelastic continuum damage approach (VECD), among others, for determining the fatigue behavior of asphalt mixtures (Mello *et al*., 2010; Shen *et al*., 2006). These methods provide a means for determining the fatigue behavior of asphalt mixtures that is independent of mode of loading.

As mentioned earlier, the fatigue failure analysis for flexible pavements relies on fatigue laws. Even though the dissipated energy concept relates well to damage propagation, a fatigue law cannot be determined for the mixtures. In the case of VECD, advanced numerical analyses may been used for modelling the fatigue behavior of asphalt mixtures in flexible pavements based on the characteristic curves of the materials and the continuum damage principles (Mello, 2008; Nascimento, 2015). However, current practice in pavement design in Brazil, as well as specifications from local and federal agencies, are based on fatigue laws based on strain-controlled testing.

The fatigue behavior of the field-blended rubber asphalt mixture was determined by conducting repeated flexural bending tests (four point bending) in accordance with ASTM D7460-10 (ASTM, 2010). The tests were performed on 380 mm long by 50 mm thick by 63 mm wide specimens sawed from laboratory and field-compacted specimens using the IPC Global Pneumatic 4 Point Bending apparatus. Tests were conducted in constant-strain mode using a frequency of 10 Hz and a temperature of 20°C. The strain was varied between 200 and 800 $\mu\varepsilon$ at six different strain levels in order to develop a complete fatigue curve. The failure criterion of 50% reduction of the initial flexural stiffness, taken at the 50th cycle, was used for developing the fatigue law of laboratory and field specimens.

### *5.3.3. Field Performance*

Extensive binder and mixture testing are typically performed in the laboratory in an attempt to predict performance under conditions as close as possible to the ones expected in the field in order to predict pavement performance. Even though laboratory testing provides us with good estimates of field behavior, several factors may affect in situ performance, such as climatic and loading conditions.

Long-term pavement performance is one way to monitor field behavior and assess performance under local conditions. Even though monitoring field performance eliminates possible discrepancies between lab and field, many years of data collection are necessary in order to gather enough information to draw any conclusions. One way to provide this link between laboratory evaluation of materials used in pavement layers and the field behavior of these materials in pavement structures is via full-scale accelerated pavement tests (f-sAPT) (Steyn, 2012).

According to Metcalf (1996), accelerated pavement tests can be defined as “the controlled application of a prototype wheel loading, at or above the appropriate legal load limit to a prototype or actual, layered, structural pavement system to determine pavement response and performance under a controlled, accelerated, accumulation of damage in a compressed time period.” Thus, heavy vehicle simulators can be used for evaluating new road concepts and maintenance strategies and provide the option of performing a life cycle cost analysis, which, in turn, can decrease both costs and environmental impacts (Saevarsdottir *et al*., 2014). Among other applications, f-sAPT can be used to assess fatigue of in-service pavements. A study using f-sAPT has been conducted in order to understand the fatigue behavior of paving materials, such as rubber asphalt and conventional asphalt concrete overlays (Harvey *et al*., 2001).

Fatigue of the field-blended rubber asphalt mixture was determined though periodical pavement monitoring along with accelerated pavement tests using a heavy vehicle simulator. The section selected for further evaluation and monitoring of highway RJ-122 was between the 34.5 and 35.0 kilometer marks. A Pavement Scanner was used to determine the cracked area after 3 and 4 years of service life. The Pavement Scanner is a vehicle mounted with a Laser Crack Measurement System (LCMS) that allows the automatic detection of cracks and the evaluation of rutting, macrotexture, and other road surface features. Detailed information about the Pavement Scanner can be found elsewhere (Fialho, 2015).

A representative test section was chosen for f-sAPT using a full scale, mobile traffic simulator (FSMTS) from *Simular Tecnologia do Pavimento Ltda*. The FSMTS was positioned at the northbound 35.035 kilometer mark. The FSMTS has a total length of 25 meters and width of 3 meters, weighing approximately 50,000 kg. The equipment consists of a dual-wheel half-axle that can apply loads up to 100 kN, which corresponds to an equivalent dual-wheel single axle load of 200 kN. The half-axle moves along an axis either directional (8,000 cycles/day) or bidirectional (16,000 cycles/day) with longitudinal displacements of up to 8.6 meters and transversal displacements of up to 1 meter. In addition, the FSMTS is equipped with sensors to automatically record air humidity, ambient temperature (inside the FSMTS and outside), pavement temperature, load applied, and number of passages.

Tests were conducted using a bidirectional 78.1 kN load, which is approximately 45% higher than the allowable load of 53.8 kN for this type of axle according to the Brazilian legislation at the time. For determining the cracked area under the FSMTS, visual inspections were performed periodically in order to detect any cracking for the 7-meter long test section.

## 5.4. Results and Discussion

### *5.4.1. Asphalt Binder*

The master curves for the dynamic shear modulus (|G*|) and phase angle (δ) are shown in Figure 5.1 for the neat and rubber asphalt binders at 46°C reference temperature. Data obtained for the higher temperatures were ignored because there was a lot of variation, possibly because the binders became more fluid at these temperatures. As expected, the |G*| of the rubber asphalt is higher than the neat binder for all frequencies tested. However, the increase in dynamic shear modulus for the rubber asphalt, in comparison with the neat binder, is more prominent at lower frequencies. In addition, there is a decrease in the slope of the curve of the rubber asphalt as a result of the asphalt binder modification. Thus, the addition of the rubber modifier results in a reduction of dependency of |G*| on frequency/temperature.

In terms of phase angle, the addition of rubber caused a reduction of phase angle for all the frequencies in relation to the neat binder. The phase angle varied from approximately 59 to 68 ° for the rubber asphalt and from approximately 78 to 88 ° for the neat binder. This reduction in phase angle indicates that the rubber asphalt binder is more elastic than the neat binder. Typically, modified asphalt binders show a higher |G*| and a lower δ, which means they are, in general, stiffer and more elastic than neat binders.

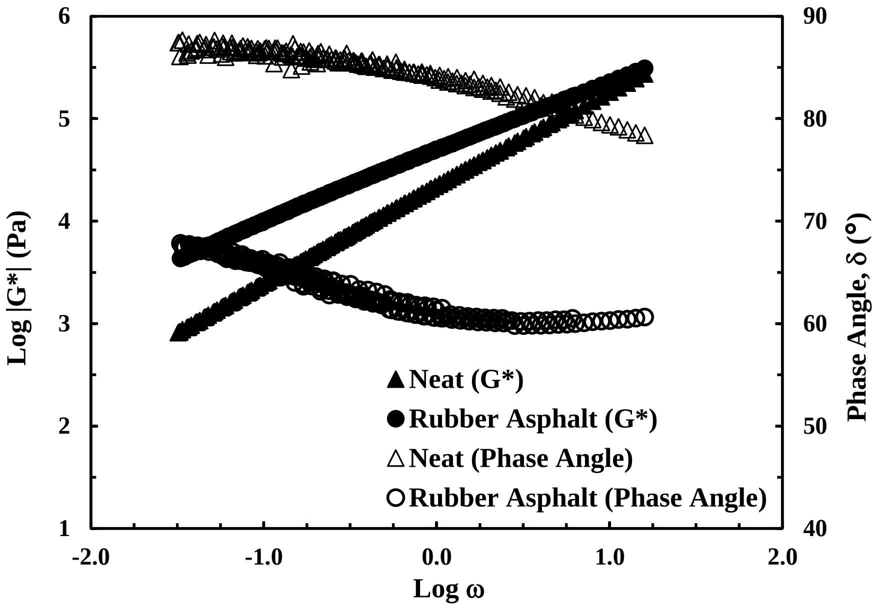



**Figure 5.1.** Master curves for dynamic shear modulus (|G*|) and phase angle (δ) at 46°C for the neat and rubber asphalt binders

The effects of conducting the RTFOT test at 163 and 175°C on the rubber asphalt binder are shown in Figure 5.2. The test was performed at 175°C to allow for proper aging, since the binder did not roll inside the glass bottle at 163°C, and to better simulate field conditions. Aging the binder at a higher temperature resulted in a decrease of phase angle and an increase in dynamic shear modulus at lower frequencies (stiffer binder). Thus, fatigue testing of the binder was conducted on samples aged at 175°C (worst scenario for fatigue characterization).

The results for the time sweep tests (TST) are shown in Figure 5.3 for the strain-controlled fatigue tests performed on the neat and rubber asphalt binders at 20°C and at a frequency of 10 Hz for 2%, 3%, and 4% strains. The fatigue life ($N_f$), determined as the number of cycles for a 50% reduction of the initial dynamic shear modulus, for the rubber asphalt is significantly higher (approximately 7 to 11 times) than the neat binder for the range of strains tested. Other researchers have also reported higher fatigue life of the rubber asphalt binder in relation to the neat binder (Giuliani & Merusi, 2010).

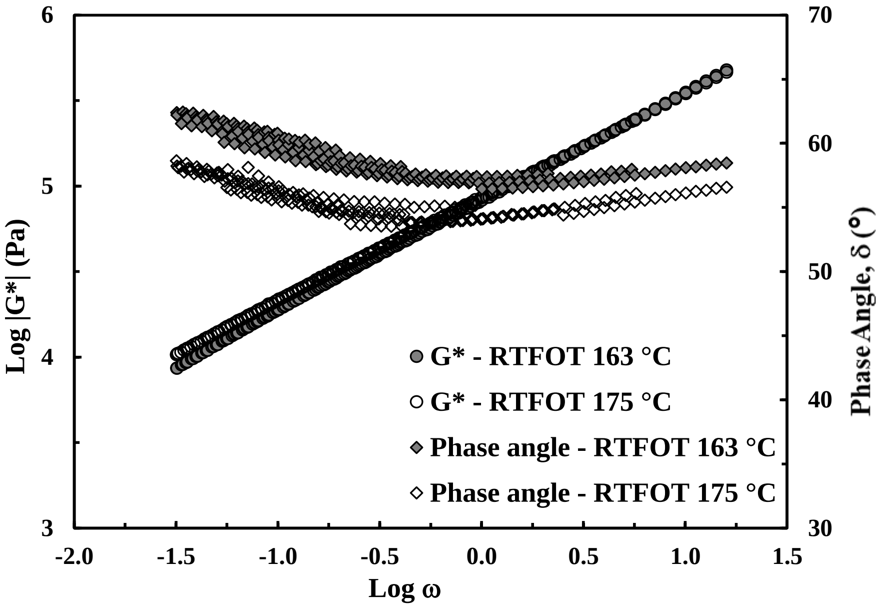



**Figure 5.2.** Dynamic shear modulus (|G*|) and phase angle ($\delta$) master curves at 46°C for rubber asphalt after RTFOT aging at 163°C and 175°C

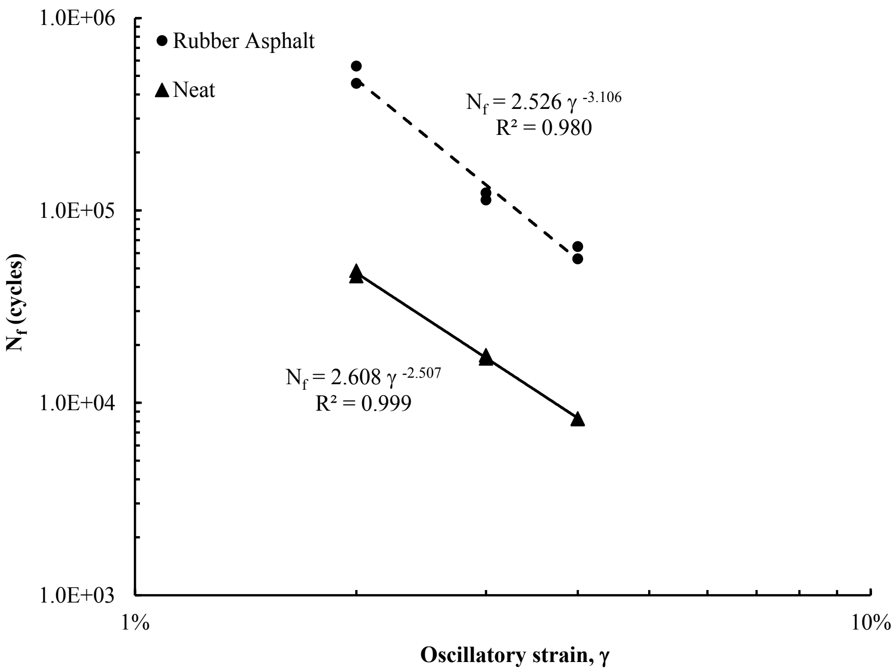



**Figure 5.3.** Fatigue laws for the neat and rubber asphalt binders derived from the TST tests

Fatigue laws were developed based on the results of replicate specimens for the neat and rubber asphalt binders (Table 5.4). The rubber asphalt exhibited a fatigue life of approximately 500,000 cycles for the lowest strain level (2 %), whereas the fatigue life of the neat binder for the same strain level was just below 50,000 cycles. The rubber asphalt, on the other hand, exhibited higher coefficient of variations (5.9 to 14.6 %) than the neat binder (0.5 to 4.5 %). The coefficient of determination of 0.999 and 0.980 were obtained for the neat and rubber asphalt binders, respectively.

**Table 5.4 -** Results of the Time Sweep Tests for the neat and rubber modified binders at 20°C and 10 Hz

| Binder | Strain (%) | $N_f$ (Cycles) | | a | b | $R^2$ |
|---|---|---|---|---|---|---|
| | | Mean | Coefficient of variation | | | |
| Neat | 2 | 47,104 | 4.5% | 2.608 | -2.507 | 0.999 |
| | 3 | 17,333 | 2.7% | | | |
| | 4 | 8,268 | 0.5% | | | |
| Rubber asphalt | 2 | 509,675 | 14.6% | 2.527 | -3.106 | 0.980 |
| | 3 | 118,391 | 5.9% | | | |
| | 4 | 60,434 | 10.5% | | | |

For the Linear Amplitude Sweep (LAS) test, the change in shear stress as a function of the applied shear strain is shown in Figure 5.4 for two samples of each binder. The stress-strain curves for duplicate specimens of both binders are very similar, which shows a good test repeatability. The neat binder shows a fast increase in shear stress with increasing strain level, reaching a peak value of 900 kPa at approximately 8% strain. The shear stress then falls sharply until the 15% strain level and then slowly approaches zero until approximately 25% strain. The rubber asphalt, on the other hand, shows a different stress-strain behavior. The peak value of 450 kPa (approximately 50% lower than the neat binder) is slowly reached at an approximate 12% strain level and the shear stress is slowly dissipated until the end of the test (30% strain level). In addition, the shear stress are still at about 200 kPa at the high strain level. Similar stress-strain behavior was also observed by Micaelo *et al*. (2015) when comparing neat and modified binders.

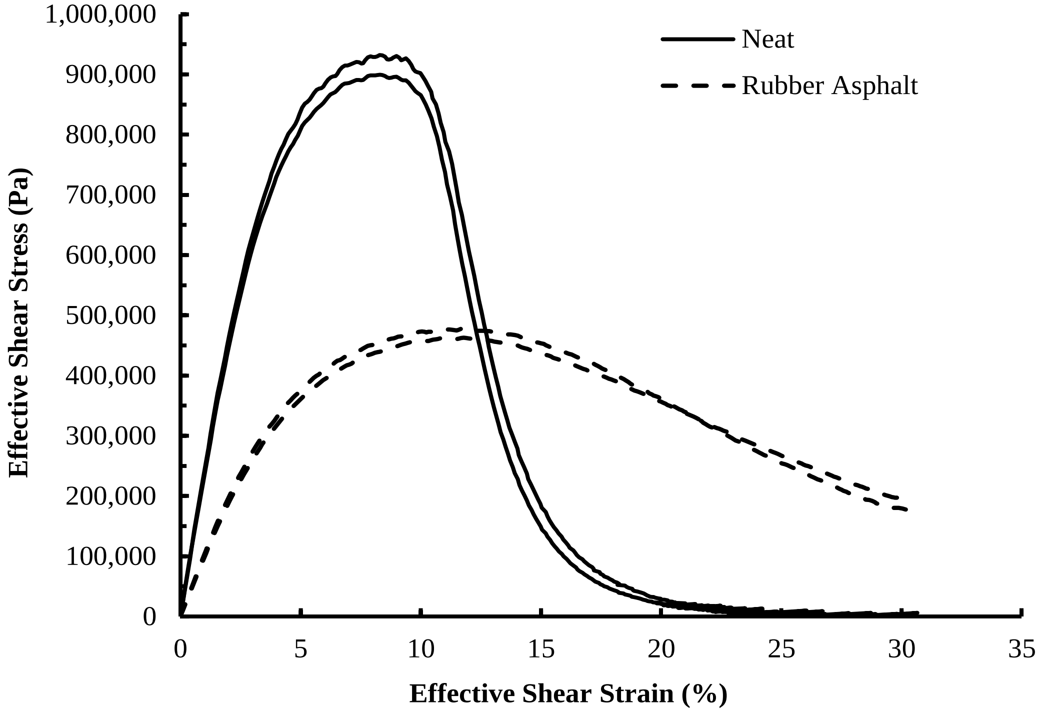



**Figure 5.4.** Stress-strain curves for the neat and rubber asphalt binders in the LAS test

In order to determine the damage characteristic curve for the binder in the LAS test, the $|G^*|\cdot\sin\,\delta$ was taken as the integrity parameter, C, and compared to the development of the damage intensity, D. The damage characteristic curves for sample A of the neat and rubber asphalt binders are shown in Figure 5.5 along with the fitted curves. It is important to note that the curves obtained for sample B were similar to those for sample A.

As expected, the integrity of both binders decreases as the damage intensity increases. However, the decrease in integrity is faster for the neat binder when compared to the rubber asphalt. In addition, the neat binder reaches zero integrity at a damage intensity of 250, whereas the rubber asphalt still exhibits some integrity (roughly 0.1) at the end of the test at a damage intensity of about 450. In addition, the rubber asphalt failed at a damage intensity of 94, whereas the neat binder failed at damage intensity of 58.

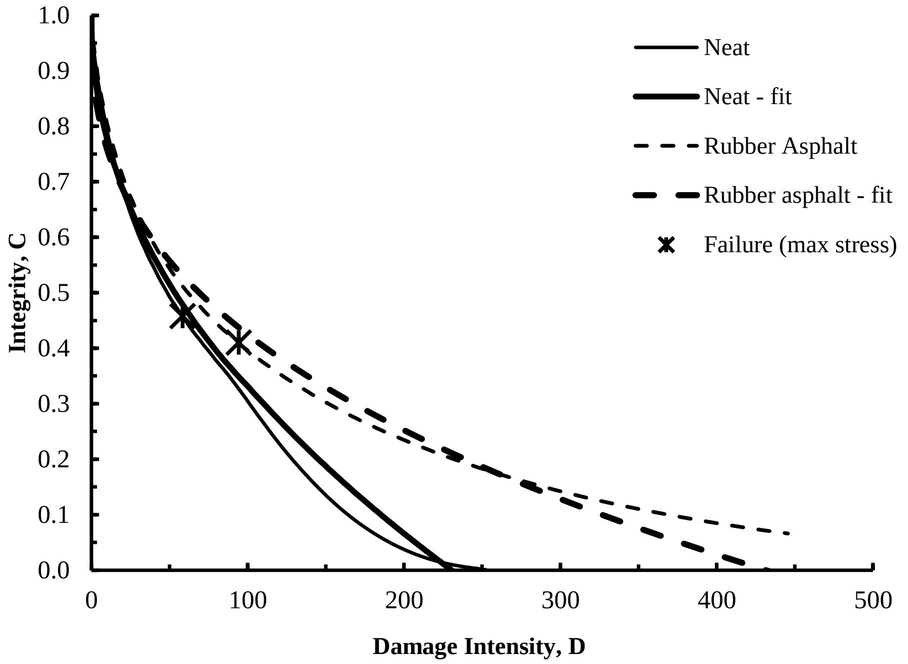



**Figure 5.5.** Integrity versus damage intensity (damage characteristic curve) for the neat and rubber asphalt binders in the LAS test

The model inputs for the LAS test are summarized in Table 5.5 for both binders along with the fatigue life for two strain levels (2 and 4% strain). The model parameters A and B for the LAS are similar for the replicate specimens of the same binder. As expected, the A parameters for the rubber asphalt are larger than the ones obtained for the neat binder. Typically, an increase in A parameter indicates an increase in fatigue resistance. The addition of rubber resulted in an average 11.4 and 7.6-fold increase of fatigue life for 2 and 4% strain levels, respectively. The average increase in fatigue life due to the presence of rubber is shown in Figure 5.6 for up to a 30% strain level. The average increase in fatigue life due to the rubber is highest at lower strain levels, ranging from 17.2-fold (1% strain) to 2.3-fold (30% strain).

**Table 5.5 -** Model inputs for the LAS test for the neat and rubber asphalt binders

| Binder | Sample | A | B | $R^2$ | 2% $N_f$ | 4% $N_f$ |
|---|---|---|---|---|---|---|
| Neat | A | 1.621E+05 | -2.845 | 0.993 | 22,568 | 3,141 |
| | B | 1.629E+05 | -2.765 | 0.991 | 23,961 | 3,525 |
| Rubber asphalt | A | 2.893E+06 | -3.346 | 0.980 | 284,526 | 27,979 |
| | B | 2.713E+06 | -3.452 | 0.981 | 247,936 | 22,660 |

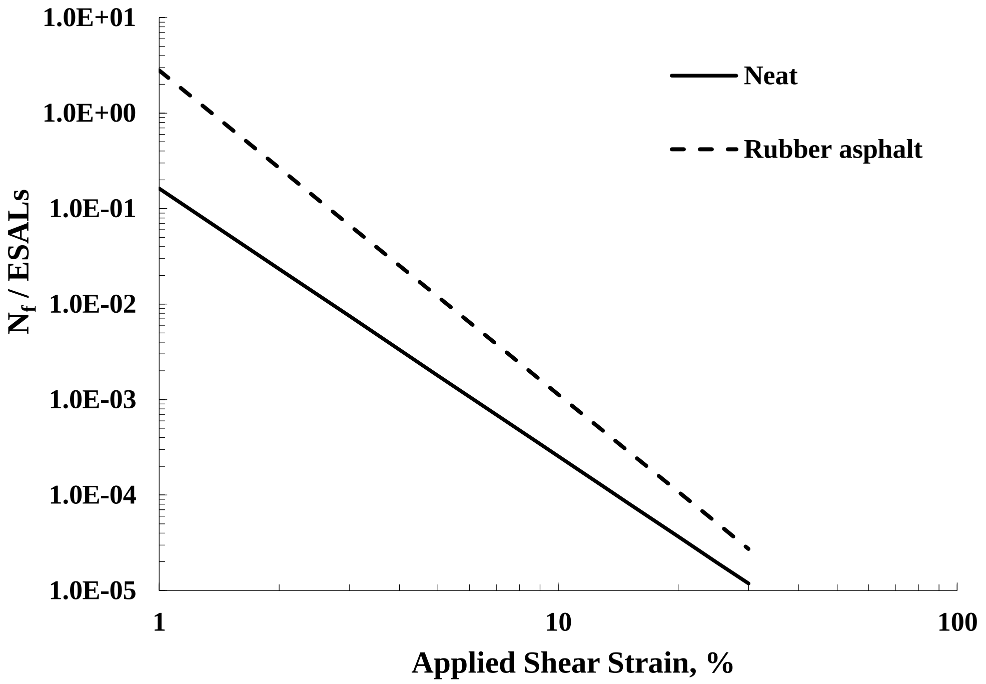



**Figure 5.6.** Average fatigue parameter, $N_f$ (normalized to 1 million ESALs), as a function of the applied shear strain for the neat and rubber asphalt binders

The average number of cycles to failure in the TST test was approximately 2.3 times higher than the number of cycles in the LAS test for the neat binder. Similarly, the average number of cycles to failure in the TST test was approximately 2.0 times higher than the number of cycles in the LAS test for the rubber asphalt. It is important to note that different criteria were used for determining failure in the TST and LAS tests, which may account for the observed differences in fatigue life. However, regardless of the methodology used to assess the fatigue life of the binders, the rubber asphalt exhibited a superior resistance to fatigue than the neat binder for the range of strain levels tested. As a result, asphalt mixtures using the field-blended rubber asphalt binder tested herein are expected to provide a good resistance to fatigue cracking in the field. A verification of the fatigue behavior of an asphalt mixture using this asphalt binder is detailed next.

### *5.4.2. Mixture Fatigue*

The fatigue life, in cycles to failure, of the field and laboratory-compacted specimens are shown in Figure 5.7 as a function of the tensile strain. The fatigue life of the laboratory-compacted specimens is higher than the fatigue life of the field-compacted specimens for the range of strains tested (300 to 800 με). However, even though the laboratory-compacted specimens exhibited higher fatigue lives, the tendency is reversed at a strain of approximately 225 με and the field specimens will have higher fatigue lives for typical tensile strains expected in the field (between 100 and 200 με).

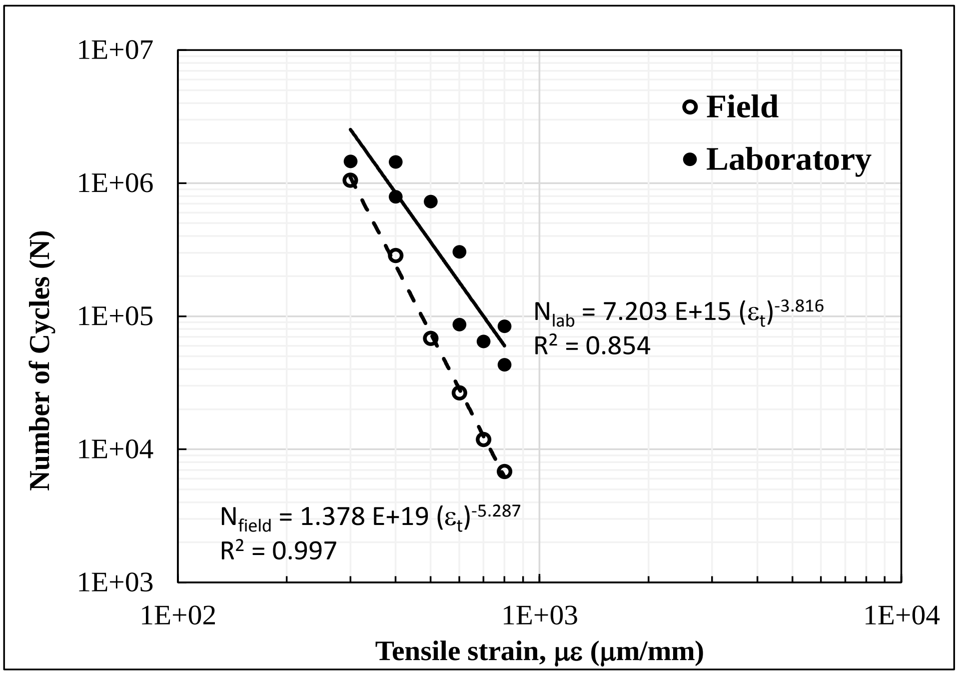



**Figure 5.7.** Fatigue life of the field and laboratory-compacted specimens of the field-blended rubber asphalt mixture as a function of tensile strain

The $k_1$ and $k_2$ coefficients, along with the correlation coefficient ($R^2$) and the number of cycles at a strain level of 100 με ($N_{100}$) are presented in Table 5.6. The fatigue law based on tests conducted on laboratory-compacted specimens exhibited a lower coefficient of determination (0.854) than the fatigue law for field-compacted specimens (0.997), which is an indicative that field compaction resulted in more homogeneous specimens than the ones fabricated in the laboratory. In addition, the $N_{100}$ for the field-compacted specimens is 2.2 times higher than the $N_{100}$ for the laboratory- compacted specimens. The variability for the fatigue lives exhibited by the laboratory specimens may have contributed to the inversion of tendency between the field and laboratory fatigue lives, since the slope of the lines are not parallel.

**Table 5.6 -** Fatigue laws for the field and laboratory-compacted specimens of the field-blended rubber asphalt mixture

| Condition | a | b | $R^2$ | $N_{100}$ |
|---|---|---|---|---|
| Laboratory | 7.203 E+15 | -3.816 | 0.854 | 1.65E+08 |
| Field | 1.378 E+19 | -5.287 | 0.997 | 3.63E+08 |

Another factor that may have influenced the results is the observed stiffness during the flexural bending tests. The average flexural beam stiffness for the field and laboratory-compacted specimens determined in the repeated flexural bending test (four point bending) are shown in Figure 5.8. The average initial flexural beam stiffness for the field specimens (5,638 MPa) is significantly higher than the average initial flexural beam stiffness of the laboratory specimens (2,320 MPa). Thus, all things being equal, the higher stiffness for the field compacted specimens could account for the reduction of fatigue life in comparison with the laboratory specimens for the range of strains tested.

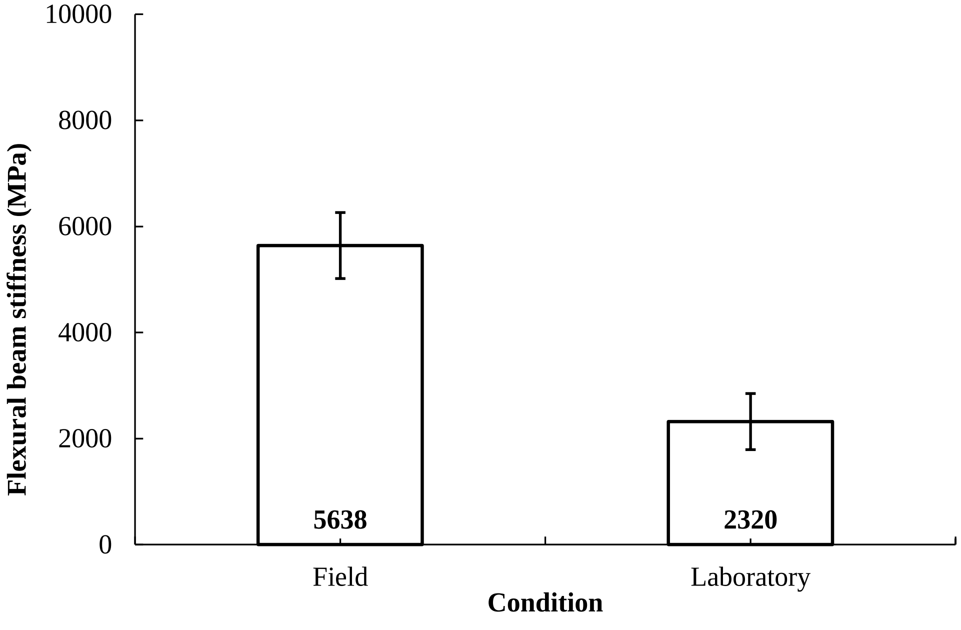



**Figure 5.8.** Average flexural beam stiffness determined in the four point bending test

The higher stiffness exhibited by the field specimens in relation to the laboratory specimens may be attributed to possible differences in air void content or in the mixing process in the field (binder stiffness). Further analysis showed that the void content for the field specimens varied from approximately 4 to 5%, whereas the void content for the laboratory specimens varied from approximately 6 to 7%. Typically, higher void contents and the mixing process in the field (higher binder stiffness) result in a higher stiffness for asphalt mixtures. On the contrary, higher void contents and the mixing process in the field result in a reduction of fatigue life for asphalt mixtures. Thus, it appears that a combination of the air void content and binder stiffness between the laboratory and field specimens may have had an impact on the obtained fatigue laws. In addition, a possible difference in the arrangement of the mineral skeleton of the field and laboratory specimens may have also played a role in the observed stiffness.

The results indicate that the field-blended rubber asphalt mixture used in RJ-122 has a good resistance to fatigue when compared to other data in the literature. Fontes *et al.* (2008) studied a continuous blend asphalt fabricated in the laboratory in a dense graded mixture (Asphalt Institute type IVb gradation). The binder for the continuous blend had 21% of rubber and asphalt content of 8.0%, with an air voids of 5.0%. The binder viscosity was 1644 cP at 175°C (spindle 27), which is slightly lower than the viscosity of the field-blended binder in the present study (1975 cP). A fatigue life of approximately 4.0 x$10^5$ and 2.0 x $10^4$ was obtained at the 400 and 800 strain levels, respectively. The results found in the present study for laboratory-prepared specimens show a higher fatigue life for both strain levels. The average fatigue life for a strain level of 400 $\mu\varepsilon$ was approximately 1.0 x$10^6$ (2.8 times higher), whereas the fatigue life for 800 $\mu\varepsilon$ was approximately 6.4 x $10^5$ (3.8 times higher).

In another study, the fatigue behavior of a terminal-blended rubber asphalt binder, produced with 20% of rubber obtained from ambient grinding and an AC 50/70 pen asphalt, in a CALTRANS gap-graded mixture was studied using four point bending tests (Fontes *et al*., 2009). The gap-graded mixture had 8.5% of asphalt content and 6.0% air voids. The binder a high viscosity at 175°C (2,179 cP), which is higher than the viscosity of the field-blended rubber asphalt in this study. A fatigue life of approximately 8.0 x$10^4$ and 5.0 x $10^4$ were obtained at the 400 and 800 strain levels, respectively. The results obtained in the present study for laboratory-prepared specimens also show a higher fatigue life for both strain levels. The average fatigue lives were approximately 1.4 and 1.3 times higher for the 400 and 800 $\mu\varepsilon$ strain levels, respectively.

### 5.4.3. Field Performance

The 500-m test section in RJ-122, periodically monitored using the Pavement Scanner, showed no fatigue cracking after 4 years of service life, except for a transversal crack of approximately 1 meter in length detected at the 34.965 kilometer mark after 3 years. It is important to note that the crack appeared in a region where deflections were the highest before and after the job. Therefore, the cracking may have occurred due to the underlying pavement conditions.

In addition to periodic pavement monitoring, full scale accelerated pavement tests were conducted. The load applications yielded 2.47 million equivalent single axle loads (ESALS) according to the American Association of State and Highway Transportation Officials (AASHTO), which is approximately 7 years of traffic for RJ-122. This is a typical time interval for overlaying existing pavements in Brazil (7 to 8 years). No cracking was observed during f-sAPT simulation, which indicates that fatigue life was not reached during the 7 years of traffic simulated. These results are in agreement with results obtained in periodic field monitoring campaigns, since only a transversal crack of approximately 1 meter in length was observed after 4 years of service life of the 500-m test section.

The results confirm binder and mixture testing conducted in the laboratory. In addition, they show a better resistance to cracking under f-sAPT simulation than other tests conducted in Brazil with terminal-blended rubber asphalt mixtures. At BR-116, in São Paulo, the test section with 5.5% asphalt content reached its service life at a number of ESALS 30% lower than the one simulated at RJ-122. Similarly, a test section with 6.0 % asphalt content at BR-116, in Rio de Janeiro, exhibited 40% cracking of the testing area at a number of ESALS 50% lower than the one simulated at RJ-122 (Vale, 2008).

## 5.5. Conclusions

This study was conducted in order to evaluate the fatigue behavior of a field-blended rubber asphalt recently used in a gap-graded mixture in Brazil for the first time. The fatigue behavior of the field-blended rubber asphalt binder was first evaluated and compared to the neat binder. Then, the asphalt mixture was assessed in the laboratory and in the field, in order to characterize the fatigue behavior at different levels. Based on this data, the following conclusions can be drawn:

- The fatigue behavior of the field-blended rubber asphalt determined with the time sweep and the linear amplitude sweep tests exceeds the one for the neat binder.
- Rubber modification resulted in a binder with better stress-strain behavior and the damage characteristic curves for the binders show that the rubber asphalt has a better response to damage than the neat binder (higher integrity for the same damage level).
- Fatigue behavior of the neat binder and rubber asphalt may be estimated from strong relationships (fatigue laws) derived for the binder in the TST and LAS tests.
- Flexural bending tests on prismatic specimens (four point bending) indicate a good resistance to fatigue for the field-blended rubber asphalt in a gap-graded mixture.
- Fatigue laws were developed for the field-blended rubber asphalt mixture (field and laboratory-compacted mixtures) and field specimens showed a better fatigue behavior for the expected strains in the field, possibly because of a combination of the differences in air void content and the mixing processes for the field and laboratory specimens.
- When compared to the fatigue life of similar mixtures available in the literature, the expected fatigue lives for the field-blended rubber asphalt mixture used in RJ-122 are higher, which is an indicative that the mixture shows a good resistance to fatigue cracking.

- Pavement performance in periodic pavement monitoring and full-scale accelerated pavement testing conducted in tests sections in RJ-122 confirmed the results obtained at the binder and mixture level.

# 6. CASE HISTORY STUDY: FIELD MONITORING AND PERFORMANCE PREDICTION OF A FIELD-BLENDED RUBBER ASPHALT MIXTURE IN BRAZIL

## 6.1. Introduction

The vast majority of roads in Brazil are composed of flexible pavements, in a country were more than 60% of the freight transport is through highway transportation (CNT, 2015). Even though highway transportation is the main transportation mode, there is still an urgent need for new technologies for the construction and rehabilitation of pavements. This lack of road network, associated with elevated traffic conditions carrying, at many times, excessive loads that have exponential effects on pavement degradation, implicates in undesirable pavement conditions.

One way to extend the durability of pavements is by using binders that promote the enhancement of the mechanical properties of the asphalt mixtures. The mechanical properties of binders can be incremented, among others, by using polymers (RET, SBS, SBR, etc.) and crumb rubber modifier from tire wastes. Rubber modification techniques have been used worldwide in the past few decades (particularly in the North-American states of Texas, Arizona, Florida, and California) and nationally, for more than 10 years (Leite *et al*., 2000; Oda & Fernandes Jr., 2000; Bertollo *et al*., 2000). According to the Brazilian Association of Highway Concessionaries (ABCR), rubber asphalt was used in approximately 1,867 kilometers of conceded highways (roughly 10% of the total extension granted) between the years of 2008 and 2012 (ABCR, 2013).

It is well known that rubber modification results in gains of the rheological properties of binders (Bahia & Davies, 1994; Moreno-Navarro *et al*., 2015), as well as an improved resistance to aging (Sainton, 1990), resulting in an asphalt mixture of high performance and a prolonged pavement service life. According to the California Department of Transportation, the advantages of using rubber asphalt in comparison to conventional binders are increased viscosity, which allows for greater film thickness in paving mixes without excessive drain down or bleeding, and increased elasticity at higher temperatures. In addition, rubber asphalt mixtures result in pavements with increased durability, increased resistance to surface initiated (top-down) and fatigue/reflection cracking, reduced temperature susceptibility, improved aging and oxidation resistance, improved resistance to rutting (permanent deformation), and lower pavement maintenance costs due to improved pavement durability and performance (CALTRANS, 2006).

Among the fabrication processes available for obtaining rubber asphalt, terminal blend is the most commonly used in Brazil. However, the use of field-blended rubber asphalt has been around in the United States for decades, especially in the state of Arizona. The field blending process for rubber asphalt results in a highly viscous material with enhanced engineering properties, but requires a specific equipment that is typically installed at the job site or close to the supplying asphalt plant.

The lack of information regarding this technique in Brazil, however, inhibits a more common use in pavement construction/rehabilitation. For instance, in a survey conducted in the United States in 2011, 42 of the 51 departments of transportation in the country answered a questionnaire regarding crumb rubber modified binders. As a result, forty-two percent of the state departments that have never used these types of binders have not done so because its performance is still uncertain (Bandini, 2011).

Thus, keeping in mind the possible technological advantages of using a field-blended rubber asphalt mixture and the lack of information regarding this technique in Brazil, there is a necessity to develop studies to assess the performance of this type of material in our environmental conditions and axle loading configurations. Moreover, pavement engineers need to know how to deal with pavements using field-blended rubber asphalt that will yield equal or better performance than pavements constructed with conventional asphalt mixtures.

This case history study was conducted in order to characterize the field performance of the first pavement rehabilitation job in Brazil using a field-blended rubber asphalt mixture and attempt to predict its future performance. A 500-meter test section using a field-blended rubber asphalt mixture with a gap-graded aggregate gradation as a surface course was selected for periodic monitoring through pavement deflection tests, roughness, rutting, and surface conditions. In addition, accelerated pavement testing (f-sAPT) was conducted in a representative test section using a full scale, mobile traffic simulator. Results from the f-sAPT simulation were then used to calibrate the performance models in the Highway Development and Management Model (HDM-4).

### 6.2. Project Description

A portion of highway RJ-122 in Cachoeiras de Macacu, Rio de Janeiro, Brazil, was chosen as a test section for monitoring. The project segment runs from highway BR-116 (Guapemirim, RJ) to highway RJ-116 (Cachoeiras de Macacu, RJ), with a total length of 35.6 kilometers. It is a two-lane, single carriageway with an average daily traffic (ADT) of approximately 1,923 in 2009 with a high truck percentage.

The existing pavement, originally constructed in the early 70´s, consisted of an existing asphalt layer with extensive fatigue cracking (110 mm HMA and 90 mm of bituminous macadam), and base and sub-base layers composed of selected soil. Typically, the rehabilitation design would consist of total reconstruction of the pavement. However, the Rio de Janeiro State Department of transportation (DER-RJ), with the collaboration of several international experts, opted to use field-blended rubber asphalt mixtures as an alternative solution. This was the first Brazilian experience with field-blended rubber asphalt. The rehabilitation project was designed by Consulpav (Sousa, 2009).

The rehabilitation project by Consulpav consisted of applying three different methods of analyses:

- The CALTRANS' method, which is a method based on the deflections under the load;
- An empirical-mechanistic method proposed by SHELL Company. The method consists of controlling the deflection at the top of the soil foundation in order to minimize permanent deformations and to control tensile strain and stresses at the bottom of the bituminous layers to minimize cracking of such layers; and
- An empirical-deterministic method, developed by CONSULPAV for Recycled Asphalt Pavements, based on an effort to control stresses at the bottom of reinforcement layers in order to minimize crack reflection.

The expected number of equivalent single axle loads (ESAL) for a 20 year design period was determined using the Rubber Pavements Association load equivalency factor (LEF), which led to 2.46 million ESALS (7.02 million using AASHTO LEFs). The general design suggested for the rehabilitation of RJ-122 consisted in a surface friction course composed of a 25 mm rubber asphalt layer with in an open-graded type mix and a 45 mm layer of a rubber asphalt layer in a gap-graded gradation, both using a field-blended rubber binder. A levelling course layer with conventional binders was used along with localized treatment of highly degraded areas. Shoulders were built all along RJ-122 and the drainage system was improved. Also, the open-graded mixture was replaced by 40 mm of the gap-graded mixture in some portions of highway RJ-122 (close to urban perimeters and in areas with intersections) due to intense transversal traffic loading, since the open-graded mixture is more susceptible to drag under shear stresses.

The project design for the portion of highway that runs from the 34.000 kilometer to the 35.100 kilometer mark consisted of overlaying the existing cracked pavement with 85 mm of field-blended rubber asphalt mixture in two lifts (45 mm on the bottom layer and 40 mm at the surface). The overlay was preceded by a levelling course with conventional binders with approximately 30 mm. It is important to note that the asphalt overlay was laid directly over the cracked pavement, with no previous corrective measures taken. Pavement conditions before and after the rehabilitation job are shown in Figure 6.1.

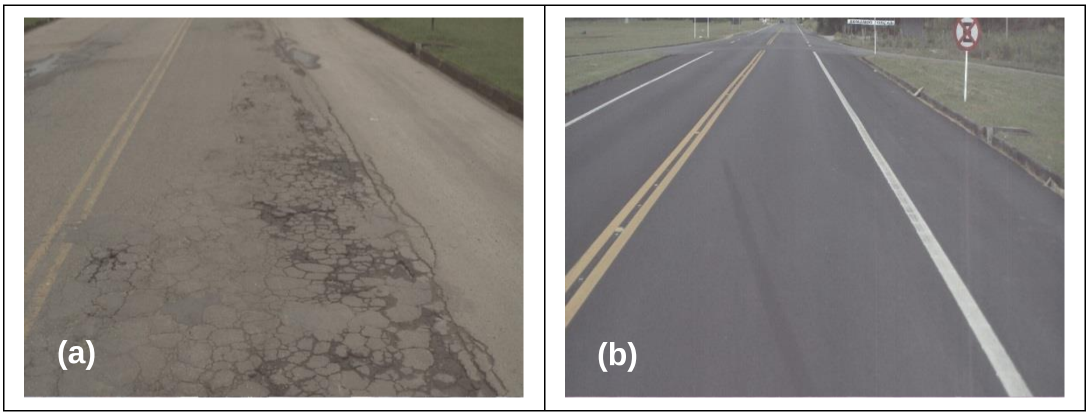



**Figure 6.1.** Pavement conditions at the 34.600 kilometer mark (a) before and (b) after rehabilitation

## 6.3. Materials

### *6.3.1. Aggregates and Mix Gradations*

The mix gradation obtained for the test section is a gap-graded similar to that specified by the Arizona Department of Transportation. Three granite crushed stone fractions from Sigil stone quarry, in Araruama, RJ, were used for composing the gap-graded mixture and cement was chosen as the mineral filler. The mean grain size distribution curve for the test section is shown in Table 6.1 along with ADOT specifications.

**Table 6.1 -** Grain size distribution curve for the RJ-122 test section

| Sieve | Diameter (mm) | Mean Curve (%passing) | ADOT Specifications | |
|---|---|---|---|---|
| | | | Lower limit | Upper limit |
| 3/4" | 19.000 | 100 | 100 | 100 |
| 1/2" | 12.500 | 87.4 | 80 | 100 |
| 3/8" | 9.520 | 69.8 | 65 | 80 |
| Nº 4 | 4.800 | 22.7 | 28 | 42 |
| Nº 8 | 2.360 | 13.0 | 14 | 22 |
| Nº 200 | 0.075 | 2.1 | 0 | 2.5 |

### *6.3.2. Crumb Rubber*

The crumb rubber used for modifying the binder was obtained from *Ecobalbo Reciclagem de Pneus Ltda*, in Cravinhos, São Paulo, Brazil. The crumb rubber was composed of tires both from automotive and truck scrap tires at equal parts by weight (roughly 1 truck tire for every 4 automotive tires).

The crumb rubber modifier, obtained from ambient grinding, showed a gradation meeting ADOT specifications for Type B crumb (ADOT, 2000) with 100% passing the #16 sieve (1.18 mm). The grain size distribution for the crumb rubber is shown in Table 6.2 along with ADOT specifications.

**Table 6.2 -** Grain size distribution curve for the crumb rubber modifier along with ADOT specifications

| Sieve | Diameter (mm) | % passing | % passing (ADOT Specs) | |
|---|---|---|---|---|
| | | | Type A | Type B |
| Nº 8 | 2.360 | 100 | 100 | |
| Nº 10 | 2.000 | 100 | 95-100 | 100 |
| Nº 16 | 1.180 | 100 | 0-10 | 65-100 |
| Nº 30 | 0.600 | 59 | | 20-100 |
| Nº 50 | 0.300 | 24.6 | | 0-45 |
| Nº 200 | 0.075 | 3.5 | | 0-5 |

### *6.3.3. Asphalt Binder*

An asphalt binder of penetration grade 30/45 (AC 30/45) was used to mix with the CRM previously described. The physical properties of the conventional binder provided by the asphalt distributor are summarized in Table 6.3.

**Table 6.3 -** Physical properties of the conventional binder AC 30/45

| Property | ASTM standard | Requirements | AC 30/45 | Units |
|---|---|---|---|---|
| Penetration | D 5 | 30 to 45 | 38 | 0.1 mm |
| Softening Point | D 36 | > 52 | 52.7 | º C |
| Brookfield Viscosity 175°C-SP21 | D 4402 | 76 to 285 | 86 | cP |
| Ductility 25°C | D 113 | >60 | > 150 | cm |
| Solubility in Trichloroethylene | D 2042 | >99.5 | 99.97 | % mass |
| Flash Point | D 92 | >235 | > 300 | º C |

The physical properties of the crumb rubber modified binders are summarized in Table 6.4 for the three CRM contents tested (18, 19, and 20%) and a 60-minute reaction time as specified by the design project. Results showed that, unlike the other physical properties, binder viscosity varied significantly (from 2,700 to 4,200 cP), and is dependent on CRM content. The CRM content was determined as a function of the viscosities achieved. Even though all three CRM contents met the design targets, the CRM content of 19% was selected for the test section. The binder viscosity was also in accordance with the Brazilian specification for an AB-22 rubber asphalt (2,200 – 4,000 cP).

**Table 6.4 -** Physical properties for 60 minutes of reaction time and varying CRM contents along with the target values

| Property | CRM Content | | | Design Targets |
|---|---|---|---|---|
| | **20%** | **19%** | **18%** | |
| Brookfield Viscosity 175ºC (cP) | 4175 | 3537 | 2737 | 1500 - 5000 |
| Penetration (0.1 mm) | 20.0 | 21.0 | 20.5 | 20-75 |
| Resilience (%) | 41.3 | 32.0 | 30.6 | ≥ 25 |
| Softening Point (ºC) | 70.5 | 69.4 | 70.1 | ≥ 54 |

### *6.3.4. Asphalt Mixture*

The mixture design was done using Marshall compaction in order to determine the optimum binder content. The volumetric properties and optimum binder content for the field-blended rubber asphalt (19% CRM) are presented in Table 6.5 for a 75-blow energy. Because there isn´t a Brazilian specification for this mixture, the results are compared with the Brazilian specification DNIT 112/2009 for a rubber asphalt terminal blend or “wet process” (DNIT, 2011a).

**Table 6.5 –** Volumetric properties of the field-blended rubber asphalt mixture

| Property | Mixture | Limits (DNIT 112/2009) |
|---|---|---|
| Air Voids (%) | 5.6 | 4-6 |
| Optimum asphalt content (%) | 8.0 | 5-8 |
| Voids in Mineral Aggregate (%) | 23.1 | >14 |
| Stability (kN) | 7.28 | >7.0 |

The unconfined Dynamic Modulus (|E*|) of the rubber asphalt mixture was determined using an MTS servo-hydraulic testing machine. Testing was conducted in accordance with AASHTO T342-11 (AASHTO, 2011) on duplicate specimens for a variety of frequencies (0.1, 0.5, 1.0, 5, and 10 Hz) and temperatures (4.4, 21.1, 37.8, and 54.0°C). A temperature of -10°C could not be achieved for this test because the cooling system available could not reach such low temperatures. The average air void content of the cored specimens was 5.1%. The master curve for |E*| for a reference temperature of 21.1°C is presented in Figure 6.2.

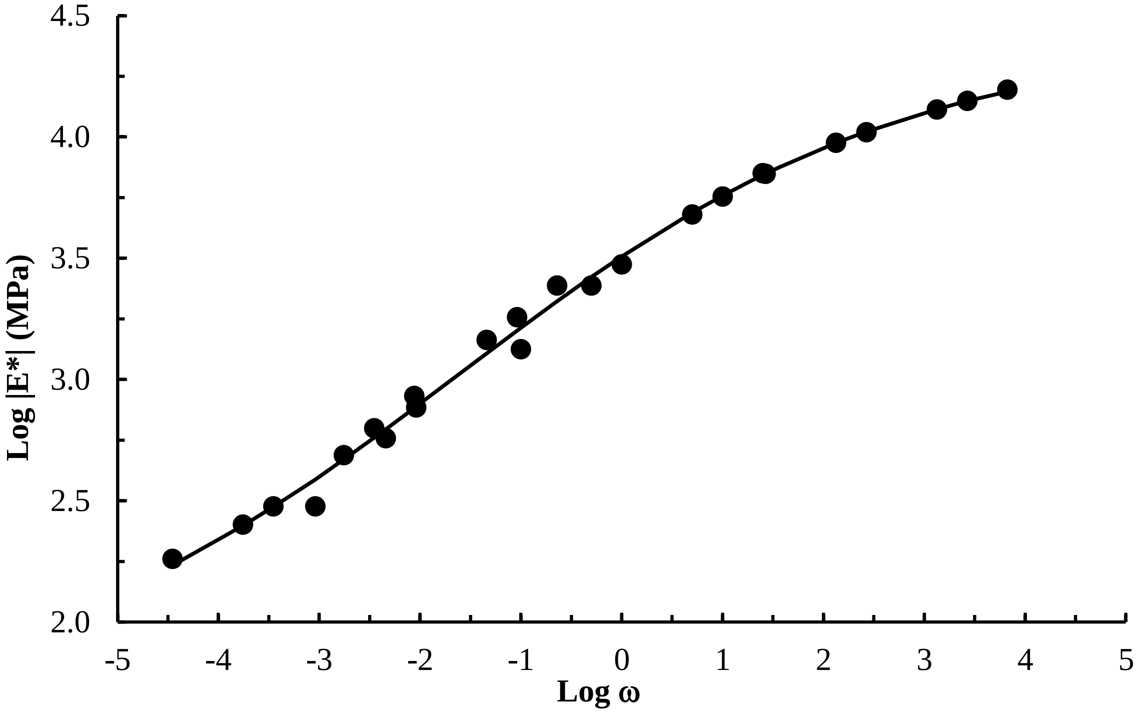



**Figure 6.2.** Master curve for |E*| for the field-blended rubber asphalt mixture

## 6.4. Methods

### *6.4.1. Field Monitoring*

The roadway stretch between the 34.5 and 35.0 kilometer marks was chosen for field monitoring. The last 100 meters of highway RJ-122 were not monitored because this portion was closed for traffic. The structural and functional characteristics of the 500-meter test section were monitored through non-destructive testing. Tests were conducted before and after the rehabilitation job, as well as up to 4 years of service life in order to verify pavement performance under local environmental and traffic conditions. The functional characteristics were determined by monitoring surface distress, roughness, rutting, and friction, whereas the structural conditions were determined by monitoring pavement deflections.

#### *6.4.1.1. Structural Conditions*

Structural conditions were determined by monitoring pavement deflections using a Dynatest 800 Falling Weight Deflectometer (FWD) impact load test at 20-meter intervals (40 meters in each direction) before and after the job, as well as after 3 and

4 years of service life. Tests were performed using a standard load of 41 kN on a 300 mm diameter plate to simulate the passage of a 82 kN standard axle load. The maximum allowable deflection in the design was 0.65 mm.

##### *6.4.1.2. Functional Characteristics*

Surface distress surveys were conducted by visual inspection before and after the rehabilitation job according the Brazilian standard DNIT 006/2003-PRO for flexible pavements (DNIT, 2003). Observations were recorded at 20-meter intervals. A Pavement Scanner was used in lieu of visual inspection to determine pavement surface conditions after 3 and 4 years of service life.

The Pavement Scanner is a vehicle mounted with a Laser Crack Measurement System (LCMS) that allows the automatic detection of cracks and the evaluation of rutting, macrotexture, and other road surface features. Detailed information about the Pavement Scanner can be found elsewhere (Fialho, 2015). Rutting was also determined using the Pavement Scanner, which uses 4,000 points to determine the maximum rut depth in a lane, at every 1 m in each wheel track. Rut depth was defined as the maximum rut between the two wheel paths for each direction.

Pavement roughness was determined using a Dynatest 5051 Mk III Road Surface Profilometer, which is a Class I according to the ASTM E950 classification system (ASTM, 2003). Roughness was determined before and after the job, as well as after 3 and 4 years of service life. Longitudinal road profile was calculated in terms of International Roughness Index (IRI) at 100-meter intervals.

Skid resistance was measured using a Findlay Irvine MK2-D Grip Tester (GT), which is a continuous measurement device used to measure the GripNumber (GN) under fixed slip conditions to simulate the breaking of a vehicle with antilock brakes (Lu & Steven, 2006). Testing was conducted at a constant speed of 50 km/h on a wet pavement with approximately 0.50 mm of water film, which is the film typically used (Henry, 2000). The GN was calculated for 100-meter intervals.

A GN of 0.52 was specified for the job, which can be classified as rough according to the Brazilian Highway Research Institute´s (IPR) skid resistance classification system (IPR, 1998). Skid resistance was measured after approximately 2 years of service life. It was not measured before the job because the distinct material characteristics and because the main objective was to verify the skid resistance provided by the field-blended rubber asphalt mixture.

### *6.4.2. Accelerated Pavement Tests*

Full-scale accelerated pavement tests (f-sAPT) were conducted with a heavy vehicle simulator in order to predict future pavement performance and evaluate the gap-graded field-blended rubber asphalt used for the overlay. Heavy vehicle simulators can be used for evaluating new road concepts and maintenance strategies and provide the option of performing a life cycle cost analysis, which, in turn, can decrease both costs and environmental impacts (Saevarsdottir *et al.,* 2014).

Simulation with f-sAPT was chosen so that long-term performance could be analyzed in a shorter period. According to Metcalf (1996), f-sAPT can be defined as “the controlled application of a prototype wheel loading, at or above the appropriate legal load limit to a prototype or actual, layered, structural pavement system to determine pavement response and performance under a controlled, accelerated, accumulation of damage in a compressed time period.”

A representative test section was chosen for f-sAPT using a full scale, mobile traffic simulator (FSMTS) from Simular Tecnologia Do Pavimento Ltda. The FSMTS was positioned at the northbound 35.035 kilometer mark. The f-sAPT simulation was conducted according to the procedures proposed by Vale (2008) and later complemented by Negrão (2012). The FSMTS used in the present study is shown in Figure 6.3.

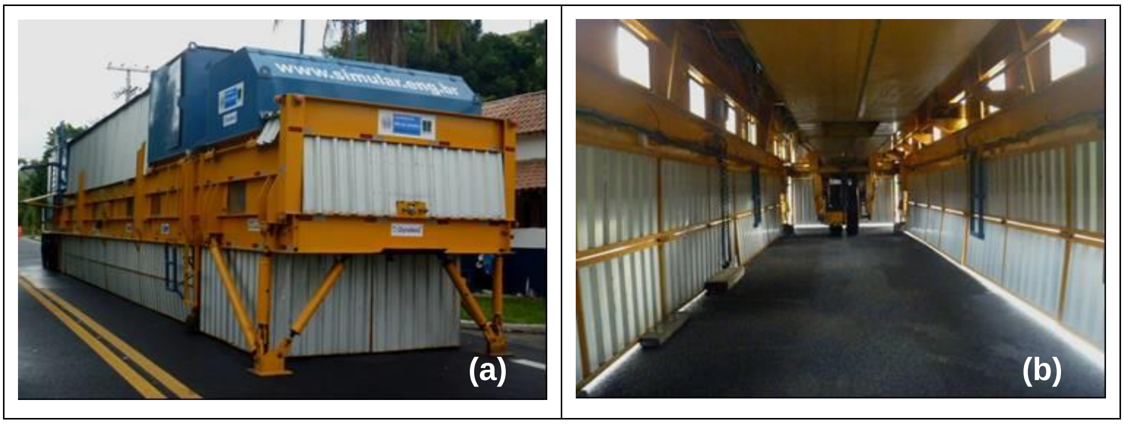



**Figure 6.3.** Full scale, mobile traffic simulator used in RJ-122 (a) outside view and (b) inside view

The FSMTS has a total length of 25 meters and width of 3 meters, weighing approximately 50,000 kg. The equipment consists of a dual-wheel half-axle that can apply loads up to 100 kN, which corresponds to an equivalent single axle load of 200 kN. The half-axle moves along an axis either directional (8,000 cycles/day) or bidirectional (16,000 cycles/day) with longitudinal displacements of up to 8.6 meters and transversal displacements of up to 1 meter. In addition, the FSMTS is equipped with sensors to automatically record air humidity, ambient temperature (inside the FSMTS and outside), pavement temperature, load applied, and number of passages.

After the FSMTS was installed at the site, the f-sAPT section was delimited on the pavement using spray paint. The test section had 7.0 meters in length and was 0.70 meters wide. The section was divided into 7 sub-sections for monitoring, each with an area of 0.70 $m^2$. Tests were conducted using a bidirectional 78.1 kN load, which is approximately 45% higher than the allowable load of 53.8 kN for this type of axle according to Brazilian legislation at the time.

The first pavement monitoring was done before the traffic simulation to verify the starting conditions. Tests included deflection testing, visual inspection for surface distress, rutting, and skid resistance. Climatic conditions were also monitored in terms of temperature, humidity, and rainfall. Every information collected was associated to the accumulated number of passages. FWD impact load tests were performed before and after the f-sAPT at a 0.4-meter interval. During the f-sAPT tests, a Benkelman beam was used to monitor change in deflection daily at each sub-section because the FWD could not be used due to operation restrictions. Surface distresses were monitored by visual inspection and, in case of any cracks, spray paint was used.

For determining pavement rutting, DNIT 006/2003 – PRO requires the use of a rut-depth measurement device that uses a similar concept to that of a straightedge. An adapted rut-depth measurement device (Figure 6.4) was fabricated in order to measure depth at every 0.1 m and determine the maximum rut-depth for each sub-section.


**Figure 6.4** Adapted rut-depth measurement device used for determining pavement rutting

Continuous measurement of skid resistance was not possible during f-sAPT and a British Pendulum was used instead. Tests were conducted in a wet condition and results are presented in terms of a British Pendulum Number (BPN). A summary of f-sAPT pavement monitoring tests is shown in Table 6.6 along with the desirable limits specified by the DER-RJ.

**Table 6.6 -** f-sAPT pavement monitoring test summary

| Parameter | Type | Equipment | DER-RJ Limits |
|---|---|---|---|
| Deflection | Structural | FWD/benkelman beam | < 0.65 mm |
| Cracking | Functional | Visual Inspection | < 20 % |
| Rutting | Functional | rut-depth measurement device | < 7 mm |
| Skid Resistance | Functional | British Pendulum | BPN > 47 |

Pavement monitoring was conducted for each sub-section at least at a daily basis and at cycles 0, 1,000, 3,000, 10,000, 30,000, and 100,000 in case premature distresses occurred. Daily tests were always conducted at the same time of the day and a technician monitored f-sAPT at all times to record any relevant incidents. Total rainfall was monitored using a pluviometer situated next to the traffic simulator.

### *6.4.3. Calibration of the HDM-4 Performance Models*

Results of the f-sAPT simulation done with a full scale, mobile traffic simulator were used in order to calibrate the HDM-4 performance models in terms of cracking and rutting. These parameters, along with pavement deflections and roughness, are the most commonly used parameters in Brazil to verify pavement performance. The HDM-4 pavement performance models have mechanistic-empirical concepts that allow for determining the annual progression of technical parameters, such as the international roughness index (IRI), cracking area, rutting, and the structural conditions expressed in terms of the modified structural number (SNC). The technical relationships available in the Highway Design and Maintenance Standards (HDM-III) model (Watanatada *et al*., 1987) were updated and calibrated in an international effort undertaken by The World Road Association (PIARC) in the 1998 International Study of Highway Development and Management (Kerali, 2000), which resulted in the HDM-4 tool.

Yearly cracking and rutting were obtained from the f-sAPT tests along with the approximated annual number of ESALS for each pavement monitoring survey. Thus, the HDM-4 tool was used to calibrate the cracking and rutting coefficients in respect to the observed yearly progression of cracking and rutting by trial and error. The rut depth in HDM-4 at any time is the sum of four components: initial densification ($k_{rid}$), structural deformation ($k_{rst}$), plastic deformation ($k_{rpd}$), and surface wear ($K_{rdw}$) (Odoki & Kerali, 2000). The present study consisted of calibrating the plastic and structural deformation coefficients, which indicate the deformations associated to the surface layer and the pavement as a whole, respectively. Initial densification only applies to new construction or reconstruction, whereas surface wear relates to the use of studded tires, which is not the case. The equation for predicting structural deformation (Eq. 6.1) without cracking is as following:

$$\Delta RDST_{UC} = K_{rst}\,(a_0\, SNP^{a_1}\, YE4^{a_2}\, COMP^{a_3}) \qquad \text{(eq. 6.1)}$$

where $\Delta RDST_{UC}$ is the incremental rutting increase in structural deformation without cracking in the analysis year in mm, $K_{rst}$ is the calibration factor for structural deformation, YE4 is the annual equivalent standard axles in millions/lane, SNP is the average annual adjusted structural number of the pavement, COMP is the relative compaction in percentage, and $a_i$ are coefficient values that depend on pavement type. For asphalt mix on asphalt pavement (AMAP), the coefficients are null. The equation (Eq. 6.2) for predicting plastic deformation is shown next:

$$\Delta RDPD = K_{rpd}\, CDS^3\, a_0 Y\, E4\, Sh^{a_1}\, HS^{a_2} \qquad \text{(eq. 6.2)}$$

Where $\Delta RDPD$ is the incremental increase in plastic deformation in the analysis year in mm, CDS is the construction defects index for bituminous surfacings, Sh is the speed of heavy vehicles in km/h, HS is the total thickness of bituminous surfacing in mm, $a_0$ is 2.46, $a_1$ is -0.78, and $a_2$ is 0.71. For permanent deformation modelling, the $K_{rpd}$ and $K_{rst}$ coefficients were varied from 0 to 10 at 0.5 intervals and root mean square error (RMSE) was used to choose the best combination that fit the measured data. The coefficients were then varied at smaller intervals to get a best fit.

Because no cracking was observed, the calibration of cracking initiation ($K_{cia}$) and all structural cracking progression ($K_{cpa}$) coefficients was not possible. However, cracking initiation was conservatively estimated as the number of ESALS obtained in f-sAPT simulation. Initiation of all structural cracking is defined by Eq. 6.3 for asphalt overlays:

$$ICA = K_{CIA}\left\{CDS^2\left[MAX\left(a_0 \exp\left[a_1\, SNP + a_2\left(\frac{YE4}{SNP^2}\right)\right] * MAX\left(1 - \frac{PCRW}{a_2}, 0\right), a_4 HSNEW\right)\right] + CRT\right\} \qquad \text{(eq. 6.3)}$$

where ICA is the time to initiation of all structural cracks in years, $K_{cia}$ is the calibration factor for initiation of all structural cracking, CDS is the construction defects indicator for bituminous surfacings, PCRW is the area of wide cracking before latest reseal or overlay in % of total carriageway area, HSNEW is the thickness of most recent surfacing in mm, CRT is crack retardation time due to maintenance in years, $a_0$ is 4.21, $a_1$ is 0.14, $a_2$ is -17.1, $a_3$ is 30, and $a_4$ is 0.025.

## 6.5. Results and Analyses

### *6.5.1. Field Monitoring*

#### *6.5.1.1. Structural Conditions*

Pavement deflection under the impact load is a measure of the structural capacity of the existing pavement as a whole, including the subgrade, sub-base, base, and surface layers. The deflections for the test section in RJ-122 are shown in Figure 6.5 for the four surveys conducted. Deflections were temperature corrected to 25°C using the methodology proposed by the São Paulo Department of Transportation (DER/SP, 2006) and normalized to a standard load of 41kN for comparison.

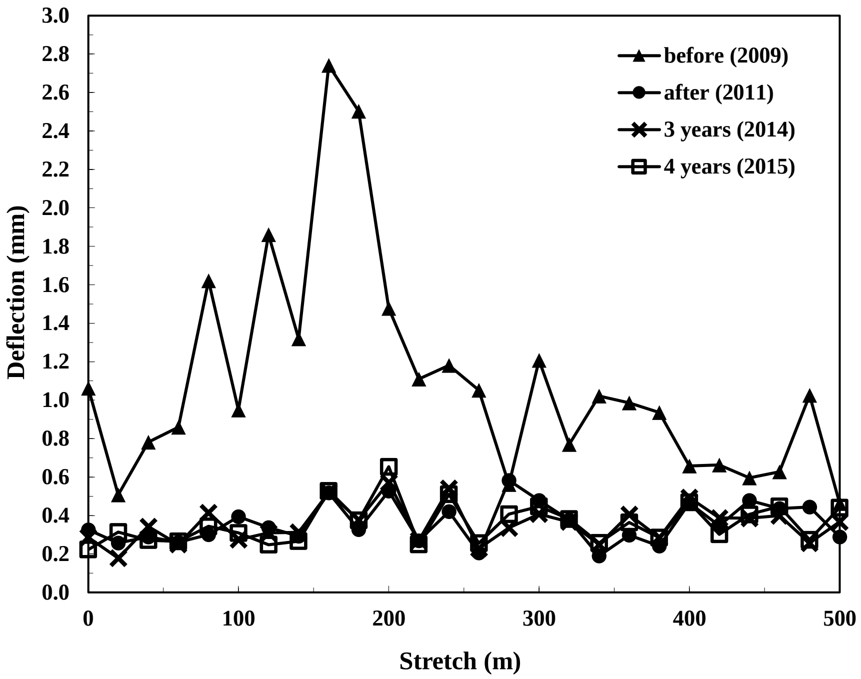



**Figure 6.5.** Deflections under the impact load for the test section in RJ-122

The deflections before the rehabilitation job were very high, ranging from 0.462 mm to as high as 2.742 mm. In addition, the deflections along the test section show a high variability, with higher values for the first 250 meters of the test section. The average deflection before the job was 1.098 ± 0.565 mm, which resulted in a coefficient of variation of 51.5%. Hence, as mentioned before, structural conditions were such that a conventional design pointed to a total reconstruction of the pavement.

The deflections after the rehabilitation job were reduced, on average, by 67%. The mean deflection was 0.360 ± 0.106 mm, with a coefficient of variation of 29.4%. Thus, the field-blended asphalt overlay resulted in a considerable reduction of the deflections under the impact load and a better homogeneity of the structural capacity of the pavement expressed by a 43% reduction of the coefficient of variation. The average deflections under the impact load are summarized in Table 6.7.

**Table 6.7 -** Average pavement deflections for all deflection tests

| Parameter | Before | After | 3 Years | 4 years |
|---|---|---|---|---|
| Minimum | 0.462 | 0.190 | 0.178 | 0.223 |
| Maximum | 2.742 | 0.583 | 0.572 | 0.653 |
| Average | 1.098 | 0.360 | 0.354 | 0.358 |
| Standard deviation | 0.565 | 0.106 | 0.100 | 0.107 |
| Coefficient of variation | 51.5% | 29.4% | 28.1% | 30.0% |

Deflections after 3 and 4 years of service life appear to have been maintained with slight variations. Thus, an analysis of variance (Anova) test was used to assess if deflections under the impact load are statistically maintained over the 4-year span. In this analysis, the probability of falsely rejecting the null hypothesis (averages are statically the same), referred to as the p-value, is determined and compared to the significance level, $\alpha$. A p-value lower than $\alpha$ would indicate that the mean deflections are different amongst them. A significance level of $\alpha = 0.05$, commonly used in hypothesis testing (Berthouex & Brown, 2002), was used.

The p-value obtained in this analysis was P = 0.983. Thus, we cannot reject the null hypothesis and conclude that the pavement deflections have been maintained over time for a 95% confidence level.

### *6.5.1.2. Functional Characteristics*

The skid resistance of the field-blended rubber asphalt mixture was GN = 0.653 ± 0.092 after approximately 2 years of service life. Thus, the average GN obtained for the test section in RJ-122 met the specified requirements (GN> 0.52), which results in a pavement with adequate skid resistance.

Surface distresses were identified by means of a visual inspection before and after the rehabilitation job in accordance with Brazilian specifications. The test section before the rehabilitation job exhibited crocodile cracking, mostly with erosion, in 100% of the survey stations. In addition, 61% of the stations showed permanent deformation and 52% contained bleeding. According to the Brazilian pavement classification index (DNIT, 2003), the pavements were characterized by a very poor surface condition. Surface conditions indicated the need for total reconstruction, which is consonant with the poor structural conditions observed. No distresses were observed after the rehabilitation job.

Also, in terms of pavement distresses determined through the Pavement Scanner, no distresses were observed after 4 years of service life, except for a transversal crack of approximately 1 meter in length was detected at the 34.965 kilometer mark after 3 years. It is important to note that the crack appeared in the region were deflections were the highest before and after the job. Therefore, the cracking may have occurred due to the underlying pavement conditions.

The average maximum rut-depths in the test section after 3 and 4 years of service life are shown in Figure 6.6 for each direction of traffic. The average maximum rut-depth after 3 years of service life was 3.0 ± 0.8 mm and 2.2 ± 1.0 mm in the northbound and southbound directions, respectively. After another year of service life (4 years total), an 8.5% increase (3.2 ± 0.8 mm) was shown for the northbound direction, whereas the increase was 17.4% (2.6 ± 1.0 mm) for the southbound direction. Trucks are typically more loaded in the northbound direction, which accounts for the higher rut-depth observed in comparison to the southbound direction. Also, the observed rut-depths after 4 years of service life are well below the limits (7 mm) used for toll roads in Brazil.

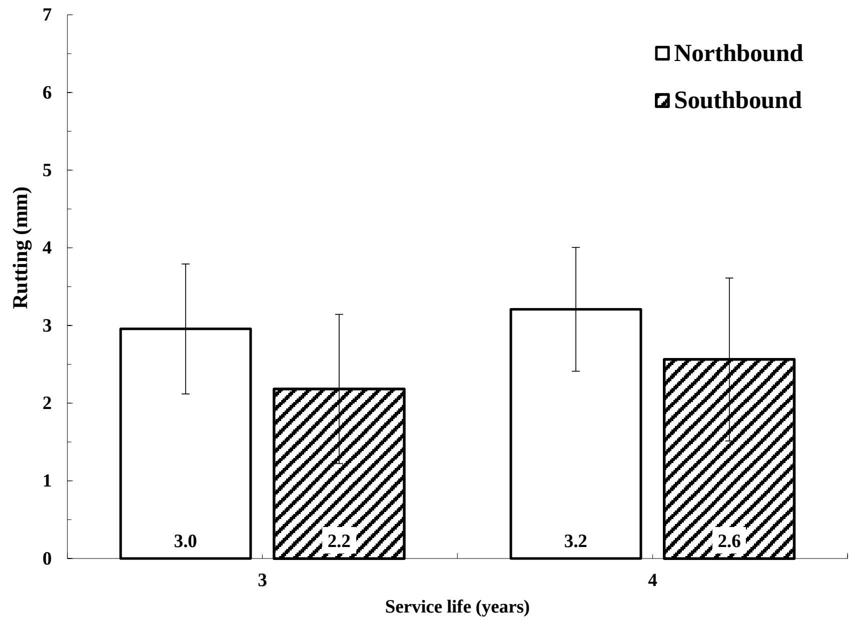



**Figure 6.6.** Maximum rut-depth after 3 and 4 years of service life for each direction of traffic in RJ-122

Pavement roughness in terms of IRI for the test section in RJ-122 is shown in Figure 6.7 for the four surveys conducted. The average IRI for the test section before the job was 4.21 m/km, which is considered as bad riding and safety conditions for the roadway users according to the Brazilian specifications (DNIT, 2006b). The rehabilitation job resulted in a 59.6% decrease of pavement roughness (from 4.21 to 1.70 m/km), bringing the riding conditions to good conditions. Furthermore, a slight increase in roughness was observed after 3 and 4 years of service life, but conditions are still considered good (1.76 and 1.80 m/km, respectively). The coefficient of variation also increased with time, from 19% after the job to 21% and 29% after 3 and 4 years of service life, respectively.

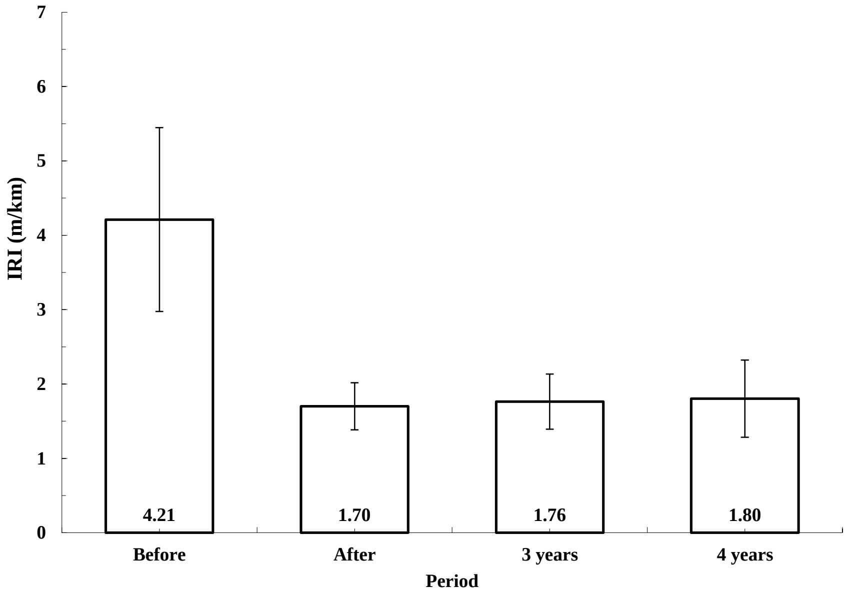



**Figure 6.7.** IRI for the test section in RJ-122

Negrão (2012) conducted a similar study using a gap-graded mixture in accordance with CALTRANS gradation specifications and meeting Brazilian specifications for an AB8 type rubber asphalt. A 300-m test section was built in July of 2010 on Highway BR-376/PR between the 672.0 and 671.3 kilometer marks using the gap-graded mixture with an asphalt content of 6.0% of a terminal blend rubber asphalt. The existing pavement was overlaid with approximately 55 mm of the asphalt mixture and periodic monitoring was conducted for up to 6 months of the job in order to determine the structural and functional characteristics of the pavement.

Periodic monitoring showed an accelerated deterioration of the pavement overlaid with the terminal-blended rubber asphalt mixture. Deflections increased from approximately 0.37 mm to 0.55 mm within 6 months of the job. Similarly, the IRI was increased from approximately 2.75 m/km to 3.60 m/km. The pavement showed even poorer conditions in terms of permanent deformation and cracking. The permanent deformation showed an approximate 4-fold increase after 6 months (14 mm after 6 months), whereas a cracked area was around 18% for the test section. However, highway BR-376/PR was subjected to more traffic in the field due to a higher ADT than RJ-122.

### *6.5.2. Accelerated Pavement Tests*

Accelerated pavement testing was conducted in April of 2012 and terminated after 17 days of continuous testing, with 150,000 load applications. The average load applied was 78.1 kN (156.2 kN on a single axle), which yields 2.47 million equivalent single axle loads (ESALS) according to the American Association of State and Highway Transportation Officials (AASHTO). The expected number of ESALS for the 20-year design project was 7.02 million. Thus, the f-sAPT simulated approximately 7 years of traffic, which is the typical time in Brazil for overlaying existing pavements (7 to 8 years).

During f-sAPT simulation, the average pavement temperatures inside the FSMTS was 28.4 ± 4.3°C, whereas minimum and maximum pavement temperatures were 20.9°C and 40.6°C, respectively. In addition, average air temperature inside was 27.9 ± 3.8°C, whereas minimum and maximum temperatures were 17.0°C and 38.5°C, respectively. A total precipitation of 140 mm was also observed. The observed temperatures are within the expected range for the region, which varies from 9°C to 35°C. On the other hand, the observed temperatures are higher than the annual average of 23.1ºC (Bochner, 2010). The total precipitation observed was approximately 30% higher than the average precipitation of 107.6 mm (INMET, 2015).

A summary of f-sAPT simulation is shown in Table 6.8 for the 150,000 cycles applied in the aforementioned RJ-122 test section. An average reduction of 5.7% (51.3 to 48.4) of skid resistance was observed in terms of BPN. The final BPN, however, is still above the minimum requirement of 47.

**Table 6.8 -** f-sAPT simulation results

| Parameter | DER-RJ Limits | Initial | Final |
|---|---|---|---|
| Deflection, mm | < 0.640 | 0.237 ± 0.123 | 0.329 ± 0.176 |
| Cracking, % | < 15 | 0 | 0 |
| Permanent Deformation, mm | < 7 | 0 | 4.2 ± 1.2 |
| Skid Resistance, BPN | > 47 | 51.3 ± 1.7 | 48.4 ± 1.4 |

No cracking was observed during f-sAPT simulation, which indicates that fatigue life was not reached during the 7 years of traffic simulated. These results are in agreement with results obtained in periodic field monitoring campaigns, since only a transversal crack of approximately 1 meter in length was observed after 4 years of service life of the 500-m test section.

The results show a better resistance to cracking under f-sAPT simulation than other tests conducted in Brazil with terminal-blended rubber asphalt mixtures. At BR-116, in São Paulo, the test section with 5.5% asphalt content reached its service life at a number of ESALS 30% lower than the one simulated at RJ-122. Similarly, a test section with 6.0 % asphalt content at BR-116, in Rio de Janeiro, exhibited 40% cracking of the testing area at a number of ESALS 50% lower than the one simulated at RJ-122 (Vale, 2008).

In terms of permanent deformation, an average maximum rut depth of 4.2 ± 1.2 mm was achieved at the end of f-sAPT. Even though permanent deformation was observed, the limit of 7 mm used in toll roads in Brazil was not achieved during the 7-year simulation. When comparing the permanent deformations obtained in f-sAPT with the ones for field monitoring, some differences were observed. The average maximum rut depth under the test section at 3 and 4 years of simulation were 2.3 and 3.9 mm, respectively, whereas the average maximum rut depth for the field observations in the northbound direction for the same period were 3.0 and 3.2 mm.

The observed permanent deformations in f-sAPT were approximately 20% below the field permanent deformation after 3 years and approximately 20% higher after 4 years. This difference may have occurred due to several reasons, such as the different methodologies for determining the rut-depth (manual vs. LCMS), higher load used in f-sAPT simulation, different scales, and different load wander characteristics between the real traffic and f-sAPT. However, simulated rut-depths were conservatively estimated and in the same range as the real observed rut-depths for the 4 years of service life.

Finally, the deflection under the impact load measured with the FWD exhibited, on average, a 1.39 fold increase (from 0.249 mm to 0.329 mm). This increase in deflection was not observed in the field monitoring surveys done in the 500-meter test section. In addition, the observed precipitation was not sufficient to have such an impact on the deflections, especially taking into account that a drainage system had been recently installed during the job.

Therefore, a backcalculation analysis of a representative deflection bowl using the ELSYM5 software before and after simulation was performed to determine the resilient moduli and identify the possible causes (Figure 6.8). The overlay layers (rubber asphalt and levelling course) were grouped in a layer, the existing asphalt layers (HMA and macadam) were grouped in the second layer, and the base/sub-base layers were combined in the third layer for this analysis.

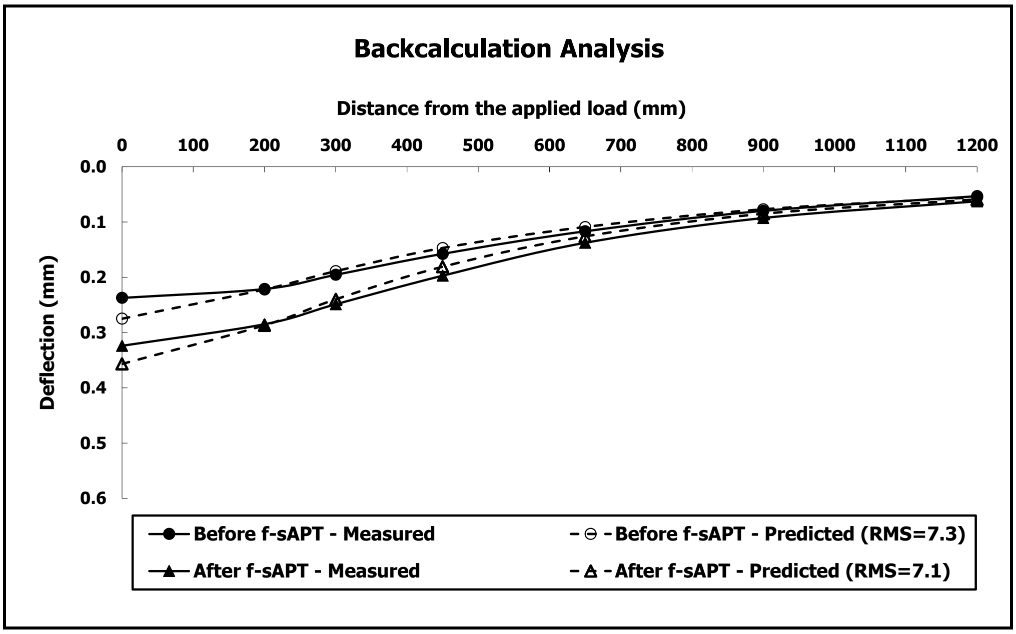



**Figure 6.8.** Backcalculation analysis of the representative deflection bowls before and after f-sAPT simulation

The resilient moduli for the new overlay ($E_1$), the existing asphalt layers ($E_2$), the base/sub-base soils ($E_3$) and the subgrade ($E_4$) are summarized in Table 6.9. The resilient moduli for the overlay materials remained constant in both backcalculation analyses. However, a decrease in resilient moduli for the underlying materials was observed, especially for the existing asphalt layers (58.3% decrease). The base/sub-base layers and the subgrade exhibited a slight variation in resilient moduli (9.1% and 11.1% reduction, respectively), probably due to the precipitation observed during the simulation.

**Table 6.9 -** Summary of the resilient moduli before and after f-sAPT simulation

| Condition | Backcalculated Moduli (MPa) | | | | RMSE |
|---|---|---|---|---|---|
| | $E_1$ | $E_2$ | $E_3$ | $E_4$ | |
| Before f-sAPT | 6000 | 600 | 110 | 180 | 7.3 |
| After f-sAPT | 6000 | 250 | 100 | 160 | 7.1 |

where $D_0$ is the deflection under the impact load, $E_i$ is the backcalculated modulus for layer "i" and RMSE is root mean square error.

Therefore, the increase in deflection observed was probably due to the loss of support exhibited by the existing pavement layers, especially the asphalt layers, which were in very poor conditions before the rehabilitation job. The applied load (45% above regulations) may have had a greater impact on pavement performance during f-sAPT, causing the underlying asphalt layers to collapse.

Negrão (2012) also conducted f-sAPT on the pavement overlaid with the terminal-blended asphalt mixture using the FSMTS. The FSMTS was used to simulate 2.22 million ESALS using the AASHTO load equivalency factors and a 7-m test section under the simulator was monitored. Deflections in the test section remained constant at roughly 0.50 mm during simulation and no cracking was observed. Average rutting under the test section increased from 0.8 mm to 5.4 mm.

Even though periodic monitoring of the field-blended and terminal-blended rubber asphalt mixtures showed different pavement performance, test sections in f-sAPT showed similar results in terms of permanent deformation and cracked area. The number of ESALS in both studies was approximately the same (the present study simulated approximately 10% more ESALS than the previous study). No cracking was observed under the f-sAPT test section for both the field and terminal-blended mixtures after simulation. The average increase in rutting exhibited by the terminal-blended mixture (4.6 mm) was approximately 10% higher than the field-blended mixture (4.2 mm), even though the field-blended mixture was subjected to more traffic. The terminal-blended mixture, however, exhibited rutting readings under the test section over the 7 mm limit for Brazilian concessionaries.

### 6.5.3. Calibration of the HDM-4 Performance Models

The f-sAPT data used for calibrating the HDM-4 performance models are summarized in Table 6.10. The respective cracking area and average maximum rutting under the f-sAPT test section were associated with the approximated annual equivalent axle loads (ESALS). This data was first input into the HDM-4 software and the calibration constants were varied in order to obtain the smallest root mean square error (RMSE).

**Table 6.10 -** Data used for calibrating the HDM-4 performance models

| Cycles | ESALS | Year | Average Maximum Rutting, mm | Cracking, % |
|---|---|---|---|---|
| 0 | 0.00E+00 | 0 | 0.0 ± 0.0 | 0 |
| 14,573 | 2.40E+05 | 1 | 1.5 ± 2.2 | 0 |
| 35,621 | 5.86E+05 | 2 | 2.1 ± 2.5 | 0 |
| 60,577 | 9.97E+05 | 3 | 2.3 ± 2.6 | 0 |
| 81,392 | 1.34E+06 | 4 | 3.9 ± 1.5 | 0 |
| 105,185 | 1.73E+06 | 5 | 4.1 ± 1.3 | 0 |
| 118,166 | 1.94E+06 | 6 | 4.1 ± 1.3 | 0 |
| 150,000 | 2.47E+06 | 7 | 4.2 ± 1.2 | 0 |

The cracking initiation coefficient ($K_{cia}$) was conservatively determined as 2.78 for the present study. Thus, the test section resulted in a cracking initiation coefficient 2.78 times higher than the standard value of 1.0, which shows a higher resistance to cracking initiation for the test section in RJ-122. In relation to permanent deformation modelling, a combination of a $K_{rpd}$ of 3.0 and $K_{rst}$ of 3.0 resulted in the lowest error (RMSE = 0.43). The final calibrated permanent deformation model is shown in Figure 6.9 along with the f-sAPT observed values. The calibrated HDM model resulted in a $R^2$ = 0.864 (p-value = 0.03).

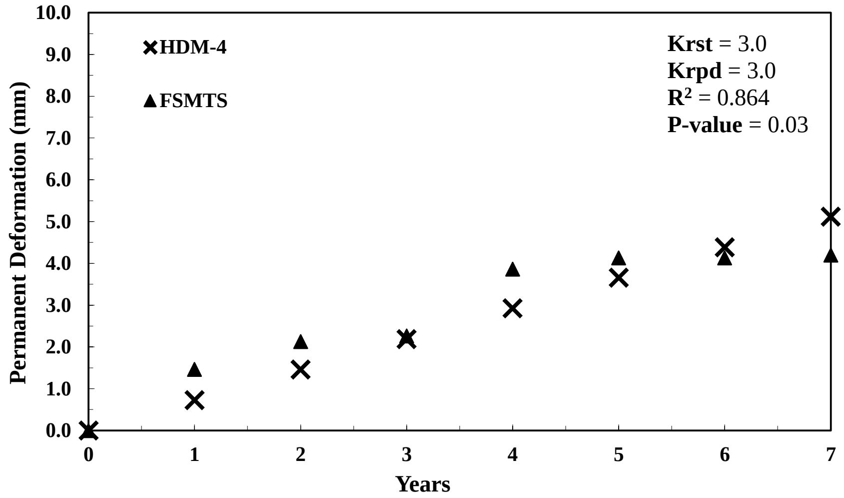



**Figure 6.9.** Calibration of the HDM rutting coefficients ($K_{rpd}$ and $K_{rst}$)

As mentioned earlier, Negrão (2012) obtained higher permanent deformation for a terminal-blended gap-graded mixture under f-sAPT simulation for slightly smaller ESALS. However, different calibrations coefficients were obtained ($K_{rpd}$ of 6.0 and $K_{rst}$ of 1.5) than the ones obtained in this study. The final calibrated HDM-4 performance models for both studies are shown in Figure 6.10 for comparison. The field-blended mixture results in smaller permanent deformation over time, which will yield less maintenance costs.

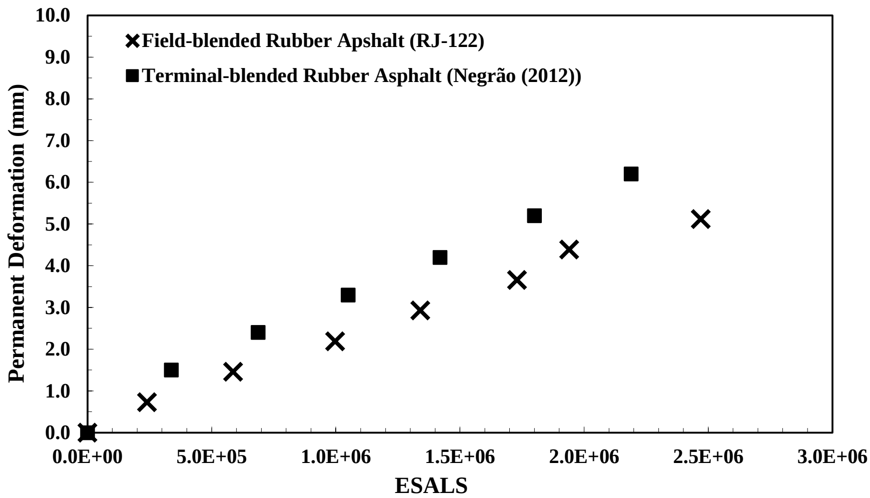



**Figure 6.10.** Calibrated HDM rutting models for the field-blended and a terminal-blended rubber asphalt mixtures

## 6.6. Conclusions

This case history study presented the results of the field performance and performance prediction of the first pavement rehabilitation job in Brazil using a field-blended rubber asphalt mixture. A 500-meter test section with a gap-graded field-blended rubber asphalt surface courses was periodically monitored through non-destructive tests and f-sAPT simulation was conducted in a representative test section. Results from the f-sAPT simulation were then used to calibrate the performance models in HDM-4. The following conclusions can be drawn:

- The asphalt overlay using a gap-graded field-blended rubber asphalt applied in RJ-122 reduced the deflections under the impact load from 1.098 ± 0.565 mm to 0.360 ± 0.106 mm (67% reduction) and Anova analysis showed that deflections remained constant after 4 years of service life ($P = 0.983$).
- Apart from a localized, transversal crack measuring 1 m in length observed after 3 years of service life in a region where deflections were the highest, no surface distresses were observed in the 500-meter test section.
- Rut-depths after 4 years of service were well below the limits (7 mm) used for toll roads in Brazils (3.2 ± 0.8 mm and 2.6 ± 1.0 mm north and southbound, respectively).
- The skid resistance of the field-blended rubber asphalt mixture was GN = 0.653 ± 0.092 after approximately 2 years of service life, which results in a pavement with adequate skid resistance.
- In terms of pavement roughness, good riding conditions were obtained after the rehabilitation job (1.70 m/km) after a 60% reduction in comparison with the poor conditions before. Furthermore, a slight increase in roughness was observed after 3 and 4 years of service life, but conditions were still considered good (1.76 and 1.80 m/km, respectively).
- Accelerated pavement tests using a full scale, mobile traffic simulator simulated approximately 7 years of service life (2.47 million ESALS using the AASHTO load factors) by applying 150,000 cycles of a 78.1 kN (156.2 kN on a single axle) load.
- The test section exhibited promising performance under traffic simulation since no cracking was observed, the average maximum rut depth was below limits

(4.2 ± 1.2 mm), and skid resistance reduced 5.7% (51.3 to 48.4), but was still above the desirable value (BPN = 47).

- A 1.39 fold increase (from 0.249 mm to 0.329 mm) was observed for the deflection under the impact load measured with the FWD before and after traffic simulation and a backcalculation analysis showed that the increase was due to a reduction in resilient moduli of the existing pavement layers.
- The calibration of the HDM-4 models resulted in a cracking initiation coefficient ($K_{cia}$) of 2.78, conservatively estimated since no cracking was observed in f-sAPT. In addition, calibration of the permanent deformation models resulted in a $K_{rpd}$ of 3.0 and $Kr_{st}$ of 3.0 ($R^2$ = 0.864).

Thus, the results of this case history study have shown that the field-blended rubber asphalt in a gap-graded mixture used in RJ-122 is a promising alternative for pavement construction and rehabilitation in Brazil. When compared to similar studies in Brazil using a terminal-blended rubber asphalt mixture in a gap-graded gradation, the field-blended mixture exhibited equal or better performance, which is an indicative that this type of mixture is adequate for use in Brazilian conditions. However, further research is required to better compare these two types of mixtures directly (under the same conditions).

# 7. CONCLUSIONS

## 7.1. Summary of Findings

This investigation dealt with the study of the first Brazilian experience with a field-blended rubber asphalt mixture in a gap-graded gradation used in the RJ-122 rehabilitation project, which is a heavy traffic highway in the state of Rio de Janeiro. The objective was to evaluate the performance of surface courses using field-blended rubber asphalt mixtures by means of rheological binder testing, laboratory permanent deformation and fatigue tests, accelerated pavement tests in the field using a large scale, mobile traffic simulator, and periodic pavement monitoring campaigns of an in-service test section in order to study and predict performance.

The field-blended rubber asphalt binder was first compared to typical binders used in Brazil: a neat binder of penetration grade 30/45 (AC 30/45) and a binder modified with an elastomeric polymer (SBS) that is typically used in Brazil for improving binder fatigue and permanent deformation resistance. It is important to note that there was a difference in binder aging procedure, since the rubber asphalt was aged in the RTFOT at 175°C for creating the film for proper aging. Permanent deformation was determined using the MSCR test, whereas fatigue behavior was determined using the Time Sweep and the Linear Amplitude Sweep (LAS) tests. The non-recoverable creep compliances ($J_{nr}$) for the modified binders were lower than the one exhibited by the neat binder for both stress levels (0.1 kPa and 3.2 kPa) at any given temperature, which shows that the modification of the neat binder with either rubber or polymer results in a binder with a better resistance to rutting. However, the $J_{nr,3200}$ for the rubber asphalt were significantly lower (from approximately 38 to 55% lower, depending on the temperature) than the ones for the SBS binder.

The neat and SBS binders showed adequate sensitivity to stress in all temperatures tested, since the $J_{nr,\ diff}$ sensitivity parameter was lower than the 75% limit. The rubber asphalt binder, however, exhibited $J_{nr,\ diff}$ over the 75% limit for all temperatures tested. Even though the binder showed a stress dependent behavior, the $J_{nr}$ values were very small (ranging from 0.05 to 0.34 1/kPa for the higher stress level), indicating a small rutting potential in the field.

Binder modification resulted in a better fatigue behavior than the neat binder according to the time sweep and linear amplitude sweep tests. In addition, rubber modification resulted in the best fatigue behavior among the binders tested. Rubber modification resulted in a binder with a different stress-strain behavior and the damage characteristic curves for the binders show that the rubber asphalt has a better response to damage than the neat and SBS binders (higher integrity for the same damage level). Fatigue life in the TST was consistently higher than fatigue life in the LAS test, probably due to the differences in failure criteria used for each test. However, ranking of the three binders remained the same. Results showed that the field-blended rubber asphalt has a better resistance to fatigue and permanent deformation than the neat and SBS binders.

The fatigue and permanent deformation of the field-blended rubber asphalt in a gap-graded mixture was then verified at different levels (binder, mixture, and field performance). Permanent deformation of the binders was determined through $|G^*|/\sin\delta$ and MSCR tests, which are the most current tests available, whereas the permanent deformation of the rubber asphalt mixture was evaluated by a LCPC wheel-track test. Field permanent deformation was evaluated using a LCMS and a full scale, mobile traffic simulator.

The rubber asphalt binder showed a higher $|G^*|/\sin\delta$ than the neat binder for all temperatures tested both before and after aging (46 to 82°C), which indicates that the rubber asphalt has a better resistance to rutting than the neat binder. The percent recovery for rubber asphalt determined in the MSCR test was higher than the recovery for the neat binder at any given temperature, which validates the presence of the rubber modifier. For any given temperature, the non-recoverable creep compliance ($J_{nr}$) for the asphalt rubber was lower than the one exhibited by the neat binder for both stress levels. This is an indicative that the rubber asphalt binder has a better resistance to rutting than the neat binder.

The stress sensitivity parameter ($J_{nr,\ diff}$) for the rubber asphalt binder was over the 75% limit for all temperatures tested, which indicates that the binder is stress dependent. Even though the binder showed a stress dependent behavior, the $J_{nr}$

values are very small (ranging from 0.08 to 0.53 1/kPa for the higher stress level), indicating that the mixture should have a small rutting potential in the field. The $|G^*|/\sin\delta$ parameter alone would have provided enough information to show that the rubber asphalt has a better rutting resistance than the neat binder.

The permanent deformation after 30,000 cycles of the LCPC wheel-track test for the rubber asphalt mixture collected after the RJ-122 rehabilitation job was 4.2%. The results are in accordance with French specifications and below the 5% limit for Brazilian conditions. The wheel track test results are an indicative that the rubber asphalt mixture in a gap-graded gradation has adequate permanent deformation resistance, which validates the results obtained in the MSCR tests. The LCPC wheel-track test results for the RJ-122 field specimens also showed adequate permanent deformation resistance for the mixture (<5%). However, the permanent deformation of the field specimens after 30,000 cycles (2.2%) were approximately 48% lower than the one for the laboratory specimens.

The observed rut-depths of the test section after 3 and 4 years of service life (3.0 and 3.2 mm, respectively) were well below the limit used for toll roads in Brazil (7 mm). The adequate resistance to rutting shown by the rubber asphalt mixture confirmed the results obtained in the MSCR and the wheel track tests. The average rut-depth observed in the f-sAPT test section was 4.2 mm for the 7 years of simulated traffic. The results obtained for the pavement monitoring and f-sAPT were similar, which is an indicative that traffic was successfully simulated. The permanent deformation observed in the field confirmed the results obtained both at the binder and at mixture level performed in the laboratory, even though the asphalt binder showed a stress dependent behavior in the MSCR test.

Fatigue behavior of the binders was determined through Time Sweep and Linear Amplitude Sweep (LAS) tests, which are the most current tests available, whereas fatigue behavior of the mixture was evaluated by four-point bending tests. Field cracking was evaluated using a LCMS and a full scale, mobile traffic simulator.

The fatigue behavior of the field-blended rubber asphalt determined with time sweep and the linear amplitude sweep tests exceeded the one for the neat binder. Rubber modification resulted in a different stress-strain behavior and the damage characteristic curves for the binders showed that the rubber asphalt has a better response to damage than the neat binder (higher integrity for the same damage level). Fatigue behavior of the neat binder and rubber asphalt may be estimated from strong relationships (fatigue laws) derived for the binder in the TST and LAS tests.

Flexural bending tests on prismatic specimens (four point bending) indicated a good resistance to fatigue for the field-blended rubber asphalt in a gap-graded mixture. Fatigue laws were developed for the field-blended rubber asphalt mixture (field and laboratory-compacted mixtures) and laboratory specimens showed a better fatigue behavior, possibly because the field mixture exhibited a higher stiffness than the laboratory mixture.

No cracking was observed during f-sAPT simulation (approximately 7 years of traffic for RJ-122), which indicates that fatigue life was not reached during the 7 years of traffic simulated. These results were in agreement with results obtained in periodic field monitoring campaigns, since only a transversal crack of approximately 1 meter in length was observed after 4 years of service life of the 500-m test section. Pavement performance in periodic pavement monitoring and full-scale accelerated pavement testing conducted in tests sections in RJ-122 confirmed the results obtained at the binder and mixture level.

Full scale, accelerated pavement testing was conducted on a test section at RJ-122 and results were used to calibrate the HDM-4 cracking and permanent deformation models. In addition, periodic pavement monitoring was conducted for 4 years in a 500-m test section in order to assess the pavement performance under real traffic conditions and validate the results obtained in the accelerated pavement tests.

Accelerated pavement tests using a full scale, mobile traffic simulator simulated approximately 7 years of service life using the AASHTO load factors by applying 150,000 cycles of a 78.1 kN (156.2 kN on a single axle) load. The test section exhibited promising performance under traffic simulation since no cracking was observe, the average maximum rut depth was below limits (4.2 ± 1.2 mm), and skid resistance reduced 5.7% (51.3 to 48.4), but was still above the desirable value (BPN = 47).

A 1.39 fold increase (from 0.249 mm to 0.329 mm) was observed for the deflection under the impact load measured with the FWD before and after traffic simulation and a backcalculation analysis showed that the increase was due to a reduction in resilient moduli of the existing pavement layers. The calibration of the HDM-4 models resulted in a cracking initiation coefficient ($K_{cia}$) of 2.78, conservatively estimated since no cracking was observed in f-sAPT. In addition, calibration of the permanent deformation models resulted in a $K_{rpd}$ of 3.0 and $K_{rst}$ of 3.0 ($R^2$ = 0.864).

In terms of pavement monitoring, the asphalt overlay using a gap-graded field-blended rubber asphalt applied in RJ-122 reduced the deflections under the impact load from 1.098 ± 0.565 mm to 0.360 ± 0.106 mm (67% reduction) and Anova analysis showed that deflections remained constant after 4 years of service life (P = 0.983). Apart from a localized, transversal crack measuring 1 m in length observed after 3 years of service life in a region where deflections were the highest, no surface distresses were observed in the 500-meter test section. Rut-depths after 4 years of service were well below the limits (7 mm) used for toll roads in Brazils (3.2 ± 0.8 mm and 2.6 ± 1.0 mm north and southbound, respectively).

The skid resistance of the field-blended rubber asphalt mixture was GN = 0.653 ± 0.092 after approximately 2 years of service life, which results in a pavement with adequate skid resistance. In terms of pavement roughness, good riding conditions were obtained after the rehabilitation job (1.70 m/km) after a 60% reduction in comparison with the poor conditions beforehand. Furthermore, a slight increase in roughness was observed after 3 and 4 years of service life, but conditions were still considered good (1.76 and 1.80 m/km, respectively).

## 7.2. Conclusions

The field-blended rubber asphalt binder exhibited an improved performance when compared to the neat asphalt (AC 30/45 penetration grade) and the SBS-modified binders. The results obtained in the MSCR test (permanent deformation) and the

TST and LAS tests (fatigue) showed that the rubber asphalt has a better resistance to permanent deformation and fatigue than the neat and SBS-modified binders. In general, the tests were effective in ranking the three binders in terms of their rheological characteristics.

The field-blended rubber asphalt binder in a gap-graded mixture also exhibited a good performance in the laboratory and in the field. The mixture showed an adequate resistance to permanent deformation in the LCPC wheel-track test both for laboratory-compacted specimens, as well as for specimens sawed from field slabs collected at highway RJ-122. Full-scale accelerated pavement tests conducted in a test section at RJ-122 confirmed the results obtained at the binder and mixture levels, since the section exhibited adequate resistance to permanent deformation. Periodic pavement monitoring conducted over 4 years of an in-service test section validated the accelerated pavement tests.

Similarly, the rubber asphalt mixture showed an adequate resistance to fatigue in the four-point bending tests both for laboratory-compacted specimens, as well as for specimens sawed from field slabs collected at highway RJ-122. Accelerated pavement tests also confirmed the results obtained for the binder and mixture levels and field performance through periodic pavement monitoring corroborated the results obtained in accelerated pavement tests. Finally, the results obtained for the accelerated pavement tests were used to calibrate the HDM-4 cracking and permanent deformation models.

In conclusion, the excellent results obtained in the RJ-122 rehabilitation job may be attributed to the enhanced characteristics of the field-blended rubber asphalt binder used in the gap-graded mixture. The binder exhibited better permanent deformation and fatigue behavior than typical binders used in Brazil and the good results obtained in the laboratory were confirmed in the field under local climate and traffic conditions. Thus, one can conclude that the field-blended rubber asphalt is a valid technique for rehabilitation and construction of pavements in Brazil.

## 7.3. Future Studies

The results presented in this study attempted to provide more knowledge regarding the field-blended rubber asphalt technology, used for improving binder characteristics, and assess performance under local climate and traffic conditions in Brazil. Even though the results show adequate resistance to permanent deformation and fatigue, future studies must be conducted for a more complete understanding of this technology.

The following are some suggestions for future studies regarding the field blending of rubber asphalt:

- Evaluate the influence of different suppliers of crumb rubber modifier, as well as the different CRM gradations available, in the performance of field-blended binders and mixtures.
- Evaluate mixture permanent deformation and fatigue for a variety of aggregate gradations used in Brazil (i.e. dense graded, which is the most commonly used)
- Assess the performance of field-blended rubber asphalt mixtures for varying traffic levels and compare results to typical mixtures used in Brazil for the same conditions.
- Compare field-blended rubber asphalt mixtures with typical mixtures used in Brazil under the same local conditions.
- Conduct a study to compare the performance of field-blended and terminal-blended rubber asphalt binders/mixtures.
- Collect long-term pavement performance information of pavements using field-blended rubber asphalt mixtures, provided there will be a more common use of this type of mixture.
- Conduct life cycle cost analysis to assess the feasibility of using this type of mixtures in construction jobs in Brazil.

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# APPENDIX A – FIELD BLENDING PROCESS AT RJ-122

The process for obtaining the crumb rubber modifier by means of the ambient grinding process at Ecobalbo is described next. The photographs used in this study were kindly lent by Dynatest Engenharia Ltda. The process begins by selecting the appropriate amount of tires to be utilized. The automobile (light vehicles) and truck tires (heavy vehicles) selected for ambient grinding are shown Figure A.1 (a) e Figure A.1 (b), respectively.


**Figure A.1.** Automobile (a) and truck (b) tires selected for ambient grinding at Ecobalbo for fabricating the crumb rubber modifier

The machine used to remove the tire beads before the grinding process is shown Figure A.2 (a). The waste steel removed from the tires is shown in Figure A.2 (b).


**Figure A.2.** Tire bead extractor (a) and waste tire steel (b)

The tire feeding line shown in Figure A.3 is used to send the tires after the beads have been extracted. In the present study, a ratio of 4 automobile tires to 1 truck tire was used to obtain the crumb rubber modifier.

**Figure A.3.** Tire feeding line for the ambient grinding process

The tires are fed to the first cracker mill, where the grinding process initiates (Figure A.4 (a)). The scrap tire obtained after the first milling process is shown in Figure A.4 (b). The shredded scrap tire are further processed in a second mill, shown in Figure A.4 (c). The scrap tire after undergoing two milling processes is shown in Figure A.4 (d).


**Figure A.4.** First mill in the grinding process (a), the resulting scrap tire after the first milling process (b), the second mill (c), and the resulting scrap tire after the second milling (d)

During the grinding process, both the waste nylon and steel are extracted from the scrap tires. The nylon is extracted by using a vaccuum while the scrap tires are suspended on the sieves (Figure A.5 (a)). The vacuum suction is controlled such that the scrap tires are not sucked in and fall through the sieves. The steel in the tires is extracted during the grinding process by using an Electromagnetic Steel Separator, as shown in Figure A.5 (b). The waste nylon and steel from the grinding process are shown in Figure A.5 (c) and (d), respectively.


**Figure A.5.** Vaccuum (a) and Electromagnet Steel Separator (b) used to extract the nylon (c) and steel (d), respectively, from scrap tires during the grinding process

The crumb rubber modifier obtained in the ambient grinding process at EcoBalbo for use at RJ-122 is shown in Figure A.6 (a). The crumb rubber modifier is then stored in 1-ton bigbags for later modification of the asphalt binder (Figure A.6 (b)).

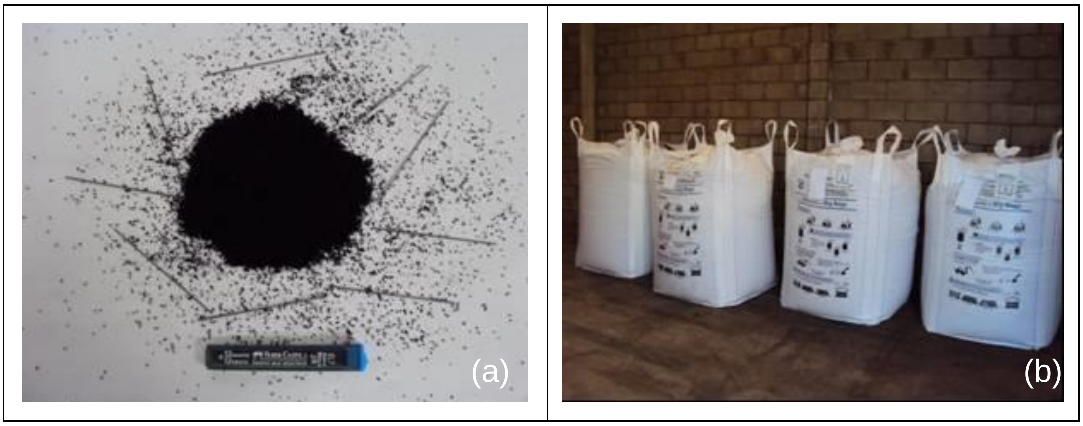


**Figure A.6.** Crumb rubber modifier from EcoBalbo for use at RJ-122 (a) and bigbags for storing 1 ton of CRM (b)

A full view of the combination blender (combo) from D&H Equipment, LTD, used in RJ-122 is shown in Figure A.7 (a). The combo has a specific silo for dumping the crumb rubber modifier from 1-ton bigbags for blending with the asphalt binder (Figure A.7 (b)). The front view of the combo is shown in Figure A.7 (c). The tank where the asphalt binder is kept is shown in the right, the jumbo for heating the asphalt binder is in the middle, and the control cabinet is in the left.


**Figure A.7.** Combination blender from D&H Equipment, LTD, used in RJ-122

Similarly, the side view of the combo is shown in Figure A.7 (d). In this view, the silo for adding the crumb rubber modifier is in the left, followed by the tank for mixing the rubber-asphalt blend, by the boiler for heating the blend, and by the control cabinet at the right. The pump for transferring the rubber-asphalt is shown in Figure A.7 (e) and the tap for collecting samples of the rubber-asphalt is shown in Figure A.7 (f).

The field application and compaction of the field-blended rubber asphalt mixture at RJ-122 is shown in Figure A.8 (a) and (b), respectively. It is important to note that the equipments for applying and compacting the mixture are the same as the ones for conventional asphalt mixtures, with the exception of the pneumatic tire roller. The rubber asphalt mixture adheres to the tires, so this type of roller should be avoided. In addition, the drum in the steel wheel rollers needs to be constantly lubricated with a water-neutral detergent mixture (or vegetable oil) mixture in order to prevent adhesion to the drum.

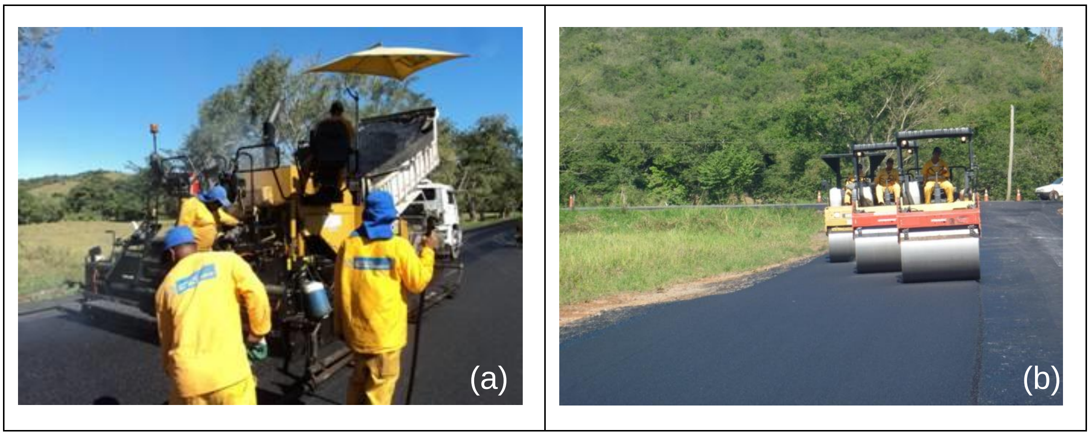


**Figure A.8.** Field application (a) and compaction (b) of the field-blended rubber asphalt mixture at RJ-122