ZILA MARIA GARCIA MASCARENHAS

Large Stone Asphalt Mixture as a rehabilitation strategy for heavy traffic highways

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RESUMO

O concreto asfáltico é comumente utilizado no Brasil como camada de rolamento em rodovias para garantir a segurança e o conforto dos usuários da via. No entanto, misturas asfálticas também são utilizadas como camadas intermediárias na estrutura do pavimento, atribuindo maior resistência às solicitações do tráfego. Assim, a ocorrência de defeitos pode ser reduzida, aumentando o intervalo entre as intervenções de manutenção do pavimento. *Large Stone Asphalt Mixture* (LSAM) é uma solução promissora para restauração de pavimentos asfálticos sob tráfego pesado que apresentem, como principal defeito, a deformação permanente em trilha de roda. Embora a sua utilização não seja recente, o estudo das propriedades viscoelásticas e do seu comportamento em campo ainda é pouco explorado. Esta dissertação apresenta uma breve revisão da literatura quanto ao histórico de uso das misturas LSAM ao redor do mundo, assim como características relevantes para a dosagem das mesmas. A avaliação de diferentes métodos de dosagem foi necessária antes da construção de um trecho experimental com a aplicação da LSAM como camada de ligação, onde tal mistura foi caracterizada mecanicamente por meio do ensaio de módulo dinâmico, ensaio de fadiga a flexão em quatro pontos, ensaio de fadiga a tração direta e ensaio de deformação permanente no simulador de tráfego francês LCPC. O pavimento restaurado foi monitorado por um período de um ano e meio com ensaios de “Falling Weight Deflectometer” e de afundamento de trilha de roda. As respostas do pavimento submetido a cargas dinâmicas foram analisadas por meio de um software que utiliza o método de camadas finitas (3D-Move), considerando as camadas asfálticas com comportamento viscoelástico linear sob diferentes velocidades de tráfego e temperaturas de análise. Os resultados poderão ser usados como referência para o dimensionamento e a análise das estruturas de pavimento compostas por camada de LSAM e para a avaliação do desempenho do mesmo. O uso da LSAM na restauração das seções experimentais foi bem sucedido quando comparado às restaurações realizadas anteriormente com outras soluções, não apresentando deformação permanente após mais de um ano de vida de serviço.

Palavras-Chave: Pavimentação Asfáltica (Reabilitação / Dosagem). Deformação. Fadiga dos materiais.

ABSTRACT

The asphalt mixtures are commonly used as surface course of highways to ensure the security and comfort for road users. The asphalt mixtures may also be used as intermediate layers of pavement structure, providing better resistance to traffic loads. Thus, the occurrence of distresses can be reduced, increasing the time between pavement maintenance interventions. Large Stone Asphalt Mixtures (LSAM) are a promising rehabilitation solution for pavements that show rutting as the main distress under heavy traffic situations. Though their use is not new, the study of their viscoelastic properties and performance in the field are still unexplored. This thesis presents a brief literature review about the historical use of LSAM around the world, as well as some relevant characteristics for its design. The evaluation of different mixture design methods was performed before the construction of experimental test site with LSAM as the leveling course. The asphalt mixtures mechanical behavior was characterized using dynamic modulus test, flexural bending fatigue test, tension-compression fatigue test and the LCPC wheel track test. The rehabilitated pavement was monitored through a period of one year and a half, using the falling weight deflectometer and rutting measurements. A finite-layer software (3D-Move Analysis) was used to simulate the structures using dynamic analysis, under different temperatures and traffic speeds, considering the viscoelastic properties of the asphalt mixtures obtained in laboratory. The results from the computer simulations can be used as a guide for the design and analysis of the pavement structure and pavement performance. Thus far, the use of LSAM in the rehabilitated test sections was successful compared to the previous rehabilitation solutions tested in the field, showing no permanent deformation after one year of service life.

Keywords: Asphalt Pavement (Rehabilitation / Design). Deformation. Fatigue.

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1. INTRODUCTION

The road infrastructure is the main transportation mode used in Brazil to move passengers and economic assets, corresponding to 90% and 60%, respectively (CNT, 2017). It is visible the increase of the road traffic volume and the vehicle loads, due to the economic development and the construction of new roads that established the connection between commercial poles. This brings consequences to pavement life cycle that may deteriorate before the designed period.

According to CNT (2017), the vehicle volume increased 194,1% in fifteen years and 58,2% of the Brazilian pavements are evaluated with some functional or structural problem. The pavement integrity is affected by the increase of distresses, reducing its load capacity and road safety.

The main modes of distress observed in flexible pavements are the fatigue cracking, thermal cracking, moisture damage from stripping, and permanent deformation. For the last one, it’s highlighted the rutting in the asphalt concrete that is characterized as longitudinal depressions which are formed in the pavement’s wheel paths. The permanent deformation may develop by surface course densification by traffic after construction, or by shear rupture of the surface course underlying layers (BERNUCCI et al., 2010).

One big issue related to rutting is the aquaplaning, that is a phenomenon in which the vehicle tire slides on water film due to the loss of tire/pavement adherence. In addition, the presence of water on pavement surface weakens the pavement structure. The advances in studies of structural solutions that extend the pavement life and decreases interventions of roads infrastructure have its importance on the rehabilitation context. Asphalt concrete has been used in pavement structures as surface and also as base course. When properly designed, it can provide good resistance to traffic loads and reduce the occurrence of distresses, increasing the time between maintenance interventions.

The Large Stone Asphalt Mixture (LSAM) is a dense and continuous hot mix asphalt concrete composed by a high portion of large aggregates, with nominal maximum

aggregate size (NMAS) above 25 mm. LSAM is a solution that aims to resist to permanent deformation, especially in heavy traffic situations (HUGO et al., 1990), and its use as a structural layer in flexible pavements is also reported to be beneficial against reflective cracking (CAO et al., 2011).

Since the early 1900s, the interest in this material has been reported due to the success of many large stone pavements which had been in excellent service for over years (HUGO et al., 1990; NCHRP, 1997). However, studies of this type of asphalt mixture are little explored in terms of its mechanical behavior, and stresses distribution in the field.

It is believed that the LSAM mixtures can present both technical and economical advantages when subjected to very heavy traffic, especially by avoiding premature rutting. The National Cooperative Highway Research Program (NCHRP) indicates the LSAM benefits as (NCHRP, 1997): (i) use of lower design asphalt binder content for mixture homogenization with complete aggregate coating when compared to conventional mixtures; (ii) require less crushing energy to meet the aggregate design parameters; (iii) has better resistance to rutting; (vi) need a thin thickness of surface course over the LSAM when it is used as the base course; (v) present a good resistance to thermal cracking; and (vi) shown a greater service life under heavy-duty traffic.

1.1. Research Objectives and Scope

The main objective of this study is to evaluate the performance of large stone asphalt mixture as leveling course using two rehabilitation test sections constructed in a slow and heavy traffic highway in Brazil.

The specific objectives of this research are:

- Evaluate the design method of LSAM by: (i) Marshall compaction, (ii) Rolling compaction and (iii) Superpave Gyratory compaction;
- Evaluate the mechanical behavior of LSAM comparing to conventional mixtures by: (i) Dynamic Modulus test, (ii) Flexural bending fatigue test and (iii) Permanent deformation test at LCPC French traffic simulator;

- Monitor the structural behavior of the test site by periodic Falling Weight Deflectometer (FWD) tests;
- Evaluate rutting performance of the rehabilitated test site for, at least, the first year of service life;
- Evaluate the pavement responses to the vehicle dynamic load over the two pavement structures of test sections through finite-layer analysis considering the viscoelasticity of the asphaltic layers.

The development of this study occurs by means of: (i) Dynamic modulus test to analyze the LSAM viscoelastic properties; (ii) laboratory permanent deformation test; (iii) fatigue test; (iv) periodic pavement monitoring campaigns of two rehabilitation test sections to analyze the performance of large stone pavements, and (v) pavement simulation to predict the fatigue performance of LSAM and validate with the test pavement sections.

Two pavement structures were evaluated, differing only on the surface course. The first pavement structure considered for the analysis is composed by dense hot mix asphalt (HMA) 9.5 mm as surface course, and LSAM 32 mm as the leveling course, resting above the remaining pavement infrastructure. The other pavement structure is composed of HMA 19 mm asphalt surface and the same LSAM 32 mm leveling course over the same remaining infrastructure. Both test sections are located in the Litoral Sul Highway in the State of Paraná, Brazil.

1.2. Thesis Outline

This thesis is structured in five chapters which the last four correspond to individual papers. Due to the structure used, some information may be repeated over this document. It is organized according to the following:

Chapter 1 presents the initial approach about characteristics of asphalt mixes with nominal maximum aggregate size (NMAS) above 25 mm, named Large Stone Asphalt Mixtures (LSAM). This chapter also mentions the research objectives, and the thesis outline.

Chapter 2 provides a literature review about the historical use of the LSAM in the world, as well as relevant characteristics of the LSAM design. The design methods of LSAM are evaluated according to the compaction mode: (i) Marshall compaction, (ii) Rolling compaction and (iii) Superpave Gyratory compaction.

Chapter 3 describes the LSAM material properties and the mechanical behavior through laboratory tests (Dynamic Modulus Test, Permanent Deformation Test, Fatigue Test and, also, Tension-Compression Fatigue Test).

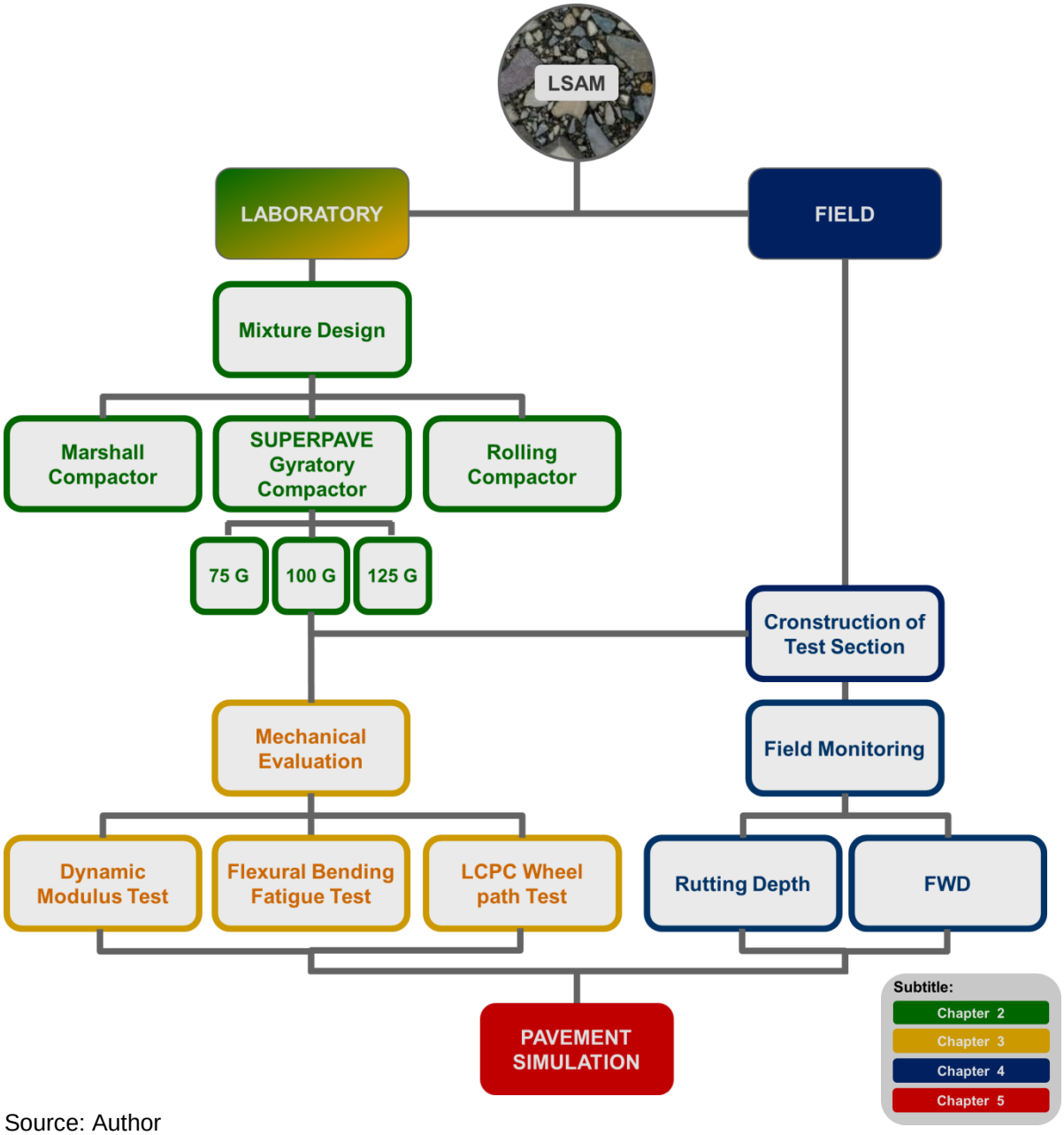
Chapter 4 presents the test sections characterization, the construction process and the field performance.

Chapter 5 focus on the pavement simulation considering the viscoelasticity of the asphalt mixtures of the test sections presented in chapter 4 and its instrumentation with strain gages and temperature sensors.

Chapter 6 presents the summary and conclusions of the thesis and recommendations for future work.

Figure 1 shows a flowchart with the steps considered in this study. The figure illustrates the three main sections: the laboratory tests, field construction and monitoring, and the pavement analysis.

Figure 1 - Flowchart of thesis steps



Source: Author

2. LARGE STONE ASPHALT MIXTURE DESIGN

2.1. Introduction and Background

Hot mix asphalt (HMA) has been extensively used as the surface course of pavements worldwide, once it provides a good traffic vehicle capacity to transport people and goods. Currently, this material is also used in lower layers of pavement structure to fighting the stresses caused by overloading, the increase of the traffic highway, and the increase of tire inflation pressure.

On 1980 in the United States, the increase of distresses associated with traffic loading brought the attention to the search on more durable solutions. The need of resistant pavement structures brought the development of the Large Stone Asphalt Mixture technology. The LSAM is defined as an asphalt concrete with nominal maximum aggregate size (NMAS) starting from 25 millimeters (KANDHAL, 1990; NCHRP, 1997; USACE, 2000). It has not been widely studied and it is mostly evaluated as a technique to reduce permanent deformation in the wheel path, and also improve the durability of pavements.

In Brazil, the Marshall mix design procedure is commonly used. However, it limits the mineral aggregate dimensions of asphalt mixture, and this is the main limitation to study the LSAM in many countries that still use the Marshall design method. The equipment specified for this procedure has a 100 mm diameter compaction mold for asphalt mixtures with maximum aggregate size (MAS) smaller than 25 mm. Although, there are other test procedures that can be used for LSAM design such as rolling compaction, gyratory compaction, and a modified Marshall design method with 150 mm diameter specimens. The last one was developed specifically for testing large stone asphalt mixtures, due to the resistance of most agencies to buy new equipment that was still expensive and unusual for mixture designs at that time (KANDHAL, 1989).

This chapter presents the evaluation of LSAMs design with different compaction methods: (i) Marshall compaction, (ii) Rolling compaction and (iii) Superpave Gyratory Compaction (SGC) varying the compaction energy in 75, 100, and 125

gyrations. The locking point was also evaluated at different gyratory compaction energies, allowing the use of an additional parameter for LSAM design analysis.

2.1.1. LSAM History and Use

Research reports usually describe the use of LSAM as base course at the flexible pavement structure. However, it could be applied as leveling course with requirement of a thicker layer due to the large dimensions of the aggregates.

This type of asphalt mixture has been used since the 1900s by the Warren Brothers Company in United States. They had been the first to use variations of aggregate gradation with particles dimensions higher than 25 mm that allowed remarkable characteristics of resistance to withstand the traffic conditions. The motor vehicle had become the imperative demander for better roads which was good enough for the horse and carriage on that time (THE CAMBRIDGE TRIBUNE, 1916).

This asphalt pavement was denominated “bitulithic pavement” and the term “bitulithic” was defined as a dense mixture of asphalt and crushed stone. The differential feature was the screening of the stone. It was used ranging from one and a half inch (38 mm) to the two hundred inch mesh (50 mm). The objective was to use various sizes of aggregate with filler and asphalt to result in a solid mass by means of pack the void space between large aggregates with smaller aggregates, resisting to the traffic use (WARREN BROTHERS COMPANY, 1912).

Since that, LSAMs had been more used among the North American highway agencies. The Project NCHRP 4-18 with final report published as NCHRP Report 386 (1997) showed that thirty of fifty-two state highway agencies in the US had constructed LSAM pavements. The main states with LSAM experience are Kentucky, Pennsylvania and Iowa.

In Kentucky, the laboratory investigation of LSAM was complemented with construction of test sections: Louisa Bypass Highway (Lawrence County) and Mountain Parkway (Powell County). A dense LSAM had been considered as the base course in pavement structure. The first one was done with thirty centimeters

(12 in) considering a mixture with 37.5 mm NMAS. The asphalt binder used was non-modified and classified as AC-20 (MAHBOUB, 1990). The experience in Kentucky had suggested that LSAM can be designed as a conventional mixture with some modifications to existing procedures. The main points to be considered are the plant and paver process. The segregation is a potential problem for LSAM at these operations.

Mahboub and Williams (1990) recommended that the lift thickness of LSAM pavement layer should not be less than 87.5 mm (3.5 in) considering a 37.5 mm top size gradation. It is necessary to enable an adequate degree of freedom for aggregate reorientation during compaction procedure.

In China, Cao et al. (2011) reported the use of LSAM as an open-graded mixture for porous layer (13% to 18% air voids). The main objectives were to contain the reflective cracking and to enable the pavement drainage. Due these functions, the mixture had been denominated as “Large Stone Porous Asphalt Mixture” (LSPM).

The rehabilitated pavement with LSPM has been evaluated during five years. Those who had the mixture applied as the leveling course presented good resistance to rutting, fatigue cracking, and reflective cracking. The others pavements, which had the LSPM as the surface course, had exhibited good resistance to the same distresses, but showed more coarse surface issues, due to the large dimensions of the aggregates (YUFENG, SONNEN and HUBER, 2009).

The increase in traffic volume and axle loads has culminated in traffic loading conditions beyond the current design class on roads in South Africa. This scenario emphasizes the need of road pavements resistance to increasing traffic loads. It had considered the search of asphalt mixtures with more stability and durability (HUGO et al., 1990).

The solution was to develop a research and implementation project to search for cost-effective heavy-duty asphalt layer pavements, conducted by the Southern African Bitumen and Tar Association (SABITA). The Large Aggregate Mixes for Bases (LAMBS) technology was studied for rehabilitation on the M2 Motorway in

Johannesburg. Several sections were constructed in South Africa. The initial ten sections had been done using various grading curves and the three last were constructed as fully-instrumented test sections (EMERY, 1996).

In the United Kingdom, the use of large stone mixtures is considered efficient and it has been denominated as “Dense Bitumen Macadam” (DBM) with aggregate dimensions above 37.5 mm (HUGO et al., 1990). The “Grave-bitume”, from France, is an asphalt concrete with significant concentration of coarse aggregates and good stone-on-stone contact between these particles. However, these large mixtures are limited to NMAS of 19 mm because the French technical believes that dimensions upon that will bring serious problems with segregation (HINGLEY, PEATTIE and POWELL, 1976).

The closest to LSAM in Brazil is the “Pré-Misturado a Quente”, specified by some Department of Transportation Agencies (“Departamento de Estradas e Rodagem do Estado de São Paulo” - DER/SP and “Departamento Estadual de Infraestrutura de Santa Catarina” – DEINFRA/SC). The maximum particle dimension of this mixture is limited to 38 mm. Table 1 presents a resume about the commented use of mixtures with large stones around the world and its relevant characteristics.

Table 1 - Characteristics of the large asphalt mixtures around the world

Country	Author	Definition for asphalt mixtures with large stones	Aggregate maximum sizes	Mixture gradation
US	WARREN BROTHERS COMPANY (1912)	"bitulithic"	From 37.5 to 50 mm	Dense graded
	MAHBOUB (1990)	Large Stone Asphalt Mixture (LSAM)	From 25 to 50 mm	Dense graded
China	CAO et al. (2011)	Large Stone Porous Mixture (LSPM)	From 25 to 63 mm	Open graded
South Africa	EMERY (1996)	Large Aggregate Mixes for Bases (LAMBS)	From 37.5 to 53 mm	Dense graded
United Kingdom	HUGO et al. (1990)	Dense Bitumen Macadam (DBM)	Up 37.5 mm	Uniform graded
France	HINGLEY, PEATTIE and POWELL (1976)	"Grave-bitume" (GB)	19 mm	Dense graded
Brazil	DER/SP (2006)	“Pré-Misturado a Quente”	Maximum of 38 mm	Open graded

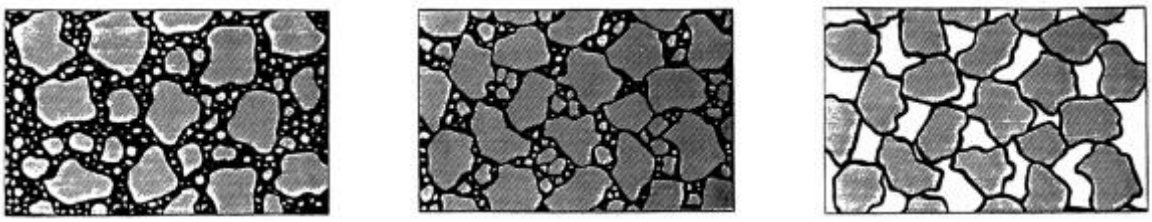
Source: Author

2.1.2. LSAM Materials

2.1.2.1. Aggregates

The LSAM can have different functions into the pavement structure, depending on the desired application. This variation is possible due to the characteristics of the selected grading curve, which can be dense mixture (Figure 2(a)), stone-filled mixture (Figure 2(b)), and open graded mixture (Figure 2(c)).

Figure 2 - Large Stone Asphalt Mixtures categories



(a) Dense graded (b) Stone-filled (c) Open graded

Source: Newcomb et al. (1993)

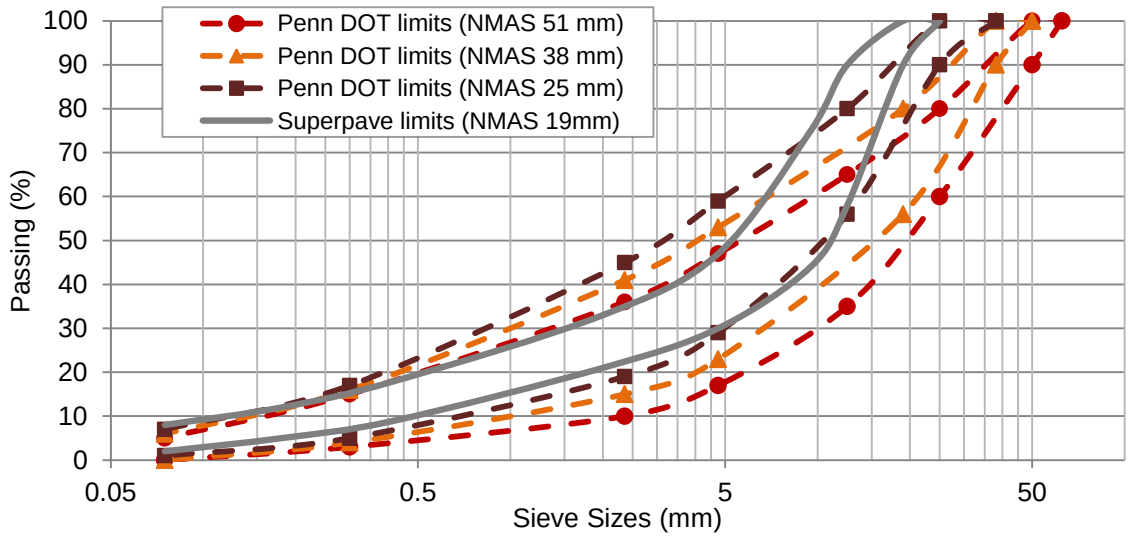
The dense mixture has been characterized as well graded which can develop resistance to load application through both aggregate interlock and asphalt viscosity, where the last one is responsible for the cohesion among the aggregate particles (HUGO et al., 1990). The mixture is characterized as high stability material with air void content varying between 4 and 8%. The Stone-filled mixture has small top size aggregates combined with large single sized aggregates which can develop resistance strength through the aggregate bridging effect. Open graded mixture is characterized by large top size, lower asphalt content and high air voids content (between 15 and 30%). Thus, this mixture presents high permeability, developing strength from the direct stone-on-stone contact (NEWCOMB et al., 1993). It also can delay the reflective cracking when applied as base course into the pavement structure (YUFENG, SONNEN and HUBER, 2009; CAO et al., 2011).

It is believed that the LSAM dense mixture is able to resist a very heavy traffic scenario, minimizing plastic deformation and, consecutively, the premature permanent deformation in the wheel paths. However, the grading composition of aggregate mixture should be the one which ensures the coarse aggregate interlock

with a good stone-on-stone contact. Besides that, it is important the use of cubic aggregate shape, rough surface texture, high abrasion resistance, and to know the volumetric characteristics of the mixture (absorption of asphalt by aggregates, proportion of fine aggregates, voids of mineral aggregates and voids full of asphalt) (NCHRP, 1997).

Newcomb et al. (1993) recommended grading limits for LSAM to the Pennsylvania Department of Transportation - Penn DOT. The suggested ranges for grading curves are presented in Figure 3 according to the target NMAS.

Figure 3 - LSAM gradation bend recommended by Penn DOT



Source: Adapted from Newcomb et al. (1993)

2.1.2.2. Asphalt Binder

The asphalt binder presents a secondary importance in HMA's mixtures for resistance to permanent deformation, against the aggregate skeleton predominantly response (HUGO et al., 1990). However, the fine aggregates involved by asphalt binder is important to the cohesiveness of the mixture, when combined with the frictional resistance of the aggregates from grading composition.

The most common binder used for LSAM production is the petroleum asphalt cement (AC) with high viscosity, usually without modification (KANDHAL, 1990; MAHBOUB, 1990; NCHRP, 1997). An adequate asphalt film thickness can ensure the workability

and durability of LSAM, been controlled by means of the asphalt content and the filler in mineral aggregate. The asphalt content must be convenient for a necessary percent voids, resulting in a maximum field density without an excessive reduction in mixture air voids (ZANIEWSKI and NALLAMOTHU, 2003).

Some researchers used modified asphalt binder for LSAM. Zhao and Huang (2010) recommended the modified asphalt binder as appropriated for LSPM when compared to conventional asphalt binder, because it results in better resistance to moisture damage. Huang (2000) also reported the use of SBS polymer modified binder for enhance large stone dense asphalt mixture performance.

2.1.3. LSAM Design Methods

The design methods for asphalt mixture have been developed to enhance the material properties and improve its field performance. The selection of the design procedure depends on the type of mixture, the local traditional method used, and the equipments available. By the design procedure, the proportion of each material is chosen to compound the asphalt mixture. The aggregate gradation is selected according to the purpose, which should ensure mixture stiffness, stability (resistance to permanent deformation), durability, workability, resistance to fatigue cracking and to moisture damage (NAPA, 2002; BERNUCCI et al., 2010).

The presence of large size aggregates in the asphalt mixture requests a laboratory compaction method able to orientate the aggregates in a way to attribute acceptable volumetric properties and good mechanical properties. The use of coarse aggregates for pavement layers is not new, but an accurate procedure to determine the asphalt content of this mixture is still not well defined. The Marshall mix design technology does not allow the right accommodation of aggregates due to the restricted dimensions of the standard compaction mold. However, the Marshall method was considered as an alternative for LSAM design using modifications in some apparatus developed by Kandhal (1990). The modified Marshall has larger mold diameter (150 mm), which became necessary a compaction with the same amount of energy per unit volume in modified mold as the conventional Marshall mixture design. After the Superpave implementation, the use of molds with 150 mm (6 inches) provided

another option for design of heavy duty mixes (as LSAM) for asphalt pavements (NAPA, 2002).

2.1.3.1. Marshall Compaction

Marshall mix design was extensively used in United States (US), achieving 76% of the states according to a survey conducted in 1984 (KANDHAL 1990; ANWAR, 2014). For large stone mixtures, this design procedure gives limitations to the aggregate dimensions, specifying a compaction mold of 100 mm (4 inches) which is appropriate to asphalt mixtures with NMAS up to 25.4 mm (DNER-ME 043/95, 1995). New methods were thus necessary to be developed to enable the LSAM investigation and use.

Several design researches about large aggregate mix used the Marshall method with some adaptations. The majority of LSAM projects in the US were designed by the Marshall modified compaction mold developed by Kandhal (1990) with diameter corresponding to 152 mm (6 inches) and 85mm (3.4 inches) height. The increase of the Marshall hammer mass (from 4.5 kg to 10.2 kg) and the number of blows (from 75 to 112 blows for heavy duty pavements) was necessary to ensure the same compaction energy per unit volume to the new specimen dimensions. Most agencies believed that the modified procedure design could save costs compared to acquire new equipment for mixture design (KANDHAL, 1990; NEWCOMB et al., 1993; PRICE and ASCHENBRENER, 1994).

2.1.3.2. Gyratory Compaction

The asphalt mixtures densification was extensively studied by technicians and engineers specialized in asphalt pavements, once the final volumetric characteristics were affected by the compaction. At the end of the 30 decades in Texas, the gyratory laboratory compaction was considered the best method to reproduce the field compaction. It was clearly observed the variation in densification when the impact and the shear compaction methods were compared (COMINSKY, LEAHY and HARRIGAN, 1994).

The Superpave (Superior Performance Asphalt Pavements) Mix Design was developed from SHRP (Strategic Highway Research Program) and it is the most used design method with the Superpave Gyratory Compactor (SGC) in the United Stated. The Superpave system uses four traffic levels (Table 2) as a design criterion based on the number of loads repetition that means the number of equivalent single axle loads (ESALs) estimated to pass over the asphalt mixture, which will receive the designed asphalt mixture, during the project period.

Table 2 - Number of gyrations according to the traffic and the road characteristics

$N_{ESAL\ (Millions)}$	Compaction parameters according to the traffic scenario			
	Nini*	Ndes**	Nmax***	Traffic characteristic
< 0.3	6	50	75	Secondary roads, low traffic volume
0.3 a < 3	7	75	115	Collecting roads, medium traffic volume
3 a < 30	8	100	160	Highways, heavy traffic volume
> 30	9	125	205	Federal Highways, very heavy traffic volume

Source: Asphalt Institute (2001)
*Nini – Initial number of gyrations
**Ndes – Design number of gyrations
***Nmax – Maximum number of gyrations

The Superpave design method (AASHTO M 323/04) has the maximum aggregate dimensions limited from 25 mm to 37.5 mm, and is reported as the most appropriate design method for asphalt mixtures (BUCHANAN and BROWN, 2001). There are preliminary studies done by the AAMAS (Asphalt-Aggregate Mix Analysis System), from NCHRP (VON QUINTUS, 1991), which indicates the gyratory compaction better than the impact compaction to simulate the particles orientation in the field. Besides that, the gyratory compactor is able to produce the LSAM due to its molds with larger dimensions (150 mm of diameter).

The NCHRP Report 386 (1997) presents the compaction conditions used for LSAM design and evaluation in NCHRP Project 4-18/1997. The design procedure was done as compatible as possible to the conventional HMA Superpave design method with respect to specimen preparation, compaction and mix analysis. For densification, the

Texas DOT Gyratory Compactor was used with an inclination angle of 5º, 120 cycles, 30 RPM and 375 kPa. Other options of compaction angle and force were investigated in the study. The report recommended the use of SGC or rolling wheel compaction as the standard practice of LSAM design, considering the Marshall modified compaction as the last possibility if the other two are not available. For SGC, the determination of a convenient angle of gyration is necessary to achieve satisfactory compaction without an excessive number of gyrations which may cause the aggregate breakage (NCHRP, 1997).

2.1.3.3. Rolling Compaction

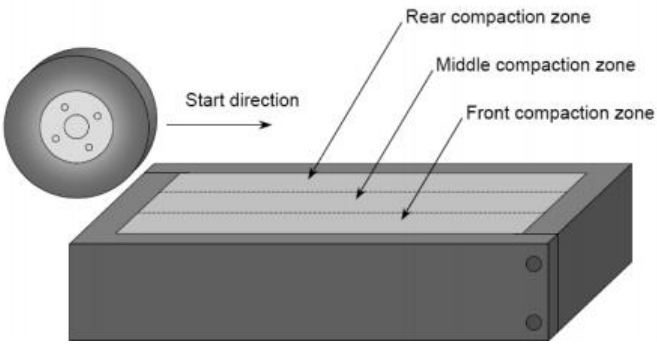
The needs to create representative laboratory specimens similar to the field specimens brought the development of the rolling wheel compaction. The rolling compaction is based on pressure distribution thought the contact area between a pneumatic wheel, or a metallic wheel, and the pavement surface. An equipment commonly used for laboratory rolling compaction is the LCPC rubber-tired compactor developed by the French *Laboratoire Central des Ponts et Chaussees* (LCPC), illustrated in Figure 4.

Figure 4 - Preparation of specimens in LCPC French Roller Compactor

Source: Author

The French Roller compacts the samples in three different wheel paths (Figure 5) allowing the realignment and orientation of the coarse aggregates in a similar way as done in the field compaction. Mixtures with large aggregate size could be easily accommodated in roller compaction mold compared to the Marshall conventional mold (SWIERTZ, MAHMOUD and BAHIA, 2010).

Figure 5 - Roller compaction wheel paths



Source: Swiertz, Mahmoud and Bahia (2010)

The procedure occurs with the passing sequence of the rubber tire which has the tire pressure varying from 0.3 to 0.6 MPa, according to the European specification EN 12697-33 (2003). Slabs compacted by the equipment have a variable height up to 100 mm and a base area of 50 cm × 18 cm.

The rolling compaction design procedure is related to the analysis of volumetric properties as the previous mentioned procedures. The specimens are prepared with variable asphalt content applied to the same grading composition. After compaction, cylindrical specimens are extracted from the slabs and used to determine the volumetric properties.

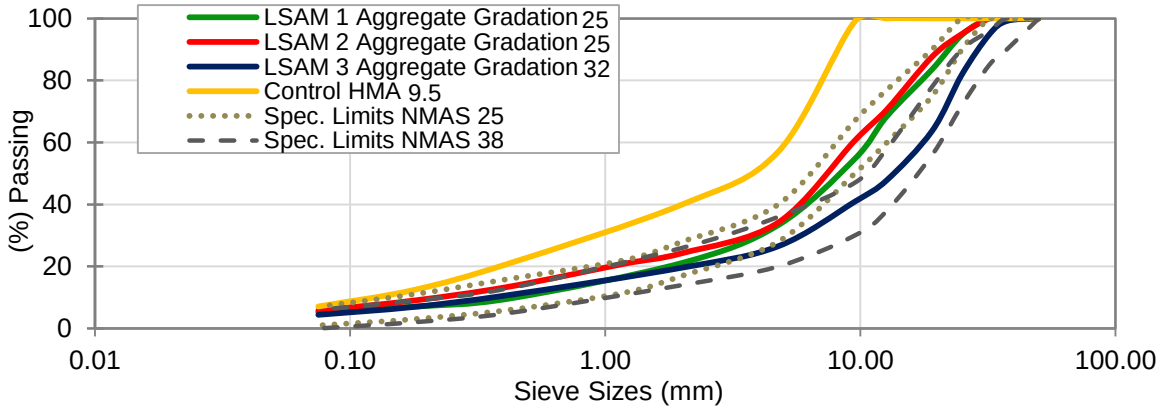
2.2. LSAM Design: Results and Analyses

2.2.1. Materials

Three aggregate gradations (continuous grading curves), with granite aggregate were selected for the design analysis of LSAM. The mix gradations have maximum aggregate size starting from 25 mm to 37.8 mm. The gradation limits were used from the Asphalt Institute (2001) specifications for NMAS 25 mm and NMAS 38 mm. It was necessary add 1.5% of hydrated lime for enhance the adhesion between the asphalt binder and the aggregates (a common practice used by the Brazilian agencies). The hydrated lime use enhances the asphalt mixture antistripping ability, and the resistance to moisture damage (LITTLE AND EPPS, 2001; AL-QADI et al., 2014).

The grain size distribution curves selected for LSAMs are presented in Figure 6 with the limits from Asphalt Institute (2001) specifications. The grading curve LSAM 1 and LSAM 2 correspond to mixtures with NMAS of 25 mm from same mineral origin, but from different quarries (Mandirituba e Itapoá, successively). The LSAM 3 is a mixture with higher NMAS (32 mm), from the same quarry of LSAM 2 (Itapoá). Aggregates physical and mechanical characteristics were evaluated as important information been presented in Table 3.

Figure 6 - Aggregate gradation curves



Source: Author

Table 3 - Aggregate characterization

Mixtures	Materials	Apparent Specific Gravity	Saturated Surface Dry Specific Gravity	Bulk Specific Gravity	Flat and Elongated Particles (1:3) (%)	Flat and Elongated Particles (1:5) (%)	Absorption	EA	LA
LSAM 1	Aggregate 1”	2.625	2.609	2.599	15	3	0.379	X	31
	Aggregate 3/4”	2.647	2.625	2.611	10	2	0.507	X	27
	Aggregate 3/8”	2.727	2.693	2.673	X	X	0.747	X	X
	Fine aggregate	2.765	2.760	2.757	X	X	0.103	76.4	X
	Hydrated lime	2.337	X	X	X	X	X	X	X
LSAM 2	Aggregate 1”	2.802	2.786	2.778	8.3	0.2	0.315	X	23.6
	Aggregate 3/4”	2.806	2.784	2.772	6	0	0.439	X	26
	Aggregate 3/8”	2.806	2.764	2.741	X	X	0.851	X	X
	Fine aggregate	2.960	2.886	2.848	X	X	1.326	60	X
	Hydrated lime	2.337	X	X	X	X	X	X	X
LSAM 3	Aggregate 1 1/2”	2.791	2.769	2.762	15	2	0.308	X	35
	Aggregate 3/4”	2.806	2.784	2.772	6	0	0.439	X	26
	Aggregate 3/8”	2.806	2.764	2.741	X	X	0.851	X	X
	Fine aggregate	2.960	2.886	2.848	X	X	1.326	60	X
	Hydrated lime	2.337	X	X	X	X	X	X	X

Source: Author

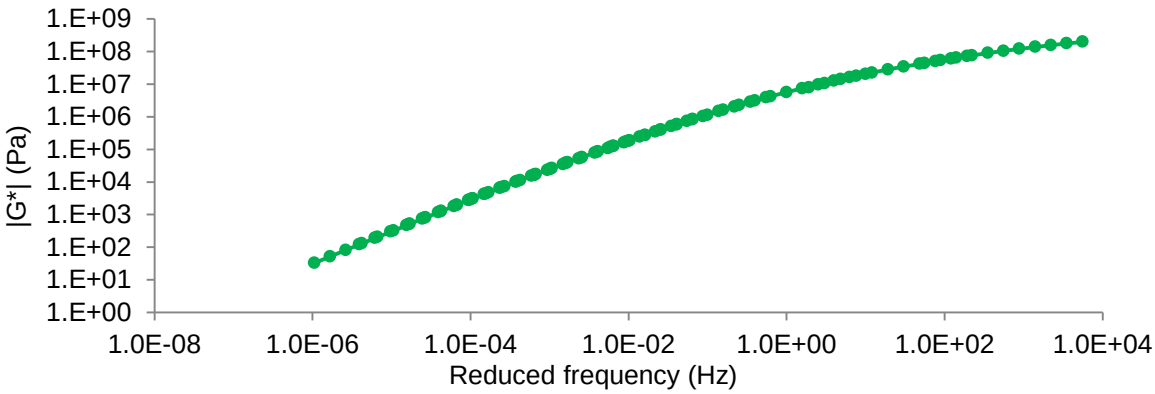
One neat asphalt binder with penetration grade 30/45 (AC 30/45) and performance grade PG 58V-XX was used in this study. The rheological properties of asphalt binder were determined by a Dynamic Shear Rheometer (DSR) test after RTFOT (Rolling Thin Film Oven Test) artificial aging in the laboratory. The Table 4 summarizes the neat asphalt binder physical properties and the rheological properties, by means of complex shear modulus and phase angle master curves (frequencies varying from 0.1 Hz to 10 Hz and temperatures starting from 0°C to 80°C with 0.1% of strain level, according recommendations from ASTM D7175-15), are presented in Figure 7 and Figure 8.

Table 4 - Physical properties of neat asphalt cement

Property		ASTM standard	Requirements	Units	AC 30/45
Penetration		D 5	30 to 45	0.1 mm	34
Density				-	1.007
Softening Point		D 36	> 52	°C	52.6
Solubility in Trichloroethylene		D 2042	> 99.5	% mass	99.9
Brookfield Viscosity*	135°C	D 4402		cP	425
	150°C	D 4402		cP	210
	177°C	D 4402	76 to 285	cP	76

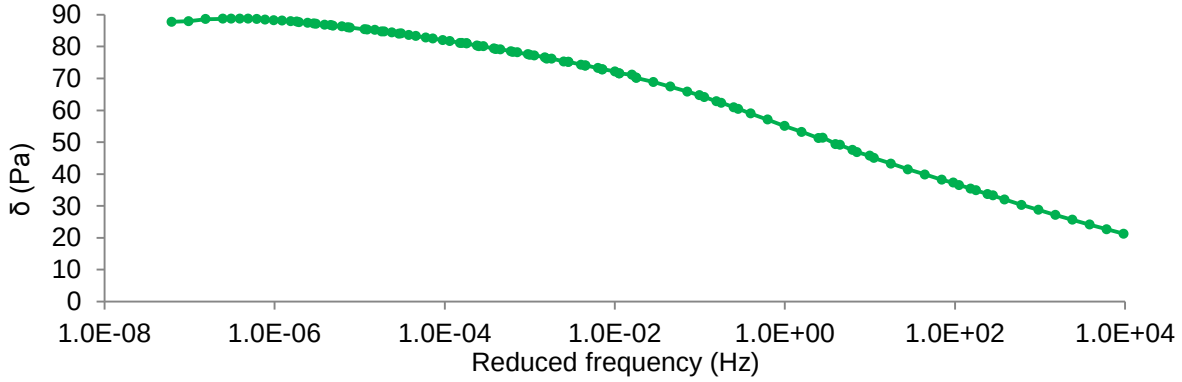
Source: Author
* Spindle 21 and 20 rpm

Figure 7 - |G*| master curve of neat asphalt binder CA 30/45 after RTFOT aging (Tref = 20°C)



Source: Author

Figure 8 - δ master curve of neat asphalt binder CA 30/45 after RTFOT aging (Tref = 20°C)



Source: Author

2.2.2. Design Requirements

The designs of LSAM were done by different compaction procedures to evaluate its influence considering the volumetric properties. The mixtures were designed at 4% air voids, in accordance with the requirements of voids filled of asphalt (VFA) and voids in mineral aggregate (VMA) from Asphalt Institute (2001) presented in Table 5.

Table 5 - Volumetric criteria for Superpave design

$N_{ESAL\ (Millions)}$	Voids in mineral aggregate (VMA) % minimum according to the NMAS (mm)						Voids filled of asphalt (VFA) %
	37.5	25.0	19.0	12.5	9.5	4.75	
< 0.3	11.0	12.0	13.0	14.0	15.0	16.0	70 – 80
0.3 a < 3	11.0	12.0	13.0	14.0	15.0	16.0	65 – 78
3 a < 30	11.0	12.0	13.0	14.0	15.0	16.0	65 – 75
> 30	11.0	12.0	13.0	14.0	15.0	16.0	65 – 75

Source: Adapted from Asphalt Institute (2001)

The mixture volumetric parameters were evaluated for the different compaction methods: (i) Impact compaction with 75 blows per face using a 100 mm diameter mold (Conventional Marshall Mix Design), (ii) Rolling compaction (LCPC) and (iii) Gyratory compaction (Superpave Mix Design) with 150 mm diameter mold varying the compaction energy by means of the number of gyrations.

The design binder content and the volumetric parameters of the mixtures were analyzed according to the compaction method. The specimens prepared with impact

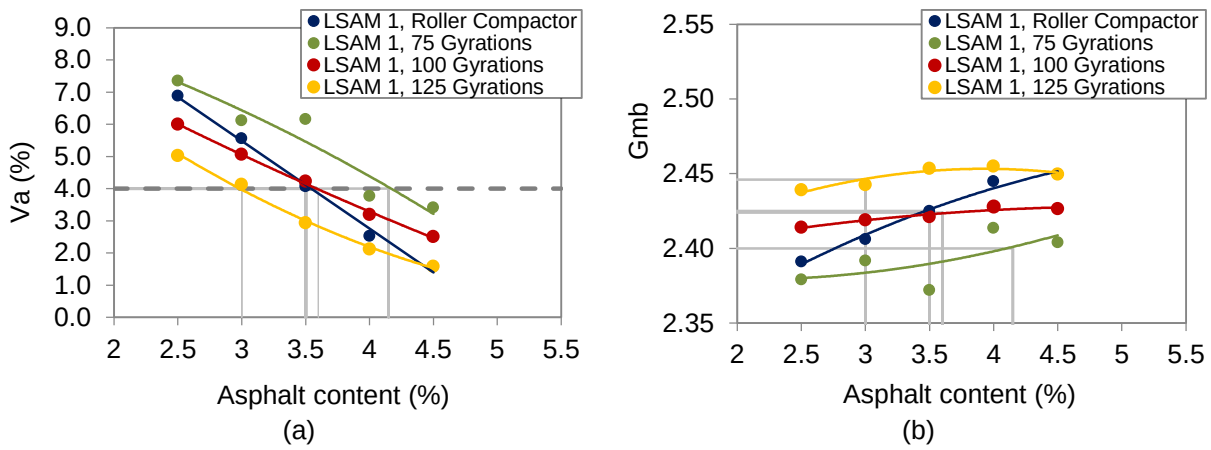
compaction had 100 mm diameter and about 63 mm high, which were also the same dimensions of extracted specimens from slabs prepared with the rolling compactor. The other specimens from gyratory compaction had 150 mm diameter and about 115 mm of height.

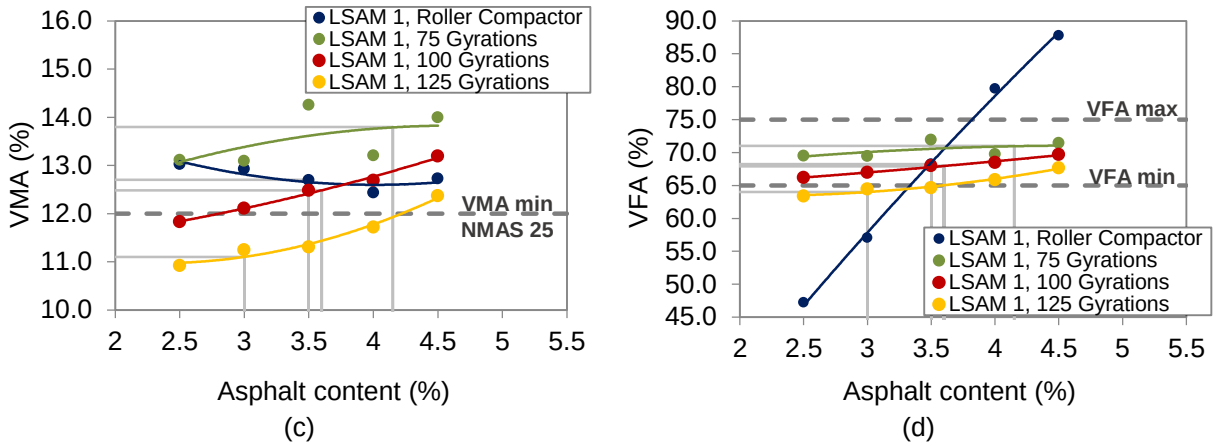
The topics 2.2.3, 2.2.4 and 2.2.5 present the comparisons of the volumetric parameters for the mixture design, and the design binder content obtained. The parameters presented are: Air voids (Va), Bulk specific gravity of compacted Asphalt Mixture (Gmb), Voids in mineral aggregate (VMA) and Voids filled with asphalt (VFA).

2.2.3. Gyratory versus Rolling Compaction

The gyratory compaction at different compaction energy (75, 100 and 125 gyrations) with 150 mm diameter mold and the French standard rolling compaction was used to evaluate the LSAM 1 (NMAS of 25 mm) mix design (Figure 9). The design binder contents for gyratory and rolling compactions are shown in Table 6.

Figure 9 - Volumetric parameters: Gyratory versus Rolling Compaction (NMAS 25 mm)





Source: Author

Table 6 - Design binder content: Gyratory versus rolling compaction

Mixture	NMAS	Design Binder Content for 4% air voids (%)			
		SGC 125G	SGC 100G	SGC 75 G	Roller
LSAM 1	25 mm	3.0	3.6	4.2	3.5

Source: Author

Different asphalt contents were obtained from the variation of compaction method and effort for the same grading composition LSAM 1. The mixtures prepared with 75 gyrations for Ndesign presented the higher design binder content when compared to the upper gyration levels, as expected. If the number of gyrations during compaction is higher, the mixture suffers more compressive effort and it needs lower asphalt content to lubricate the aggregates particles to get a desirable air voids for design.

Watson et al. (2008) observed the same in mixtures with NMAS of 25 mm concluding that as the gyratory level increase, the design binder content decrease. The authors reported a decrease of about 23%, when the number of gyrations changed from 35 to 110 using the SGC. An inappropriate design procedure may culminate in issues for field compaction and premature distresses in the surface asphalt layer caused by excess or lack of asphalt binder.

The volumetric requirements were not respected from the mixture designed by gyratory compactor with 125 gyrations at the design binder content determined by the 4% air voids. Probably, the compaction energy was high resulting lower VMA and unsatisfactory VFA. The lower VMA can be associated with a good permanent

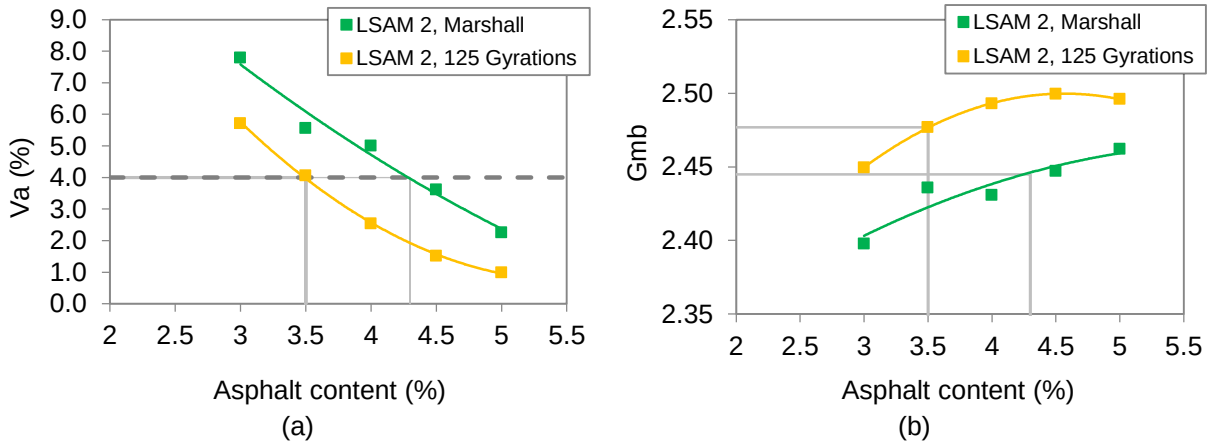
deformation resistance, but it could indicate an insufficient space between particles to accommodate the asphalt binder compromising the mixture stability (NASCIMENTO, 2008).

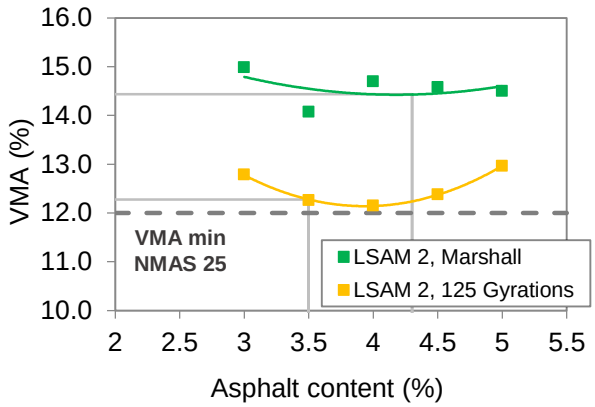
The rolling design presented a different behavior compared to the volumetric design curves of gyratory compaction. The volumetric parameters had been more sensitive to the variation of asphalt content. Engineering principles of the wheel rolling compaction were based on create specimens more representative of mixtures compacted in the field by pneumatic roller compactor. However, the compaction procedure and specimen sizes may not ensure the homogeneity of the material over the laboratory slabs (SWIERTZ, MAHMOUD and BAHIA, 2010). There is no consensus about which method is the best to simulate the field conditions. The volumetric of specimens could be identical even varying laboratory compaction methods but they might be mechanically different (VASCONCELOS, 2004; GEORGIOU, LOIZOS and LEVENTIS, 2015).

2.2.4. Gyratory versus Marshall Compaction

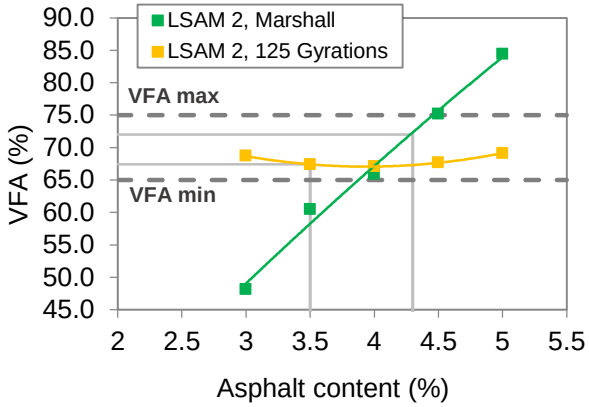
The gyratory compaction at 125 gyrations with 150 mm diameter mold and Marshall compaction with 75 blows per face in a 100 mm diameter mold was used to evaluate the LSAM 2 (NMAS of 25 mm) mix design (Figure 10) and its sensibility to different compaction procedures. The design binder content for gyratory and Marshall compactions are shown in Table 7.

Figure 10 - Volumetric parameters: Gyratory versus Marshall Compaction (NMAS 25 mm)





(c)



(d)

Source: Author

Table 7 - Design binder content: Gyratory versus Marshall compaction

Mixture	NMAS	Design Binder Content for 4% air voids (%)	
		Marshall	SGC 125G
LSAM 2	25 mm	4.3	3.5

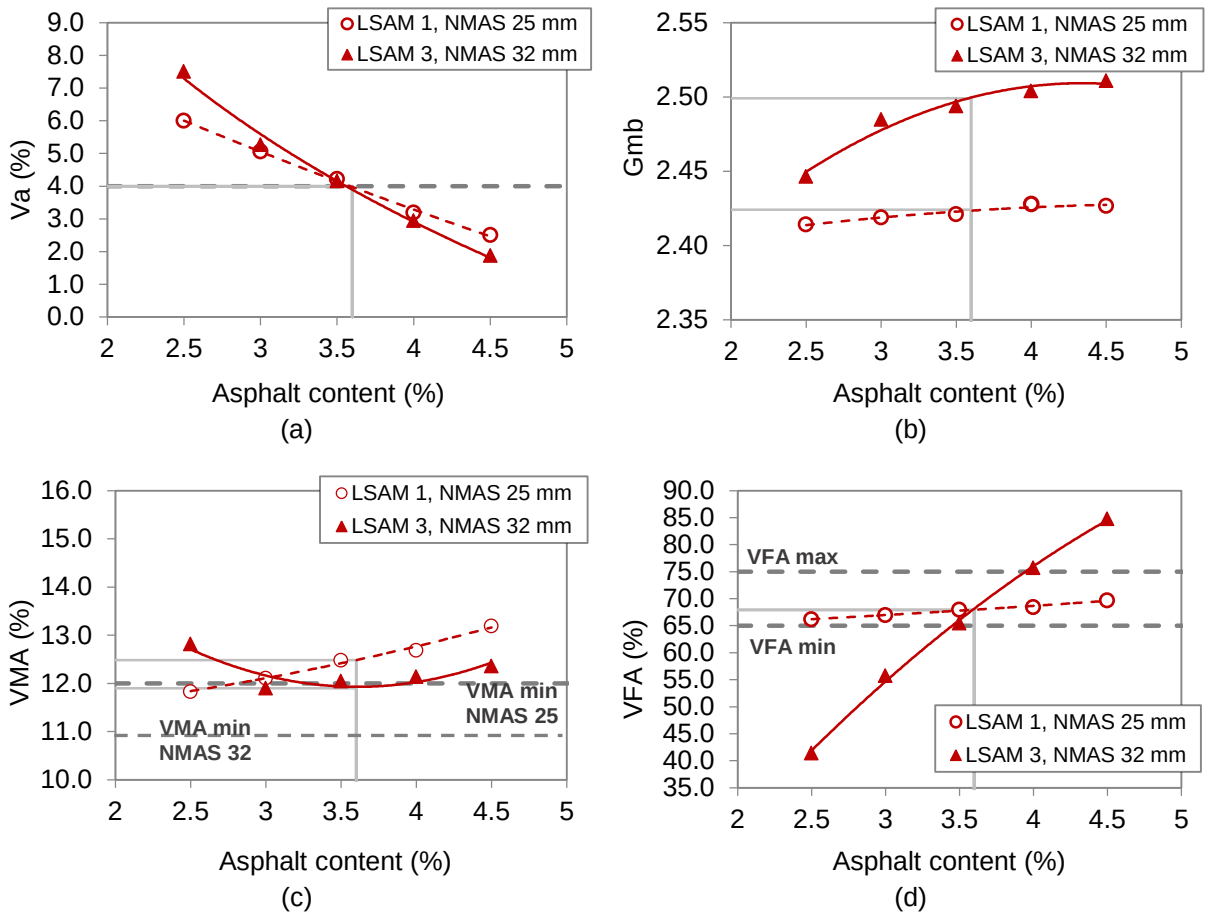
Source: Author

The LSAM 2 with NMAS 25 mm had been designed by Conventional Marshall Mix Design presenting the design binder content 23% higher than the obtained by Gyratory Compaction with 125 gyrations, due to the different compaction principle (by impact and shear movement, respectively), which allows different orientation of aggregate particles into the compacted mixture. Consuegra et al. (1989) compared the gyratory compaction, the Marshall impact hammer, mobile steel wheel simulator, California kneading compactor, and the Arizona vibratory-kneading compactor, and reported that the gyratory compaction was the method that creates specimens more similar to the pavement samples. The Marshall hammer was defined as the one with less probability to create representative of field samples. The procedure does not allow a partial free face to the self-orientation of the aggregates as occurs in the field compaction.

2.2.5. 25 mm NMAS versus 32 mm NMAS

The gyratory compaction at 100 gyrations with 150 mm diameter mold was used to evaluate the LSAM 3 (NMAS of 32 mm) mix design (Figure 11). The design binder content for gyratory compaction is shown in Table 8.

Figure 11 - Volumetric parameters: 25 mm NMAS versus 32 mm NMAS



Source: Author

Table 8 - Design binder content: 25 mm NMAS versus 32 mm NMAS

Mixture	NMAS	Design Binder Content for 4% air voids (%)
		SGC 100G
LSAM 1	25 mm	3.6
LSAM 3	32 mm	3.6

Source: Author

The LSAM 3 with NMAS 32 mm was designed by gyratory compaction due to the molds capacity to accommodate large size aggregates. After the design of the other large stone mixtures, the result analysis and experience provided the sensibility of the suitable compaction to be used for the known materials and volumetric parameters desired.

The mixtures designed by gyratory compaction with 100 gyrations presented the same design binder content. However, LSAM 1 and LSAM 3 are composed of same mineral aggregate source from different quarries and different grading curves,

presenting different densification at 4% of air voids (Gmb_{LSAM1} = 2.424 e Gmb_{LSAM3} = 2.499), which explain the equal asphalt content for mixtures with different NMAS. The resume of design binder content for each design procedure adopted is show in Table 9.

Table 9 - Effect of compaction method on design binder content

Mixture	NMAS	Quarry	Mineral Origin	Design Binder Content for 4% air voids (%)				
				Marshall	SGC 125G	SGC 100G	SGC 75 G	Roller
LSAM 1	25 mm	Mandirituba	Granite	-	3.0	3.6	4.2	3.5
LSAM 2	25 mm	Itapoá	Granite	4.3	3.5	-	-	-
LSAM 3	32 mm	Itapoá	Granite	-	-	3.6	-	-

Source: Author

2.2.6. Locking Point Analysis

The locking point (LP) was developed to avoid overcompaction of asphalt mixture samples in SGC during the design procedure (VAVRIK, 2000). The LP means the number of gyration at which the internal aggregate structure is locked and further compaction results in little densification, what might bring greater potential for aggregate breakage, compromising the mechanical behavior of the asphalt mixture (NCHRP, 2007; WATSON et al., 2008).

There are different definitions proposed to determine the locking point from SGC data. This thesis uses the definition developed by Vavrik and Carpenter (1998), where the locking point is determined as the first gyration in the first occurrence of three gyrations of the same height preceded by two sets of two gyrations with the same height. Table 10 presents the average values of locking point obtained for compacted specimens for each asphalt content of designed mixtures at different compaction energies in the SGC.

Table 10 - Average values of locking point during mixture design

Asphalt Content (%)	LSAM1 (NMAS 25 mm)			LSAM2 (NMAS 25 mm)	LSAM3 (NMAS 32 mm)
	75 G	100 G	125 G	125 G	100 G
2.5	Did not reach	91	94	-	71
3.0	Did not reach	89	89	76	92
3.5	Did not reach	91	92	74	84
4.0	Did not reach	92	89	70	82
4.5	Did not reach	90	86	76	86
5.0	-	-	-	73	-
Design binder content (%)	4.2	3.6	3.0	3.5	3.6
LP at design binder content	X	91	89	74	84
% Gmm estimated for design binder content	X	95.5	94.2	94.6	95.7

Source: Author

The average locking point for LSAMs at design binder content ranged from 74 to 91 gyrations, indicating the compaction energy at 100 gyrations as appropriated for the design of all LSAMs evaluated. Thus, the higher compaction energy (125 gyrations) may result in excessive compaction and aggregates degradation, which will change the aggregate gradation. The use of 75 gyrations for LSAM1 was not sufficient to reach the locking point in any asphalt content.

For 100 gyrations, the LSAM1 and LSAM3 have similar densification values which are represented by the reached percentage of Gmm. However, the locking point of LSAM1 (91) was higher than the locking point of LSAM3 (84), indicating LSAM 1 needed a greater number of gyrations to reach the same level of compaction which was not expected. However, the workability in compaction is also related to several other variables, other than the NMAS, as: the aggregate crushing process, aggregate properties, and gradation shape (GUDIMETTLA, COOLEY AND BROWN, 2003), producing different locking point values. Once the quarry of LSAM 1 was different from LSAM 3, the aggregates source and properties might explain the results obtained.

For 125 gyrations, the LSAM1 and LSAM2 have also close values of densification, where locking point of LSAM1 (89) was higher than the locking point of LSAM2 (74), presenting again less workability of LSAM1. At this time, the main differences

between the two aggregates skeleton is the gradation curve, aggregate quarries, but the same aggregate crushing process, and different aggregate properties, which can influence the energy needed for densification.

3. MECHANICAL BEHAVIOR OF LARGE STONE ASPHALT MIXTURE

3.1. Introduction and Background

The asphalt mixtures are the most commonly used surface course of pavements in Brazil to ensure the security and comfort of road users. However, the asphalt mixture is also used in intermediate layers of pavement structures, providing better resistance to traffic loads. Thus, the distresses occurrence can be reduced, increasing the time between pavement maintenance interventions.

Large Stone Asphalt Mixture (LSAM) is a hot mix asphalt concrete, which has nominal maximum aggregate size above 25 mm, proposed as a rehabilitation solution for pavements that show rutting. This mixture can present higher modulus than the conventional HMA mixtures, almost 16% according to Huang (2000), and higher durability when used as base or leveling course into pavement structure (MAHBOUB, 1990). The LSAM technology was extensively used over the world in the 90 decade as a structural layer of asphalt pavements to resist permanent deformation. However, LSAM was mostly applied without proper technological control and design accuracy, with few studies about the degradation mechanism of this type of mixture.

It is believed that the LSAM can present technical and economic advantages when subjected to a very heavy traffic scenario, especially avoiding premature rutting, due to the presence of higher percentage of coarse aggregates when compared to conventional mixes. It can provide better contact between the coarse aggregates and form a mixture with a stable aggregate skeleton.

Within the LSAM, the stresses are transferred through contact between the large aggregates until they reach the underlying layers of the pavement (NCHRP, 1997). In order to maximize these effects, the LSAM must be as closely as possible to the point where the load is applied, therefore it is a good option to use a thin layer of conventional hot mix asphalt (HMA) as the surface course due to the rough final texture of LSAM layer (FERNANDO, BUTTON and CROCKFORD, 1997).

Zaniewski and Nallamothu (2003) report the permanent deformation as the most important parameter to be considered for LSAM evaluation. The asphalt-aggregate mixes depend on characteristics of both individual materials and created system: size, aggregate form, viscoelastic properties of asphalt binder, and the correctly mix design.

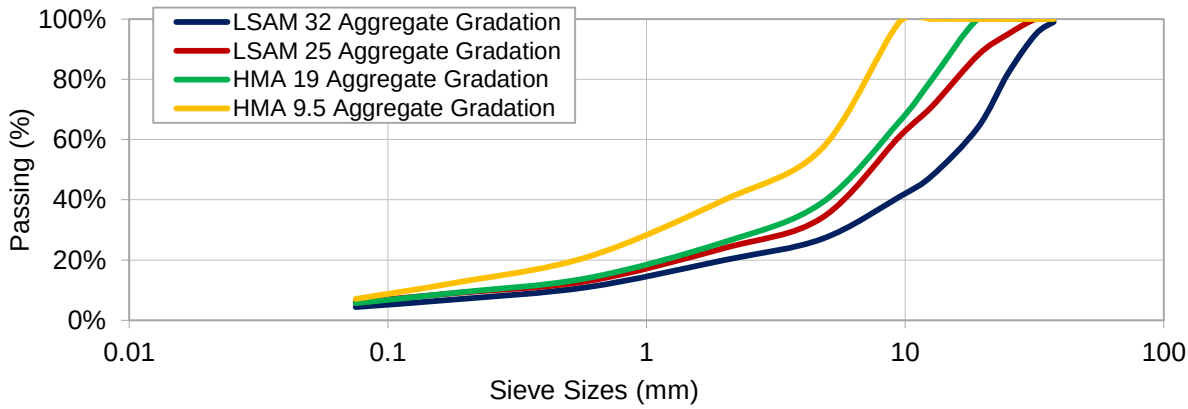
The objective of this chapter is to analyze the mechanical properties of asphalt mixtures with aggregates of the same mineral origin, but with different NMAS. The mixtures evaluated were used in the rehabilitation of two test sections in Brazil. The rehabilitation project described two test sections using the same thickness of LSAM as the leveling course with a thin layer of conventional HMA (NMAS 9.5 mm and NMAS 19 mm) as surface course. The main materials properties are described and the mixtures were characterized in laboratory through dynamic modulus test, flexural bending fatigue test, tension-compression fatigue test and the LCPC wheel track test (not all the mixtures were tested by all the mentioned mechanical tests, due to material availability).

3.2. Materials

Two granite aggregate gradations were selected for the laboratory evaluation, which are continuous grading curves for dense LSAMs. The aggregates were from Itapoá quarry. The mix gradations have maximum aggregate size starting from 25 mm to 38 mm. The grading limits were used from Asphalt Institute (2001) specifications, for NMAS of 25 mm and 38 mm. It was necessary to add hydrated lime to enhance the adhesion between the asphalt binder and the aggregates (common practice adopted by the Brazilian transportation agencies). As stated in the literature, the hydrated lime enhances the antistripping ability, improving the asphalt mixture resistance to moisture damage (LITTLE AND EPPS, 2001; AL-QADI et al., 2014).

For conventional HMA, one mix gradation of NMAS 9.5 mm and one of NMAS 19 mm were used to be compared to both LSAMs described above. All the mixtures were prepared with aggregates of the same mineral origin. The aggregate gradation curves selected for LSAMs and HMAs are presented in Figure 12.

Figure 12 - Aggregate gradation curves for asphalt mixtures analyses



Source: Author

A neat asphalt binder with penetration grade 30/45 (AC 30/45) and Performance grade, PG 58V-XX was used in this study. Table 11 summarizes the binder's physical properties.

Table 11 - Physical properties of neat asphalt cement

Property		ASTM standard	Requirements	Units	AC 30/45
Penetration		D 5	30 to 45	0.1 mm	34
Density				-	1.007
Softening Point		D 36	> 52	°C	52.6
Solubility in Trichloroethylene		D 2042	> 99.5	% mass	99.9
Brookfield Viscosity*	135°C	D 4402		cP	425
	150°C	D 4402		cP	210
	177°C	D 4402	76 to 285	cP	76

Source: Author
* Spindle 21 and 20 rpm

3.3. Asphalt Mixture Designs

All mixture designs were performed using the Superpave Gyratory Compactor (SGC) with 100 gyrations (BEJA et al., 2015), using samples of 150 mm in diameter and about 115 mm in height. The design binder contents for the LSAM 32, LSAM 25, HMA 19 and HMA 9.5 mixes were determined at 4% to 5% air voids range, and were 3.6%, 4.0%, 4.4% and 5.1%, respectively. All mixtures were in agreement with the Superpave volumetric requirements (AASHTO, 2013).

3.4. Mechanical Behavior

3.4.1. NMAS Influence on Mixture Stiffness

The complex modulus is a complex number which defines the stress-strain relationship of linear viscoelastic materials. The dynamic modulus |E*| is the absolute value of the complex modulus obtained from stress amplitude of axial load and recoverable strain amplitude of a material submitted to a continuously sinusoidal loading (DOUGAN et al., 2003; AASHTO, 2011).

The dynamic modulus test is normally performed with cylindrical specimens subjected to sinusoidal axial compression loading. The measurement of the correspondent vertical strain by linear variable differential transducers (LVDTs) is necessary to calculate the dynamic modulus and phase angle of the material.

The procedure is run at different temperatures and frequencies, which enables the construction of the material master curve at a reference temperature using the time-temperature superposition principle. The master curve as a function of reduced frequency describes the time dependency of the material, which can be used for mechanistic-empirical design procedures and for linear viscoelastic performance analysis of the asphalt concrete.

The linear viscoelastic characterization of LSAMs is still unexplored. Mohammad, Raghavendra and Obulareddy (2014) report that the mixtures designed for heavy traffic with large aggregates and higher grade binder usually have higher dynamic modulus values at high temperature. Investigation of Large Stone Porous Mixtures (LSPM) presented a good fit of dynamic modulus data using a sigmoidal function (PELLINEN, WITCZAK and BONAQUIST, 2004) to construct the master curve (WEI and LI, 2013)

The uniaxial dynamic modulus test was performed for three mixtures (with NMAS of 32 mm, 19 mm and 9.5 mm) using a hydraulic loading machine MTS 810 machine in accordance with the AASHTO T342-11 (2011) specification. The test was conducted in a stress controlled cyclic manner to measure the linear viscoelastic responses of

asphalt mixture limiting the axial strain between 50 and 150 microstrain without reach a damage stage. Each specimen was subjected to the testing frequencies of 0.1, 0.5, 1.0, 5.0, 10, 25 Hz and to the temperatures of 4.4, 21.1, 37.8, 54ºC, using 21.1ºC as the reference temperature to construct the master curves. The specimens were compacted by the SGC at the respective design binder content. The compacted specimens were cored and trimmed to obtain the dimensions of 100 mm diameter and 150 mm height.

The sigmoidal model used is the fitting function recommended by Pellinen, Witczak and Bonaquist (2004) which can fit the dynamic modulus test data obtained from temperatures varying from -18ºC to 55ºC. The sigmoidal function is presented in Equation (1). Figure 13 and Figure 14 shows the master curve for each asphalt mixture (APPENDIX A).

$$\log(|E^*|) = \delta + \frac{\alpha}{1 + e^{\beta - \gamma \log(\xi)}} \tag{1}$$

Where: $|E^*|$ is the dynamic modulus;

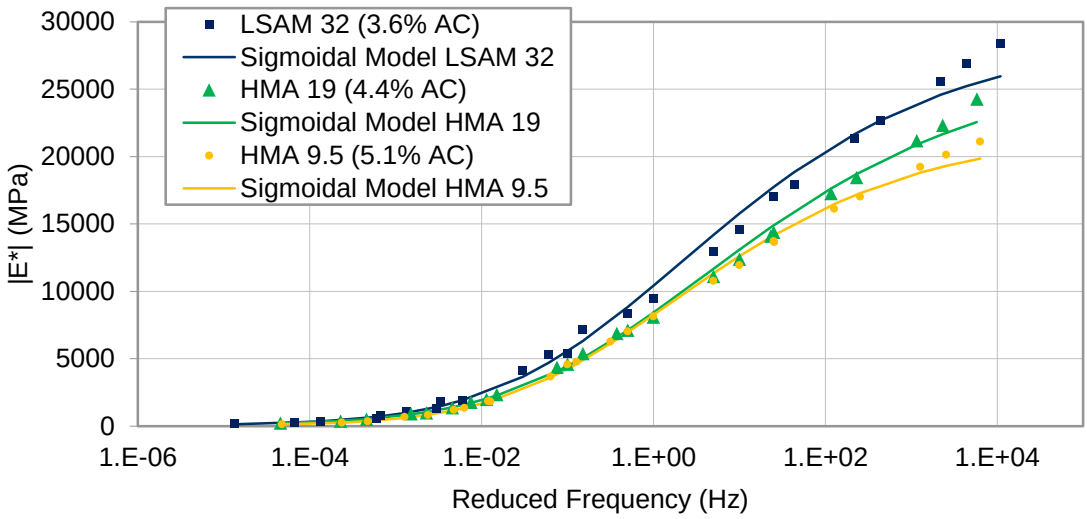
ξ is the reduced frequency;

δ is the minimum modulus value;

α is the span of modulus values;

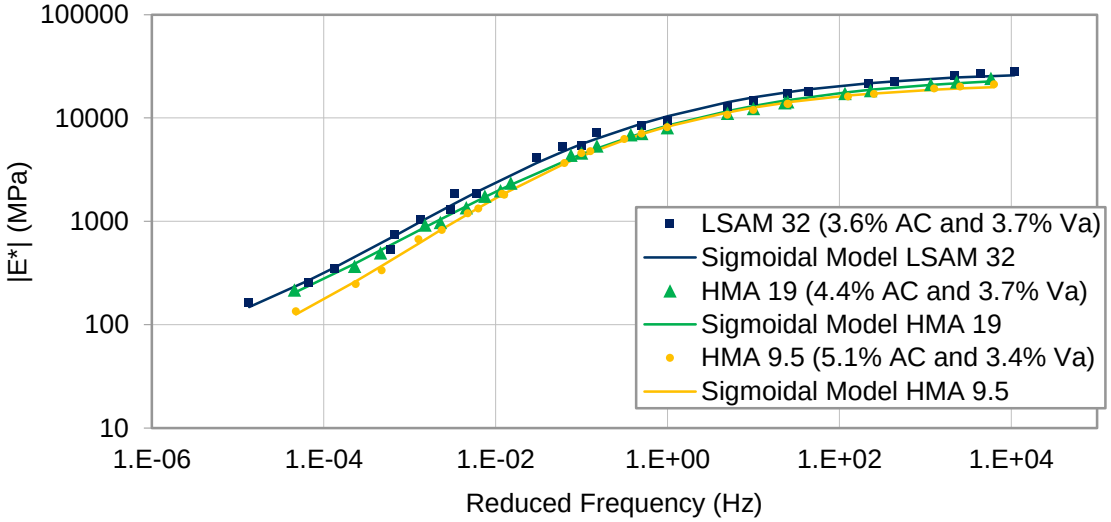
β and γ are shape parameters.

Figure 13 - Dynamic modulus test results (mastercurves in semi-log scale, T_R = 21.1°C)



Source: Author

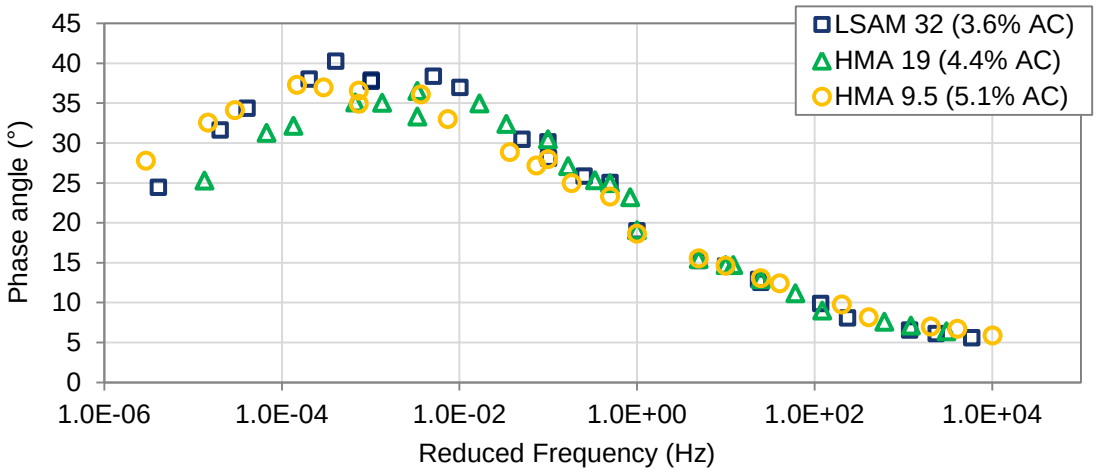
Figure 14 - Dynamic modulus test results (mastercurves in log-log scale, T_R = 21.1°C)



Source: Author

The dynamic modulus of the three mixtures with different nominal maximum aggregate has resulted in different mixture stiffness, where LSAM 32 has the highest modulus. Furthermore, the LSAM 32 presents 20 to 30% lower binder content when compared to the conventional mixtures with NMAS of 19 and 9.5 mm, which implies in the reduction of costs for the pavement rehabilitation using LSAM. The arrangement of aggregate skeleton in large stone mixtures and the lower asphalt content provide a large influence of the aggregate response under dynamic loading, maintaining a convenient stiffness when compared to the conventional HMAs. The similar viscoelastic behavior of the tested asphalt mixtures was also observed from the phase angle as presented in Figure 15.

Figure 15 - Phase angles of the asphalt mixtures

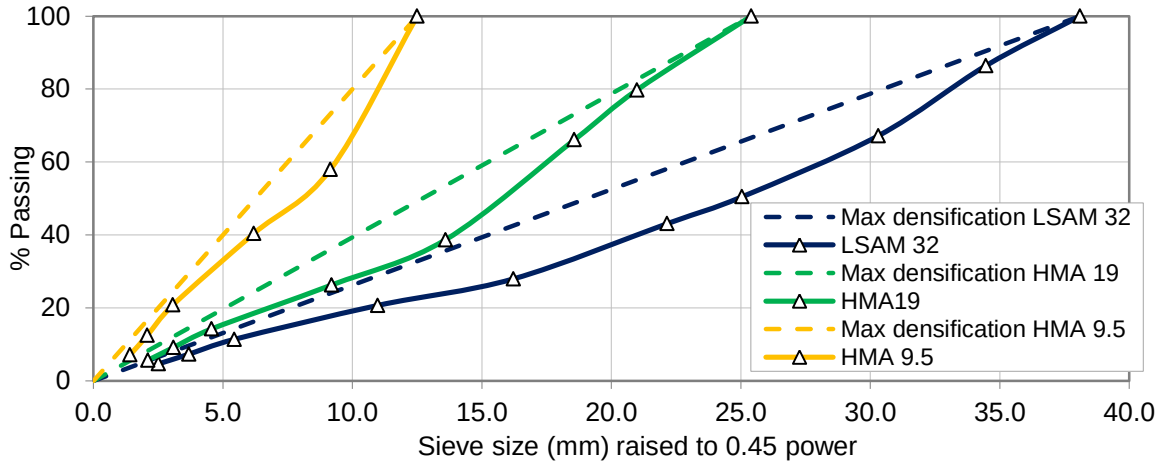


Source: Author

The mixtures phase angles present a discontinuous master curve, which values decreased as the frequency increased, corresponding to the test at low temperatures. With frequency decreases, there is a change in the curve trend where the phase angle increases up to a reduced frequency range of 1.0E-02 Hz to 1.0E-04 Hz, and it starts to decrease. The complex behavior of the phase angle at lower frequencies or higher temperatures is justified by the lower binder viscosity and the predominant influence of the aggregate interlock in the asphalt concrete response.

The aggregate gradation could affect many important properties of a HMA, as the stiffness. Gradations of maximum density could produce an asphalt mixture with low VMA, resulting a less durable mixture and also sensitive to asphalt content variations in the field (ROBERTS et al., 1996). Figure 16 presents the evaluation of the maximum density line for all three asphalt mixtures, according to Asphalt Institute (2001).

Figure 16 - Mixtures maximum density line



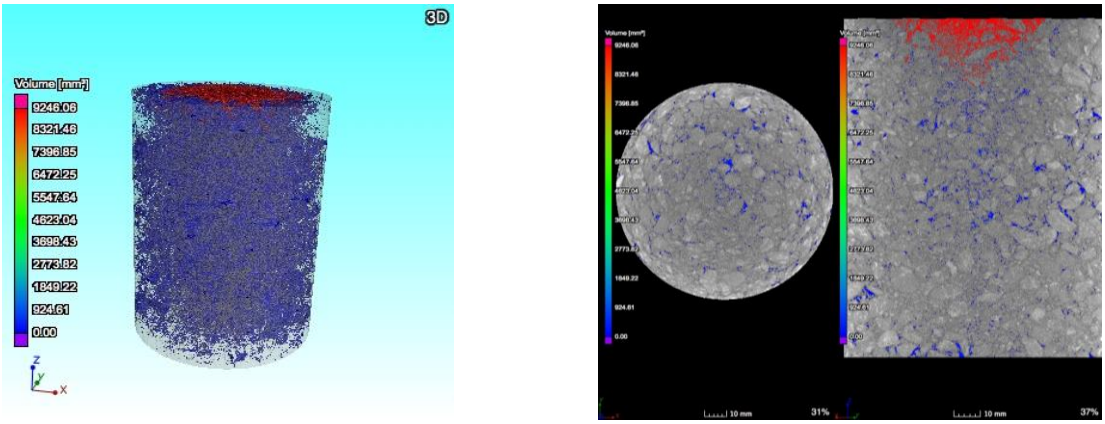
Source: Author

From Figure 12, one may observe that although the difference in the NMAS of the mixtures, the gradations are similar with respect to its maximum density line, what might be related to the similar dynamic modulus results. The three mixtures gradations are presented below the corresponding maximum density line, as some American state specifications require (ROBERTS et al., 1996), but not in close proximity.

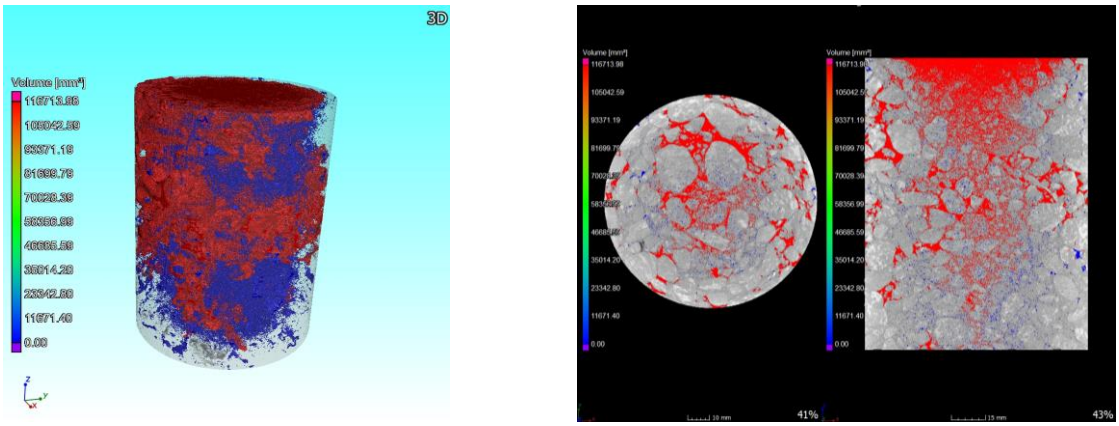
Computed Tomography

The computed tomography is a non-destructive technique which has gained increasing applications in study of asphalt materials (GATCHALIAN, 2006; MASAD et al., 2002; ALVAREZ-LUGO et al., 2014). The acquisition of digital images makes possible the evaluation of HMA internal structure from specimens compacted in the field or in laboratory. At this research, the computed tomography was performed in only one cylindrical specimen of each asphalt mixture applied in rehabilitated test sections, which images are presented in Figure 17.

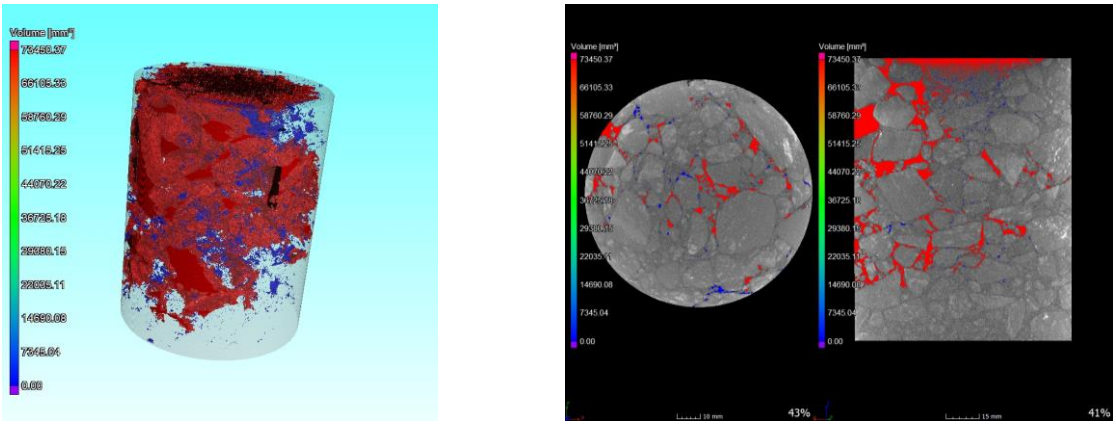
Figure 17 - Digital images of asphalt mixture specimens



(a) HMA 9.5



(b) HMA 19

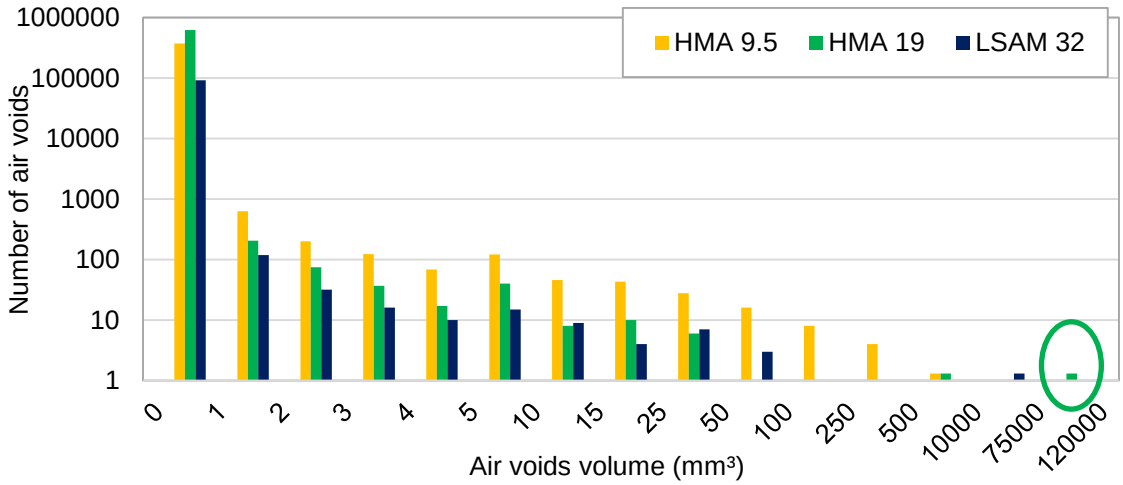


(c) LSAM 32

Source: Author

The distribution of air voids sizes over the specimens can be illustrated by the color scale presented in the left side of the images. The LSAM 32 and HMA 19 presents the larger air voids as the red scale is more pronounced, corresponding to the voids of greater volume. The HMA 9.5, even presenting larger amount of air voids in the volume range 0 to 50 mm³ compared to the other mixtures, presented lower connectivity between air voids when compared to the coarser mixtures (LSAM 32 and HMA 19). Figure 18 presents the results of mixtures porosity from the three dimensional analyses of specimens, illustrating the number of air voids distribution according to its volume range.

Figure 18 - Number of air voids regarding the range of volume



Source: Author

The analysis of mixture porosity allowed the determination of higher number of air voids concentration at 0 to 1 mm³ for every mixture, which is considerably higher for

fine mixtures. The same trend is maintained for volumes up to 25 mm³. The first perception from Figure 18 is that HMA 9.5 mm has voids with larger volume when compared to the other mixes. However, the interpretation of voids with volume from 500 to 120,000 shows the presence of really larger voids in HMA 19 and LSAM 32 which correspond to an interconnected group of voids detected like only one.

After specimens' tomography, the total volume of air voids was considered of great importance for comprehension of voids volumes and its distribution into the asphalt mixture, beyond the connectivity between them. Table 12 presents the air voids content, the number of air voids detected in the data processed from tomography and also the largest air void for each mixture.

Table 12 - Air voids contents of mixtures from tomography analysis

Mixture	Air voids (%)	Number of air voids detected	Largest air void (mm^3)
HMA 9.5	2.6	377,179	9,246
HMA 19	12.8	623,741	116,713
LSAM 32	7.5	92,105	73,450

Even if the mixtures had been designed to 4% of air voids, the air content has large variation between the specimens. The HMA 19 presents higher value, probably due to an interconnected void which was detected by the software with volume between 75,000 and 120,000 mm³ (point indicated in Figure 18). A simulation was done for HMA 19 without this specific point, where was observed a considerable change with: (i) air voids content = 1.3%; (ii) number of air voids detected = 623,740; and (iii) largest air void = 13,612mm³. The LSAM 32, even with intermediate volume of air voids (7.5%), is the one with lower number of air voids similar to the reported by NCHRP (1997) and found by Alvarez-Lugo et al. (2014) at his tomography and images analyses.

3.4.2. NMAS Influence on Fatigue Life

3.4.2.1. Flexural Bending Fatigue Test

For advanced design procedures of asphalt pavements is common to consider the fatigue cracking as one of the most significant failure mechanism of asphalt mixture, which occurs in a medium to long term. Zhang and Bai (2008) investigated in laboratory the performance of LSAM designated as base course by means of flexural bending fatigue test. The test enables the determination of the “Endurance Limit” of the mixture when the failure of the specimen does not occur for very low strain levels (between 70 and 100 $\mu\varepsilon$).

The repeated flexural fatigue test is considered by some authors the one that provides a better simulation of field conditions (TANGELLA et al., 1990; COLPO, 2014). Verhaeghe et al. (1994) performed the third-point loading fatigue test at 5ºC to LSAM, also called Large Stone Base Mixtures (LAMBS), with maximum aggregate size of 37.5 mm considering strain-controlled and stress-controlled modes. The stress-controlled mode provided a better indication of number of load application to crack initiation. On the other hand, the strain-controlled mode better represented the crack propagation phase. Conclusions show that the LAMBS were less susceptible to horizontal tensile strain than conventional mixtures with NMAS up to 19 mm.

The fatigue life evaluation was done by the four-point flexural bending fatigue test. The test was performed by cyclic load application in the middle third of the rectangular specimen with 380 mm of length, 63 mm of width and 50 mm of height (Figure 19). The solicitation of asphalt mixture specimen occurs with vertical load application by two cleavers on middle third, where has a uniform moment distribution without shear stress (TANGELLA et al., 1990; COLPO, 2014). The standard procedure for flexural bending fatigue test is described in the ASTM D7460 (2010) specification and a controlled strain level was used.

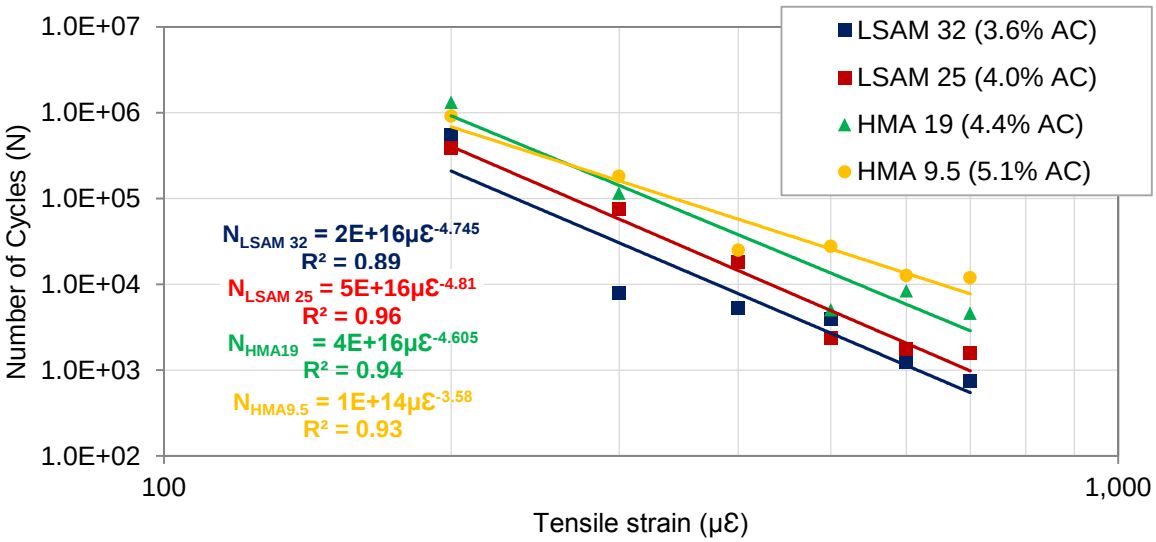
Figure 19 - Flexural bending fatigue test: (a) test equipment (b) tested beams

(a) (b)

Source: Author

Test is considered finished when the specimen exhibits a 40% reduction in stiffness compared to its initial stiffness, considered the failure point according to ASTM D7460 (2010) standard. The test temperature was 20ºC with frequency of 10Hz. Six strain levels were considered: 200, 300, 400, 500, 600, 700 microstrain for each mixture which was performed in four mixtures (LSAM 32, LSAM 25, HMA 19 and HMA 9.5) with 6 samples, compacted by LCPC rolling compactor. Figure 20 presents the fatigue curves, along with the fitting equations, for the mixtures with different NMAS. Both the asphalt binder content and the air voids have influence on the fatigue resistance of the asphalt mixtures. Because of the difficulty in keep both parameters equal to all the mixtures, it was decided to use the design binder content of each mixture in order to keep similar air voids.

Figure 20 - Flexural bending fatigue test results



Source: Author

The fatigue life of LSAMs was lower compared to the conventional HMA. As expected, the increase of binder content can reduce the fatigue deterioration of asphalt mixtures. However, the effect of NMAS and grading curve are significant variables comparing the fatigue performance of LSAM 32, LSAM 25, HMA 19 mm and HMA 9.5. The internal specimen structure by means of air voids size and distribution, along with the mode of loading can also influence the fatigue life of materials, but it was not possible in this study to evaluate its effect.

It was previously reported that LSAM (which is composed by a significant amount of coarse aggregates) can result in mixtures with fewer air voids, but of larger dimensions (NCHRP, 1997). ALVAREZ-LUGO, CARVAJAL-MUÑOZ, WALUBITA (2014) presented in their study an evaluation of air voids content and air voids size distribution for cylindrical specimens of large stone mixture with NMAS 25 mm using X-ray CT and image analysis. Comparison of investigated dense graded mixtures suggested that the one with 25 mm NMAS exhibits the largest air voids when compared to HMA with NMAS ranging from 9.5 to 19 mm. It may harm the fatigue life of LSAM materials due to the high stress concentration at this large air voids (HARVEY and TSAI, 1996) and the higher mixture permeability to air which increase the aging of the asphalt binder.

3.4.2.2. Tension-Compression Fatigue Test

The asphalt mixture fatigue performance in laboratory can also be characterized by a cyclic tension-compression test under stress-controlled or strain-controlled mode of loading. The test results can be modeled with the simplified viscoelastic continuum damage (S-VECD) model approach to obtain the damage characteristic curve which describes the deterioration of the material integrity, pseudo stiffness C, as the damage grows, S (NASCIMENTO, 2015; LEE, NOROUZI and KIM, 2016).

The uniaxial tension-compression tests were performed on hydraulic loading machine MTS 810 machine under strain-controlled mode of loading. Top and bottom of the cylindrical specimens (100 mm of diameter and 130 mm of height) were glued in steel plates to enable the application of a tension-compression sinusoidal load at

frequency of 10Hz and temperature of 21°C. The fatigue failure criterion is defined as the point where occurs the change in slope of the phase angle (LEE, NOROUZI and KIM, 2016).

Due to time constrains, only the LSAM with NMAS of 32 mm was considered for the tension-compression test, which also presented difficulties in characterizing its fatigue life by the flexural bending fatigue test. However, the LSAM specimens showed no resistance to tension at the beginning of the test for 300 and 250 microstrains, presenting microcracks that resulted in specimen rupture on the middle section (Figure 21).

Figure 21 - Specimens after attempt of tension-compression fatigue test

Source: Author

Large stone asphalt mixtures are hot mix designed with lower asphalt content due to the lower specific surface area of aggregates when compared to the conventional HMA, which has effect on the asphalt film thickness. Once the fatigue life is dependent on the asphalt content, aggregate gradation and, consequently, air voids distribution and their size, the LSAM should be evaluated by a reliable and repeatable laboratory fatigue test data considering lower strains levels and, maybe, specimens of representative volume with larger dimensions.

3.4.3. NMAS Influence on Permanent Deformation

Rutting is defined as the permanent deformation in pavement surface which appears as longitudinal depressions in the wheel paths under channelized traffic. This distress is most significant with the increase in traffic volume, moving loads and tire pressure

(AHLRICH, 1996). The WASHTO records described the densification and plastic flow as the main causes of mixture rutting (NCHRP, 1997). The densification is the consolidation of the pavement layers under traffic and the plastic flow occurs with the instability of the asphalt mixture which increases the susceptibility of longitudinal depression marks (AHLRICH, 1996).

Zaniewski and Nallamothu (2003) assert the permanent deformation as the most important parameter to be considered for LSAM evaluation. The asphalt mixture permanent deformation depends on the system binder-aggregate and the individual characteristics of each material that compounds it. The main points to be considered are the viscoelastic properties of asphalt binder, the size, form and nature of aggregates and the correct portion of each mixture component.

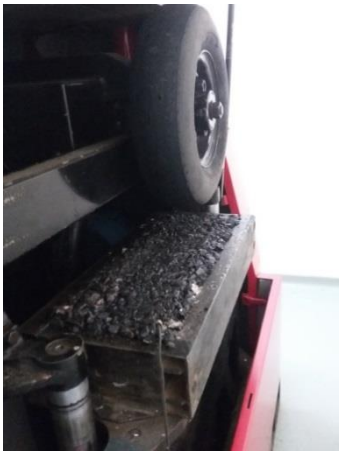
Roberts et al. (1996) present the aggregate gradation composition as relevant to workability, durability, stability, permeability, moisture resistance, adhesion, stiffness and fatigue life. In LSAM design, the grading curve should be selected to secure the load transmission through the aggregate skeleton where the larger aggregates participate actively in the load bearing capacity (NCHRP, 1997). The stone-on-stone contact between LSAM aggregates allows an efficient dissipation of compressive load and of shear stress which are known as responsible for rutting distress in flexible pavement (MAHBOUB and WILLIAMS, 1990).

Studies performed creep tests to evaluate the mixture susceptibility of LSAM to develop permanent deformation (plastic deformation). In general, the LSAM presented superior resistance to permanent deformation when compared to conventional HMA (BROWN and BASSETT, 1990; NCHRP, 1997). It has been concluded that the use of large stone mixtures is appropriated to pavements subjected to slow traffic, where the time of load application is longer (NCHRP, 1997).

The LSAM was characterized by means of the LCPC permanent deformation test (Figure 22), with the specimen prepared by the rolling compaction according to European specification EN 12697-33 (2003). The compacted slabs have 100 mm height and a base area of 500 mm × 180 mm which were submitted to the traffic simulator according to the specification EN 12697-22 (2004).

Figure 22 - LCPC permanent deformation equipament test

(a) LCPC traffic simulator

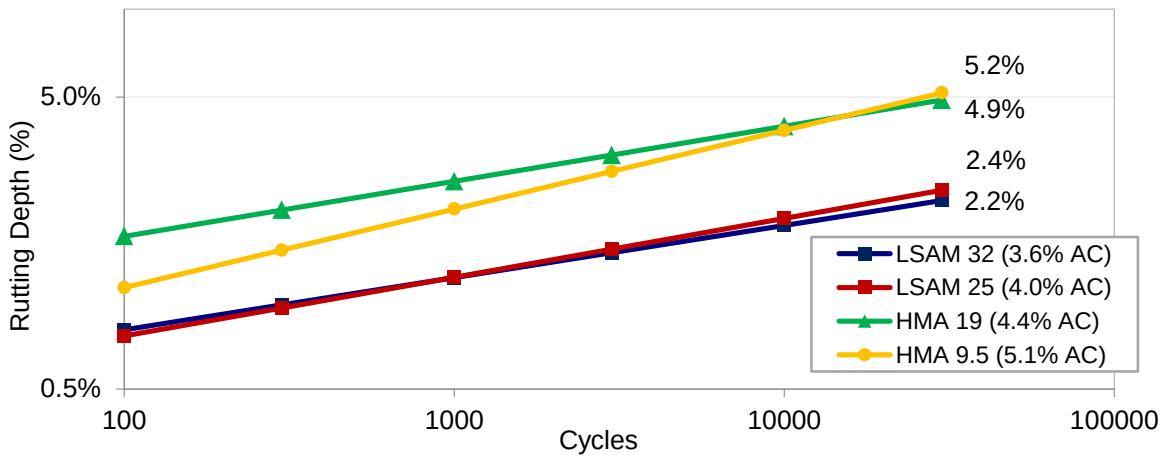


(b) Slab during test

Source: Author

The rutting depth is determined as a function of the number of cycles, performed with measurement in 15 points of loaded area over the slab when 0, 100, 300, 1000, 3000, 10000 e 30000 cycles are reached. The specification limits the maximum deformation for a selected number of cycles varying according to the type of mixture. The French evaluation parameter for LSAM (in French, “grave-bitume”) is a maximum deformation between 5.0% and 10.0% at 30,000 cycles, depending on the traffic level which mixture will be submitted, from heavy to duty-heavy traffic condition (LPC, 2007). The measured rutting depths for the LSAMs and for the conventional mixtures of NMAS of 19 mm and 9.5 mm, prepared in laboratory, are presented in Figure 23.

Figure 23 - LCPC permanent deformation test results



Source: Author

The results showed low rutting depth in general for LSAMs, below the specification limits of 5.0% for heavy traffic, indicating high resistance to permanent deformation. HMAs also had low rutting but this was twice the value of the LSAMs. The ranking of the mixtures presented an increase order of permanent deformation when the NMAS of mixtures decreases. The high temperature of LCPC permanent deformation test influences the decrease of asphalt binder viscosity and, consequently, the mixture strength to permanent deformation will rely mostly on the interlock of aggregate skeleton under moving load (OLIVEIRA FILHO, 2007). It is important to have an appropriate design, to assure a high direct stone-on-stone contact in the asphalt mixture that will create a good resistance to permanent deformation under traffic (NCHRP, 1997).

4. EXPERIMENTAL TEST SITE

4.1. Introduction

The increase of the freight transportation by roads on many Brazilian highways is a reality that demands an effective pavement structure. In order to ensure the quality and road safety, pavement rehabilitation techniques are becoming increasingly important as they can slow down the deterioration rate and avoid structural issues that might demand a pavement reconstruction. Efficient interventions are necessary, since frequent maintenance can be expensive and harmful for the road operation in a long-term analysis.

The use of Large Stone Asphalt Mixtures (LSAM) is a promising solution to improve rutting resistance for roads demanded by very heavy-duty trucks without other structural problems. LSAM is a dense hot mix asphalt concrete composed by a high portion of large aggregates, with nominal maximum aggregate size (NMAS) starting at 25 mm (MAHBOUB, 1990; KANDHAL, 1990; NCHRP, 1997; USACE, 2000).

This type of mixture gained popularity in some of the North American districts, due to its high resistance to permanent deformation, when applied as base or leveling course (ZANIEWSKI and NALLAMOTHU, 2003). However, the grading composition has to ensure the transmission of load stresses through the aggregate skeleton structure. Aggregate interlocking should be between the coarse aggregates with good stone-on-stone contact, capable to carrying the traffic load (NCHRP, 1997).

This chapter presents a brief literature review of LSAM constructive concerns and also the characterization of rehabilitated test sections in Brazil with the use of LSAM technology. The construction process is described and the quality control was done to certify the conformity with the rehabilitation project. Falling Weight Deflectometer (FWD) campaigns were done during the construction process and along 18 months of monitoring the pavement structure. The surface of the rehabilitated pavement was also monitored and characterized in terms of rutting.

4.2. Constructive Concerns on the Use of LSAM

The construction of pavements with LSAM in the structure is considered a difficult assignment for many agencies since its primary use. According to Mahboub and Williams (1990), the mixtures with large aggregate size are more sensitive to errors during the construction process when compared to conventional mixtures. Attention should be devoted to plant mix production, transportation, application and compaction for reducing the probability of segregation and aggregate breakage due to the large dimensions of aggregates.

Other constructive difficulty observed in the 90 decade was the workability of LSAM, once there was not appropriate equipments for an effective compaction (NCHRP, 1997; FERNANDO, BUTTON and CROCKFORD, 1997). The complete coating of coarse aggregates by asphalt binder is necessary to facilitate the compaction process. Longer time of plant mixing and thicker film of asphalt enables the lubrication among stones during compaction (NEWCOMB et al., 1993). USACE (2000) reported that some additional equipment wear may occur during the plant mix of large stone asphalt mixtures and during the paver application in field. Also, the compaction effort for LSAM should be monitored to prevent the aggregates breakage, since those with large dimensions becomes more sensitivity to fracture.

4.2.1. Segregation

The segregation is defined as the heterogeneous distribution of coarse aggregates into the asphalt mixture volume, occurring commonly in dense mixtures (NCHRP, 1997). At laboratory, Airey and Collop (2014) noted that segregation in cylindrical specimens could be evaluated as radial and regional means (points without specific location). Methods of laboratory and field mix compaction were analyzed including gyratory, vibratory and rolling compaction resulting in verification of various levels of particles segregation. The gyratory and vibratory compaction presented a higher rate of radial segregation compared to the rolling compacted specimens, but not higher as the field compaction. The regional segregation has appeared more in gyratory and vibratory compacted specimens than in the rolling compacted specimens and real field compaction.

The high concentration of coarse aggregates in an asphalt mixture may increase the probability of segregation, turning difficult the correct mixture densification. Emery (1996) describes the segregation as a common problem for aggregate gradations such as an open graded LSAM. Researchers presented experiences with construction of test sections, highlighting problems as significant open surface texture of segregated area when compared to the rest of the layer. These areas have low capacity of load support and are sensitive to disintegrate when subjected to traffic (NCHRP, 1997).

The segregation could be minimized with appropriate monitoring during construction. The use of uniform aggregate gradation, suitable stockpiling, proper truck loading and transportation are necessary to reduce the segregation of LSAM (KANDHAL, 1990; EMERY, 1996; GELDENHUYS, 2011).

4.2.2. Aggregate Breakage

The aggregate breakage is possible during plant mixture and compaction of large stone asphalt mixture layers. During the drying and the mixing process inside the drum mixer, the coarse aggregates are susceptible to rupture depending on the quality, hardness and angularity of the stone particles. If aggregate breakage occurs, the mixture grading curve will be different from the original project. It could result in an inefficient aggregate interlocking, affecting the permanent deformation resistance of LSAM and the performance of the pavement (PRADHAN, 1995; NCHRP, 1997).

However, the aggregate breakage could also occur during the compaction process upon the roller compactor. To prevent this to happen, NAPA (2002) suggested the placement of LSAM in thick enough lifts to allow an adequate compaction and avoid excessive aggregate breakage. Also, the vibratory rollers can be an issue for compaction of large aggregate mixtures which will depend on aggregates gradation, hardness of coarse aggregate, layer thickness and roller speed.

4.3. Rehabilitation Project Description

The LSAM technique was applied as the leveling course for the rehabilitation of two pavement sections in Brazil, where rutting was the main distress. The trial sections were on the third right lane, due to the heavy trucks traveling at low speed on slope. Different techniques have already been applied on the surface course of this segment (Table 13), varying from dense mixtures (NMAS 9.5 or 12.5 mm) with conventional AC with 30/45 penetration grade or modified asphalt cement (MAC - Rubber asphalt, 4% of SBS or 1.2% of RET) to Gap Graded with AC 30/45 or Rubber asphalt binder, without success to combat permanent deformation, which appeared in less than a year of service (MOURA, 2010; NEGRÃO, 2012).

Moura (2010) reported high rutting values in 2 kilometers of rehabilitated sections which were executed in January 2009. Measurements of 10 centimeters were reached 13 months after the opening to traffic (Figure 24).

Table 13 - Previous rehabilitation techniques used at experimental test site

Section	MOURA (2010)		NEGRÃO (2012)	
	Pavement rehabilitation in January 2009 (50 mm of milled surface layer)		Pavement rehabilitation in July 2010 (50 mm of milled surface layer)	
	Binder	Grading curves	Binder	Grading curves
1	AC 50/70	Asphalt Institute IVb	Rubber AC	Gap-graded
2	AC 50/70	Superpave NMAS 12.5 mm		
3	MAC with 4.0% of SBS	Superpave NMAS 12.5 mm		
4	MAC with 4.0% of SBS	Superpave NMAS 9.5 mm		
5	MAC with 1.2% of RET	Superpave NMAS 9.5 mm	MAC with SBS	Superpave NMAS 12.5 mm
6	MAC with 1.2% of RET	Superpave NMAS 12.5 mm		
7	Rubber AC	Gap-graded		
8	AC 30/45	Superpave NMAS 12.5 mm	MAC with RET	Superpave NMAS 12.5 mm
9	AC 30/45	Superpave NMAS 9.5 mm		
10	AC 30/45	Gap-graded		

Source: Author

Figure 24 - After 13 months of pavement rehabilitation executed in January 2009

(a) Rubber AC + Gap graded

(b) AC 30/45 + Superpave NMAS 12.5 mm

(c) AC 30/45 + Superpave NMAS 9.5 mm

(d) AC 30/45 + Gap graded

Source: Moura (2010)

The rutting measurements exceeded the limit of 7 mm from technical guidelines for Brazilian highway concessions, specified by National Land Transportation Agency (in Portuguese, Agência Nacional de Transportes Terrestres, ANTT). Thus, new pavement rehabilitation was necessary to repair the surface course and ensure the security and comfort for the road users. Negrão (2012) described in his research the rehabilitation techniques adopted for the same sections in July 2010, that also presented rutting after 6 months of service life (Figure 25).

Figure 25 - After 6 months of pavement rehabilitation executed in July 2010

(a) Rubber AC + Gap graded

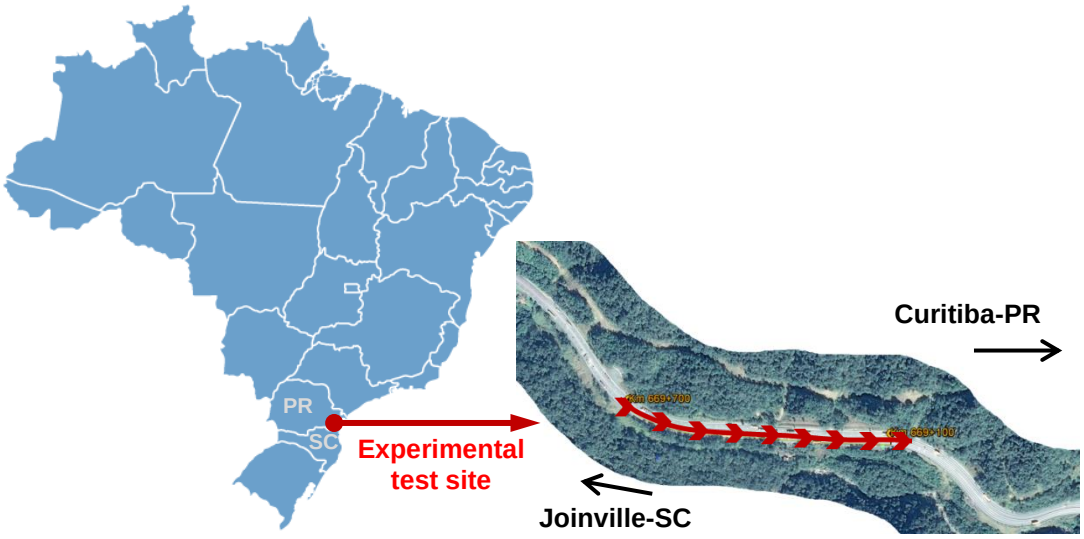
(b) SBS AC + Superpave NMAS 12.5 mm

(c) RET AC + Superpave NMAS 12.5 mm

Source: Negrão (2012)

After the summer months (December and January), high wheel path depressions were presented again in some of the rehabilitated pavement sections. The sections are located at the BR-376 highway (Figure 26), in the south region of the country (City of Guaratuba, state of Paraná).

Figure 26 - Location of the test section site in Brazil



Source: Author

An evaluation of the previously existing pavement showed low deflections and good structural conditions, indicating that the pavement suffered only plastic deformation on the surface, without compromising the underlying layers. Thus, pavement rehabilitation could be a simple solution avoiding future needs of a pavement reconstruction. The surface characteristic of the selected sections before the proposed rehabilitation strategy using the LSAM is presented in Figure 27. The structure before rehabilitation is described in Table 14 considering the investigation in different points along the experimental test site.

Figure 27 - Test site selected for rehabilitation with LSAM

Source: Author

Table 14 - Existing pavement structure before pavement rehabilitation

Point 1		Point 2	
Material	Thickness (cm)	Material	Thickness (cm)
Hot mix asphalt	24	Hot mix asphalt	20
Crushed stone	13	Cold asphalt mixture	9
Cement treated crushed stone	26	Gravel	50
Crushed stone	13	Coare crushed aggregates	20
Subgrade	∞	Subgrade	∞

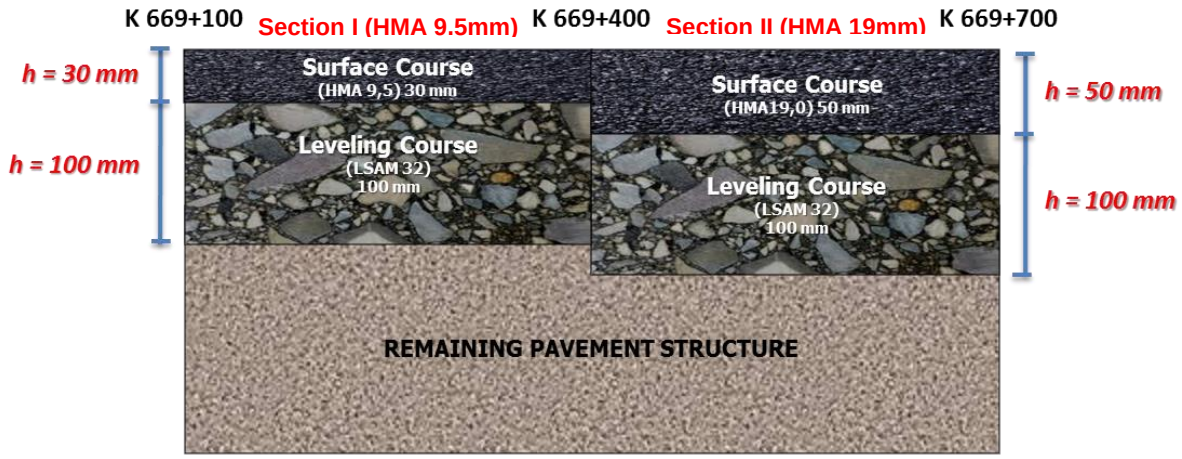
Source: Author

The selected road sections presented severe permanent deformation in the wheel paths due to the high traffic intensity, low vehicle speed and upward slope. Different pavement structures were shown in the pavement investigation before rehabilitation, which implies in a high heterogeneity along the experimental test site.

The rehabilitation was designed in two pavement configurations over 600 m length. This was the first reported Brazilian rehabilitation experience using LSAM. The first

structure was composed of a hot mix asphalt (HMA) with NMAS of 9.5 mm as surface course and LSAM with NMAS of 32 mm as leveling course, and the second pavement structure was composed of an HMA with NMAS of 19 mm as surface course and the same LSAM of the first segment as leveling course. An illustration of the pavement structures is presented in Figure 28.

Figure 28 - Configuration of the test section



Source: Author

The rehabilitation project contemplated the milling of the original pavement in two different depths (13 cm and 15 cm), followed by the application of 10 cm of LSAM as the leveling course. The asphalt mixtures used on the surface layers were necessary due to the coarse texture of the LSAM. The HMA with NMAS 9.5 mm was applied from Km 669.100 to Km 669.400 with 30 mm, and the HMA with NMAS 19 mm was applied from Km 669.400 to K 669.700.

The knowledge of the remaining pavement structure and its conditions immediately after the pavement rehabilitation is an important information for evaluation of the pavement performance over time. The structural characterization by the FWD test was performed after: (i) the pavement milling, (ii) the LSAM leveling course application, and (iii) the surface course application considered as the first deflection data to compare with the following deflection measurements over time.

4.4. Test Site Construction

The test sections were constructed in four days (from June 14th to 18th of 2016) in normal climatic conditions without rain. The quality control and quality assurance was performed during the plant mix production, the paving and the compaction activities. The plant was a drum-mixer located at SC-415 (38 kilometers away from the test sections).

Large stone mixtures present higher potential of segregation compared to the conventional HMA due to its higher amount of coarse aggregates (NCHRP, 1997). For that reason, the materials stockpile, plant and handling procedures were done with care, in order to minimize segregation. The truck loading operation was done in four steps, decreasing the slipping of larger aggregates to the bottom of the trucks. Figure 29 illustrates some initial steps of the rehabilitation procedure.

Figure 29 - Construction steps of the test sections

(a) Pavement milling

(b) Cleaning

(c) Paint binder

(d) LSAM paving

(e) Temperature control of LSAM

(f) Compaction Pneumatic Roller

Source: Author

The construction started with lane 3 closing in BR-376 Northbound from Km 669+700 to Km 669+100 at 8 PM on the June 14th. Then, the existing pavement was milled at 150 mm depth in the first 300 m, and the other 300 m were milled at 130 mm depth.

At the bottom of the milled surface there was a remaining HMA course, what was expected based on the previous evaluation.

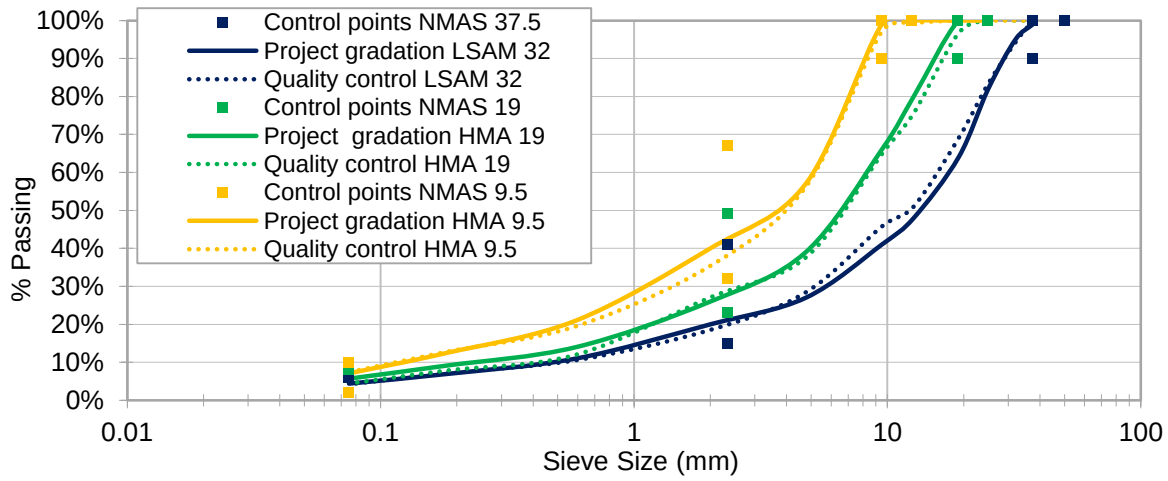
After milling, the surface of the remaining infrastructure was cleaned, and tack coat was applied before the LSAM leveling course application. The tack coat material was a cationic rapid setting emulsion, applied with a spreader truck. The LSAM layer was placed using a paver and compacted with a pneumatic roller. The mixture compaction temperature was the one established on the asphalt mixture design (150°C). The surface courses were constructed in each section using the same procedures as the LSAM leveling course.

4.4.1. Materials

4.4.1.1. Aggregates and Mixture Gradations

The aggregates were granite crushed stone from Itapoá quarry, districts next to the city of Joinville (state of Santa Catarina), in agreement with the specifications from the Brazilian National Department of Transportation Infrastructure (DNIT). The final aggregate gradations selected for the asphalt mixtures (LSAM 32, HMA 19 and HMA 9.5, showed in Chapter 3) followed the Superpave specifications from Asphalt Institute (2001), as shown in Figure 30.

Figure 30 - Aggregate gradations for test section mixtures



Source: Author

4.4.1.2. Asphalt Binder

A neat asphalt binder with penetration grade 30/45 (AC 30/45) and Performance grade PG 58V-XX was used in this study. The Table 15 summarizes the binder's physical properties.

Table 15 - Physical properties of neat asphalt cement

Property		ASTM standard	Requirements	Units	AC 30/45
Penetration		D 5	30 to 45	0.1 mm	34
Density		-	-	-	1.007
Softening Point		D 36	> 52	°C	52.6
Solubility in Trichloroethylene		D 2042	> 99.5	% mass	99.9
Brookfield Viscosity*	135°C	D 4402		cP	425
	150°C	D 4402		cP	210
	177°C	D 4402	76 to 285	cP	76

Source: Author
* Spindle 21 and 20 rpm

Once the permanent deformation is one of the main distress considered for pavement rehabilitation to this study, the asphalt binder was characterized, after aging in Rolling Thin Film Oven Test (RTFOT) according to ASTM D2872 (2012), by means of Multiple Stress Creep and Recovery (MSCR) test according to the standard ASTM D7405 (2015). The test considers 1 second of flow and 9 seconds of recovery and the material deformations are monitored. 20 cycles are applied under 0.1 kPa stress which the first 10 for conditional the sample and the other 10 cycles under 3.2 kPa of stress.

The results enable calculate the non-recoverable creep compliance (J_{nr}), which is an indicator of permanent deformation. The elastic recovery (R%), the ability of asphalt binder to recovery the elastic deformation suffered due to applied load, is also possible to be calculated. The test was performed at temperatures of 52°C, 58°C, 64°C and 70°C presenting results with low values for the R% and $J_{nr,diff}$ (sensibility to stress variation), which is common for non-modified asphalt binders (Table 16).

Table 16 - MSCR results for asphalt binder after RTFOT

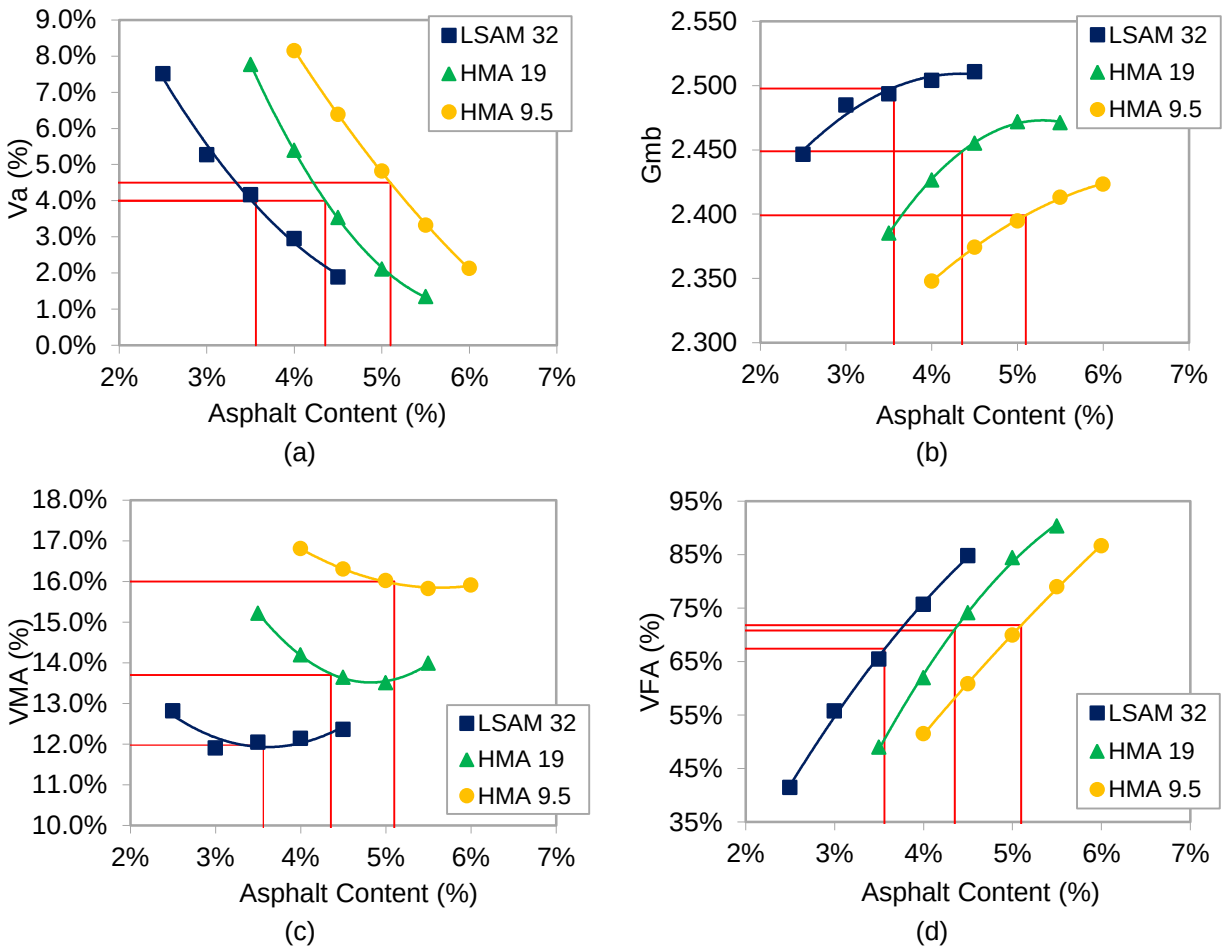
Temperature (°C)	$J_{nr\ 0,1}$ (kPa^{-1})	$R_{\%\ 0,1}$ (%)	$J_{nr\ 3,2}$ (kPa^{-1})	$R_{\%\ 3,2}$ (%)	$J_{nr,diff}$ (%)
52	0.294	9.3	0.307	6.4	4.6
58	0.909	1.5	0.943	0.0	3.7
64	2.005	0.0	2.129	0.0	6.2

Source: Author

4.4.1.3. Asphalt Mixture Design

All mixtures designs were performed using the Superpave Gyratory Compactor with 100 gyrations, as previous evaluated (BEJA et al., 2015), using samples of 150 mm in diameter and about 115 mm in height. The volumetric properties presented in Figure 31 were the parameters used for determining the design binder content.

Figure 31 - Volumetric properties of asphalt mixtures used in the test sections: (a) Air voids (Va), (b) Bulk specific gravity (Gmb), (c) Voids in mineral aggregate (VMA) and (d) Voids filled with asphalt (VFA)



Source: Author

The design binder content changed according to the mix gradation for the selected energy compaction level in the Superpave mix design. The Superpave volumetric requirements (AASHTO, 2013) for this design procedure, along with the project properties obtained, are presented in Table 17. All mixtures are in agreement with the requirements.

Table 17 - Volumetric properties and Superpave requirements

Property		Mixtures		
		LSAM 32	HMA 19	HMA 9.5
Air Voids (%)	Project	4.0	4.0	4.5
	Spec. Limits	3-5	3-5	3-5
Design Binder Content (%)	Project	3.6	4.4	5.1
	Spec. Limits	-	-	-
Voids in Mineral Aggregate (%)	Project	12.0	14.0	16.0
	Spec. Limits	>11.0	>13.0	>15.0
Voids Full of Asphalt (%)	Project	68.0	71.0	72.0
	Spec. Limits	65-75	65-75	65-75

Source: Author

4.4.2. Quality Control and Quality Assurance (QC / QA)

The QC / QA of constructed layers was done starting from comparing project data and evaluation of mixtures samples extracted during pavement rehabilitation. The parameters used were the grading curves, the asphalt content, the volumetric properties (Va, VMA and VFA), the bulk specific gravity of asphalt concrete (Gmb), the maximum specific gravity of asphalt concrete (Gmm), and the effective specific gravity of aggregates mixture (Gse). The comparison between the mixtures parameters determined in the project and obtained in the field are presented in Table 18.

The LSAM asphalt content applied in the test sections was satisfactory with variation of 0.1%, considered low and an acceptable limit. The mixtures are also in accordance with the volumetric design requirements, besides maintaining the grain size distribution curve within the working range (Figure 30). The same was observed

for the two HMA surface mixtures which have a permissible variation of ±0.2% in asphalt content. HMA volumetric properties were in accordance with Brazilian asphalt concrete specifications (DNIT 031/2004) and Superpave volumetric requirements, although the HMA 19mm presented the lower Vv compared to the other two mixtures.

Table 18 - QC/QA of asphalt mixtures applied in the trial section

	Asphalt content (%)	Gmb	Gmm	Va (%)	VMA (%)	VFA (%)	Density AC 30/45	Gse
LSAM 32								
Project Design	3.6	2.498	2.599	3.9	12	68	1.009	2.762
QC	3.7	2.456	2.571	4.5	14	67	1.009	2.736
HMA 19								
Project Design	4.4	2.449	2.551	4.0	14	71	1	2.741
QC	4.2	2.473	2.555	3.2	13	75	1	2.738
HMA 9.5								
Project Design	5.1	2.399	2.512	4.5	16	72	1	2.734
QC	5.3	2.388	2.507	4.7	17	71	1	2.732

Source: Author

4.5. Field Monitoring

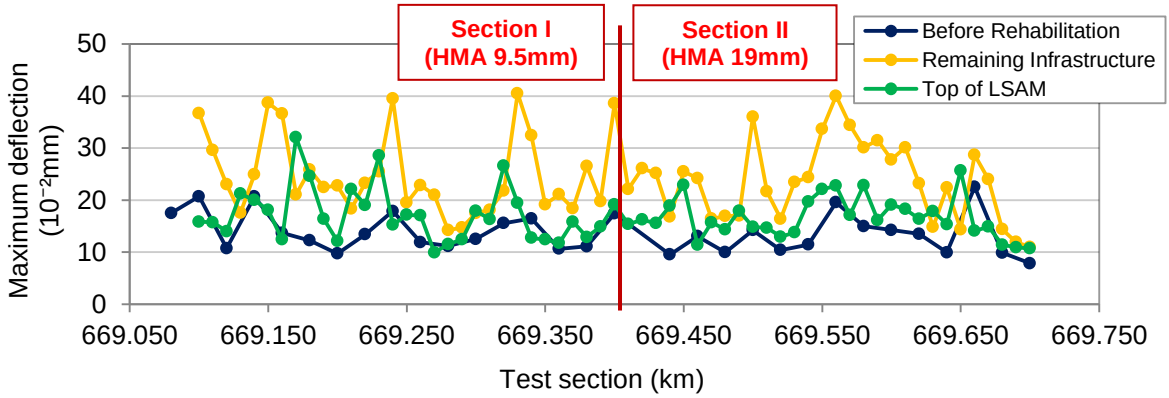
4.5.1. Structural Analysis

The pavement structure was characterized by non-destructive testing using the FWD equipment model Dynatest 8000E that simulates the passage of a vehicle load and measures the deflection on the pavement surface. In this research, the rehabilitated pavement structure was monitored through a period of one year with five campaigns of FWD tests, using the standard load of 41 kN and a 300 mm diameter plate.

During the pavement rehabilitation, FWD test was performed at the external wheel path and at the lane axis two times: (i) on the top of the milled surface (remaining infrastructure) every 10 meter spacing, and (ii) on top of LSAM leveling course, also every 10 meter spacing. The FWD test was not done on the top of the surface course due to operational restrictions in closing lanes during the weekends.

The maximum deflections obtained during the construction were monitored in three stages using a standard load of 41 kN in the outer wheel path: (i) before rehabilitation every 20 meters spacing (ii) after pavement milling every 10 meter spacing, (iii) after construction of LSAM leveling course every 10 meter spacing. Figure 32 shows the maximum deflections in the construction monitoring period.

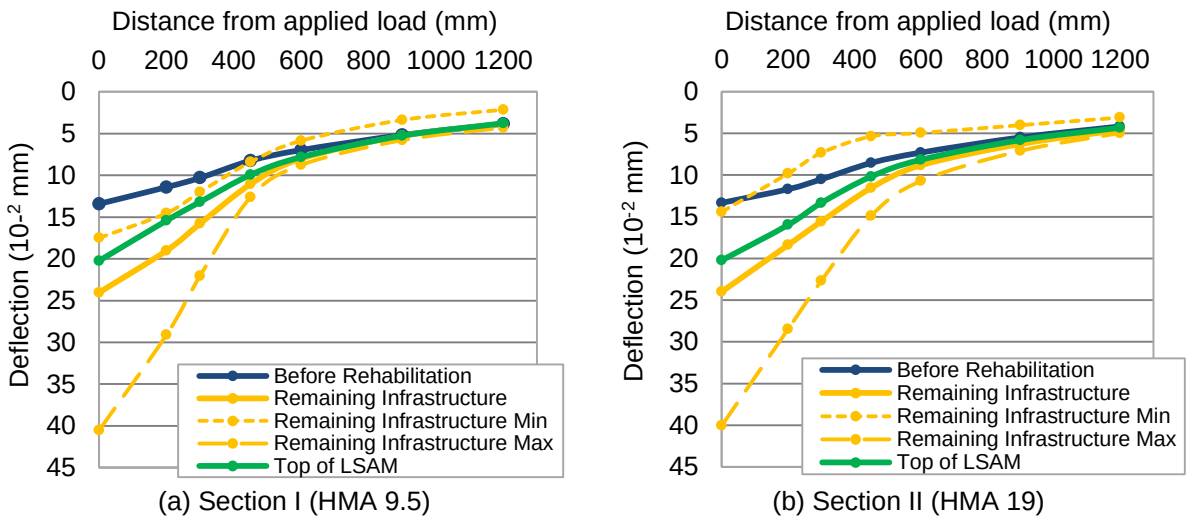
Figure 32 - Maximum deflections during rehabilitation procedure



Source: Author

Low deflections could be observed before the pavement rehabilitation, suggesting that the existing permanent deformation did not result from structural problems, such as the densification or shear rupture of underlying pavement layers, as described by Bernucci et al. (2010). It was presented that the existing rutting was caused by plastic flow of the surface course material, which could be solved by changing this material in the new rehabilitation project. Figure 33 presents the average of deflection bowl for this construction monitoring period.

Figure 33 - Average deflection bowls measure during rehabilitation



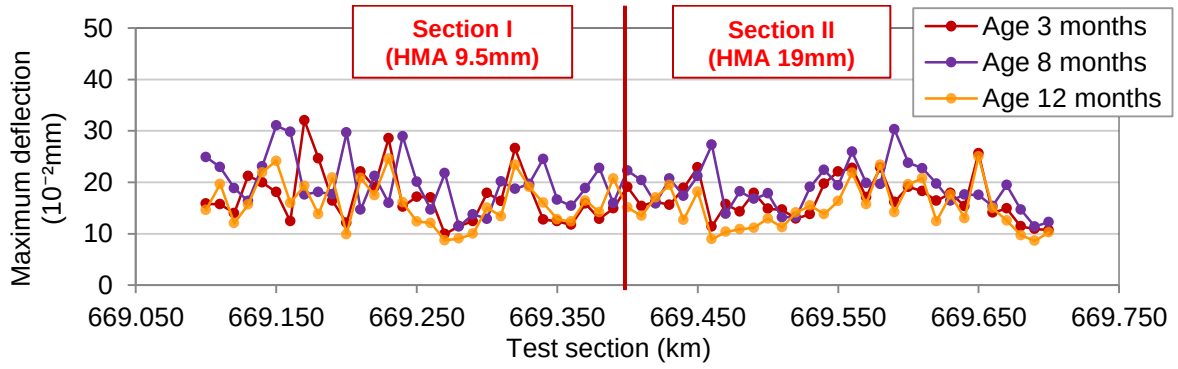
Source: Author

After application of LSAM 32 mm as leveling course, deflections were reduced when compared to the deflections on the remaining infrastructure after milling, since that some structural capacity was added back and, as expected, the pavement bearing capacity increased. The analysis of the pavement responses can be done considering several elements, and not only the maximum deflection. Thus, the importance of the deflection bowls (BALBO, 2007).

The average deflection values changed mostly in points closer to the center of load plate (0 mm), which can be verified comparing the data collected on the remaining infrastructure (after milling) and the data obtained after construction of LSAM layer. This difference is observed in lower intensity at distances larger than 0.5 meters from the applied load, which indicates a decrease in pavement deformability. The geometry of calculated average deflection bowls shows the improvement of load distribution over the pavement when the LSAM layer is constructed.

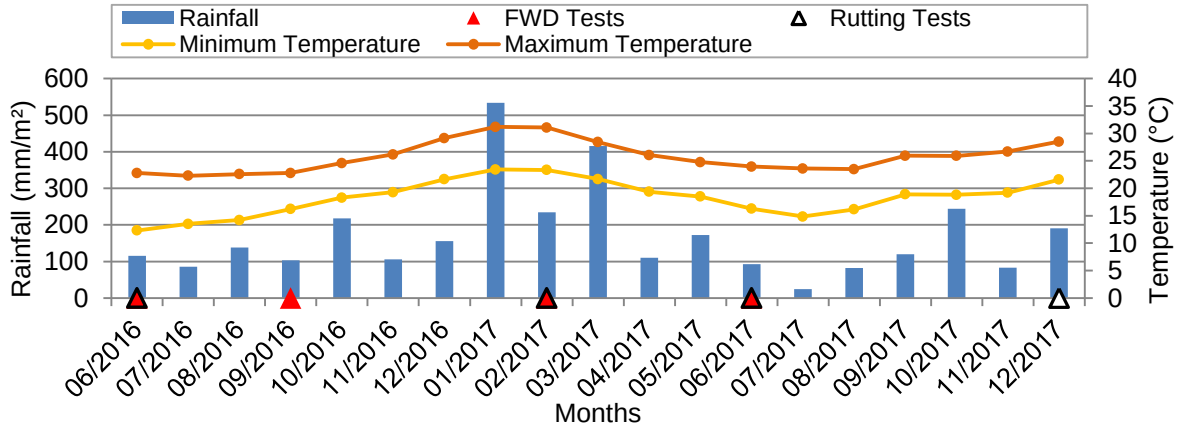
The other FWD campaigns were done 3, 8 and 12 months after the opening to the traffic, aiming to verify and evaluate pavement performance under real traffic and environmental conditions. The correspondent maximum deflections are presented in Figure 34. Climatic data were collected from the nearest weather station (City of Paranaguá, state of Paraná, at approximately 50km from the test site) to help the interpretation of the results, and are presented in Figure 35.

Figure 34 - Maximum deflections over time



Source: Author

Figure 35 - Climatic data from the nearest weather station

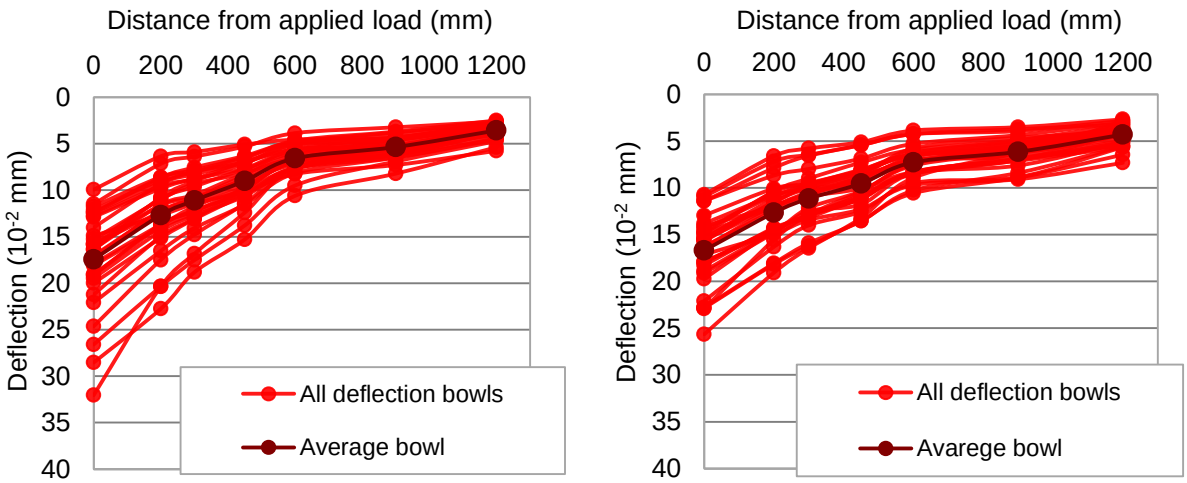


Source: Author

The maximum deflections in the four surveys ranged from 0.09 mm to 0.32 mm, what is considered reasonable for the pavement structure. It was not possible to identify a significant increase in the deflections over time, even though the 8 months survey showed higher deflections in some points, due the higher temperature and the possible water presence at the subgrade at the summer, and rainy, season (January and February). From these data, the pavement could be considered preserved during the analyzed period with respect to structural behavior.

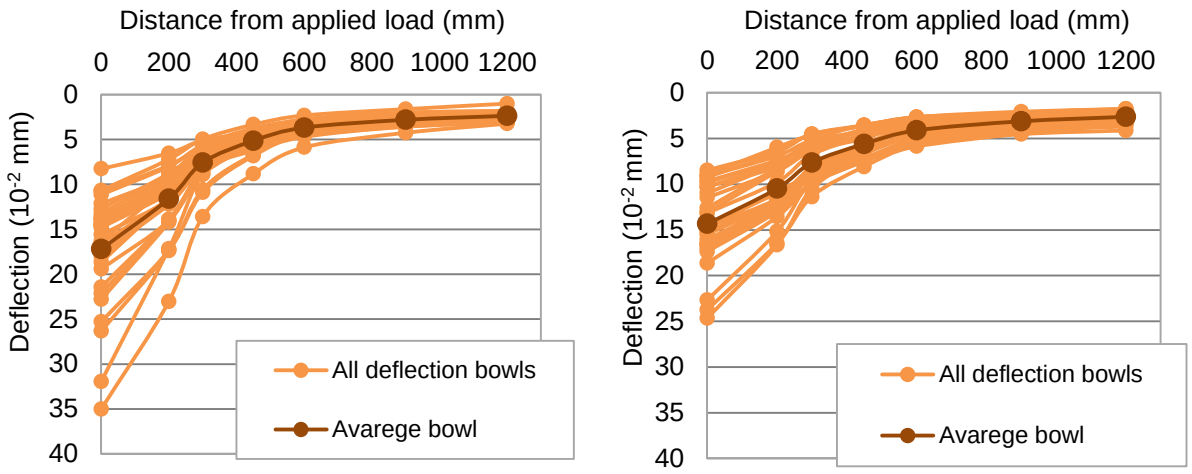
The deflections along the trial sections showed a high variability, because the remaining infrastructure is very heterogeneous. All the deflection bowls obtained were plotted and are shown in Figure 36, to illustrate this variability.

Figure 36 - Deflection bowls after 3 and 12 months



(a) Section I (HMA 9.5) after 3 months

(b) Section II (HMA 19) after 3 months

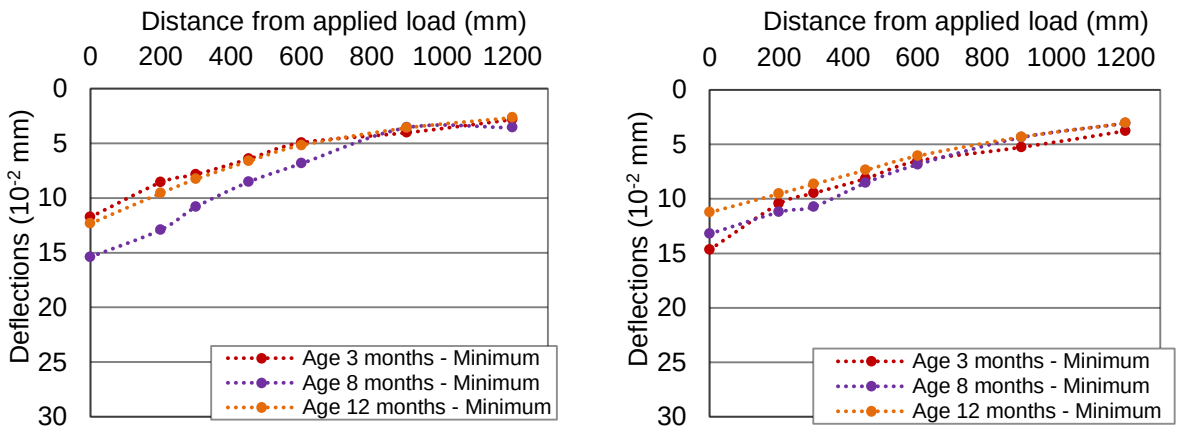


(c) Section I (HMA 9.5) after 12 months (d) Section II (HMA 19) after 12 months

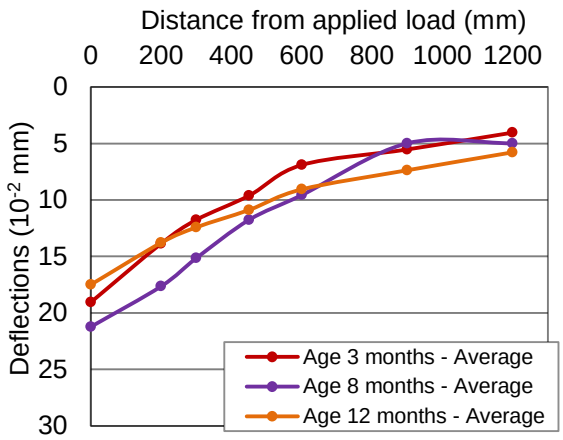
Source: Author

FWD controls obtained a large number of deflection bowls and was considered three of them at 3 months of survey to analyze the rehabilitated pavement performance in posterior ages, by means of determining the elastic modulus of LSAM over time. Three deflection bowls conditions (referring to the closest minimum, average and maximum) were selected to run the backcalculation. It was performed for different ages and for both test sites, as shown in Figure 37.

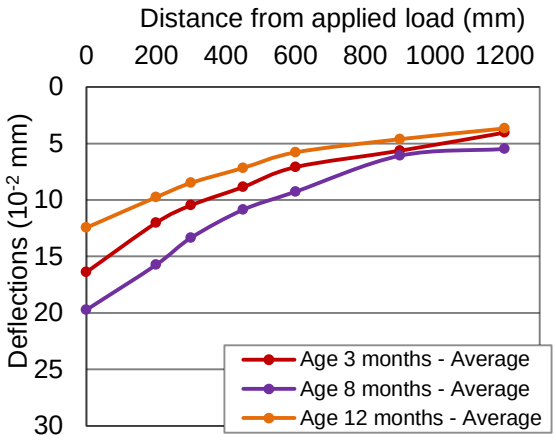
Figure 37 - The deflection bowls representative of the minimum, average and maximum



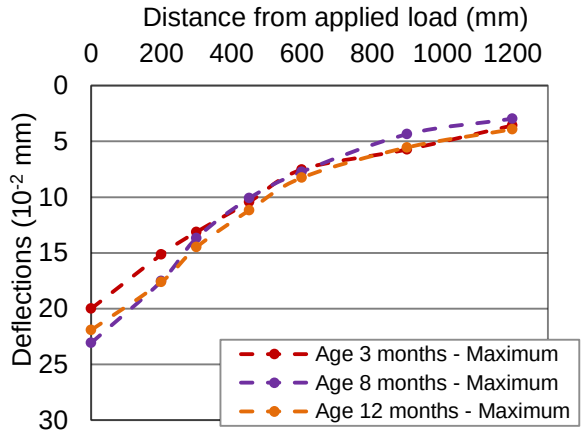
(a) Section I (HMA 9.5) (b) Section II (HMA 19)



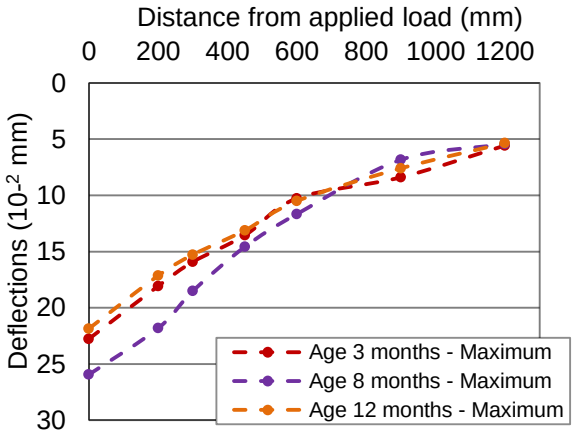
(c) Section I (HMA 9.5)



(d) Section II (HMA 19)



(e) Section I (HMA 9.5)



(f) Section II (HMA 19)

Source: Author

The FWD tests occurred in different climatic conditions, where the 8 months of survey presented the higher deflections, mainly at small distances from the center of load plate, which indicates decrease of asphalt layers' stiffness. As previously remarked, this survey was done during the summer period which implies in higher pavement deformability, as the asphalt concrete is a viscoelastic material and very sensitive to the temperature variation. The deflection blows were not corrected for same temperature analysis, which was only done for the LSAM backcalculated modulus.

4.5.2. Backcalculation

The FWD test has been widely used to quality control during pavement construction, and structural characterization for pavement management. The backcalculation

procedure is used to determine the resilient modulus of structural layers from pavement deflection data.

The Pavement Management Manual of the Brazilian National Department of Transportation ("Departamento Nacional de Infraestrutura de Transportes", DNIT 2011) suggests the use of nondestructive tests to evaluate the structural conditions of pavement and to map the pavement performance over time. During traffic loading, the pavement shows displacement that may be generally characterized in two components, non-recoverable plastic, and recoverable elastic. The first one may result in permanent deformation under the wheel path. On the other hand, the elastic displacement corresponds to the recoverable deflection at the pavement surface, which is used to evaluate the structural conditions of a service road (AASHTO, 2008; DNIT, 2011).

In this study, the FWD tests were performed at different times during the first year after the pavement construction. The software BAKFAA 2.0 was used to backcalculate the field LSAM moduli. Because of the heterogeneity of the remaining infrastructure over the 600 meters of the test section, it was considered a single semi-infinite layer. Both surface courses (HMA 9.5 and HMA 19) were considered structural layers for the backcalculation procedure.

The temperatures of FWD test for pavement surface was between 19ºC to 32ºC, during one year of monitoring, which needs a layer moduli correction after backcalculated. Once the literature about gradient temperature on pavement structures with LSAM is still very limited (FENG, YE and HAO, 2012), the Equation (2) from SHRP (MOHSENI, 1998) was used to calculate the temperature at the middle and at the bottom of LSAM layer. Posteriorly, the temperature correction of the backcalculated moduli was done to 24ºC considering the correction factor calculated from dynamic modulus test (YE, ZHUANG and ZHANG, 2012) of the LSAM applied in field, as explained below.

$$T_d = T_{surf}\,(1 - 0.063\,d + 0.007\,d^2 - 0.0004\,d^3) \qquad (2)$$

Where: T_d is the high AC pavement temperature at a depth, ºF;

T_{surf} is the high AC pavement temperature at the surface, ºF;

d is the pavement depth, in.

According to Loulizi et al. (2002), the depth bellow the pavement surface does not significantly affect the FWD pulse and shape duration which is reasonably closer to a haversine with a duration of 0.03 s. The conversion of time domain (*t*) to frequency domain (f) is considered from different equations by several researchers (DANIEL and KIM, 1998; DONGRE, MYERS and D'ANGELO 2006, AL-QADI et al., 2008). The Equation (3) from Loulizi et al. (2006), also used for FWD analysis, allowed the use of dynamic modulus at loading frequency of 5 Hz as the basis for temperature correction.

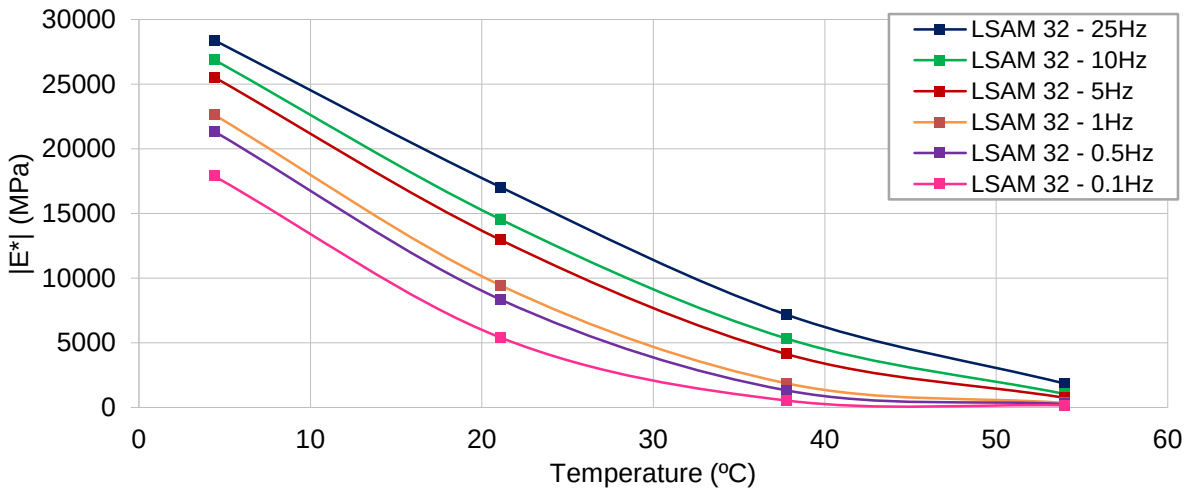
$$f = \frac{1}{2\pi t} \tag{3}$$

Where: f is the frequency, Hz;

t is the time, s.

The isochronal curve at 5Hz was constructed from the LSAM results of dynamic modulus test (Figure 38), performed in accordance with the AASHTO T342-11 (2011) specification. The test considered the temperatures of 4.4, 21.1, 37.8, 54ºC and at loading frequencies 0.1, 0.5, 1.0, 5.0, 10, 25Hz at each temperature.

Figure 38 - Isochronal curves at different frequencies from LSAM dynamic modulus test



Source: Author

Exponential regression was done between dynamic modulus and temperature at loading frequency 5Hz, which allowed the determination of the exponential correction factor (*k*). The Table 19 presented the results of regression analysis.

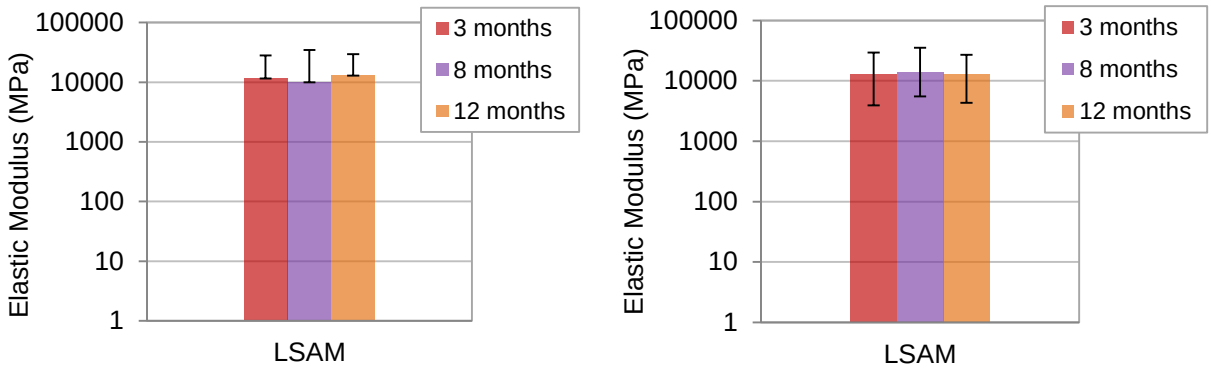
Table 19 - Regression between dynamic modulus and temperature

Mixture	Frequency(Hz)	Regression equation	Correction factor	R^2
LSAM 32	5	$E = 45090\ e^{-0.071\,T}$	$k = e^{-0.071\,(24-T)}$	0.96

Source: Author

Figure 39 presents the adjusted modulus backcalculated from FWD deflection bowls using the correction factor defined as the ratio of the modulus between standard temperature (24ºC) and the average temperature along the LSAM layer. During one year of service, no significant variations were observed in the backcalculated modulus of the LSAM layer.

Figure 39 - Backcalculation LSAM moduli



(a) Section I (HMA 9.5) (b) Section II (HMA 19)

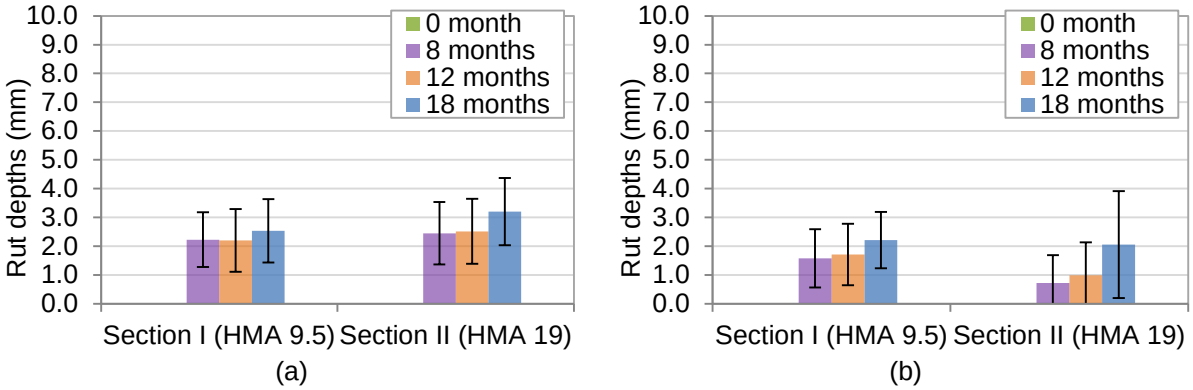
Source: Author

4.5.3. Rutting Analysis

The evaluation of rutting in the test sections was of great importance, as it was the main reason for the rehabilitation in this case. Rut depths were measured each 20 meters in the wheel paths, in four moments after rehabilitation: (i) right after the rehabilitation with no permanent deformation; (ii) 8 months to evaluate possible deformations after the summer and rainy season which could be caused by mixture stability condition, subgrade condition or pavement drainage problems, (iii) 12

months to evaluate the first year of LSAM pavement service life and (iv) 18 months. The results are presented in Figure 40.

Figure 40 - Rut depths measured in the (a) inner wheel path and (b) outer wheel path



Source: Author

The data shows small rut depths (lower than 5 mm) which did not increase between the first three measurements, indicating that this small rutting probably happened due to additional compaction of the pavement layers by traffic. The number of equivalent single axle loads (ESALs) that were applied on the pavement during the first year was calculated based on AASHTO`s equivalent load factors, resulting in 6.85E+06 repetitions. After 18 months, this number increased to 1.05E+07 repetitions.

The measurement at 18 months had an increase when compared to 12 months for the different test sections and wheel path, but it is still lower than 5 mm. There was also no significant difference between the inner and outer wheel paths measurements. Since rutting is a distress that usually happens during the early stages of the pavement service life, especially in heavy traffic situations, one may say that the rehabilitation was successful in fighting permanent deformation in both sections during 18 months when compared to the previous technics used at this site (MOURA, 2010; NEGRÃO, 2012). As showed by the results in the previous chapter, all the asphalt mixtures applied in the test sections (HMA 9.5mm, HMA 19mm, and LSAM 32mm) presented lower permanent deformation results in the laboratory wheel tracking test. However, the use of LSAM along with conventional HMA as wearing course, showed a promising solution, as the HMA alone, even with different aggregate gradations and asphalt binders, presented premature permanent deformation in the first year after rehabilitation at this same location.

5. LSAM PAVEMENT ANALYSIS

5.1. Introduction and Background

The pathologies in asphalt concretes applied in flexible pavements are commonly linked to variables as: design procedure, aggregate type (form, size and density), asphalt binder and additives, as well as the process of production and application of asphalt mixtures in the field. In attempt to combat rutting in asphalt pavements, additives (hydraulic binder, fibers, asphalt modifiers) have been used to improve asphalt mixture stability at high temperatures and subjected to increases in freight transportation.

Characteristics as grading curve of asphalt mixture, aggregate surface texture, aggregate form and sizes are commonly associated with the efficiency to mitigate the permanent deformation. Kandhal (1989) evaluated design procedures for asphalt mixtures with aggregates sizes higher than 19 millimeters in the United States, and reported its application as leveling or base course of flexible pavement structure which can minimize or even eliminate rutting at the pavement surface. However, its grading composition has to ensure that the stresses are transmitted through the interlocked aggregate particles via stone-on-stone contact (NCHRP, 1997).

Although the application of LSAM to prevent rutting is not new, the study of the stresses and strains distribution and the behavior of this material in the field are not yet well-explored. The objective of this chapter is to analyze the LSAM pavement responses of two rehabilitated test sections in Brazil considering the viscoelastic properties of LSAM and HMA. Falling weight deflectometer (FWD) was used to evaluate the structural behavior of both sections, and the structures were simulated by a finite-layer software (3D-Move) using dynamic analysis under different temperatures and traffic speeds. Also, the pavement structure instrumentation was performed to evaluate the temperature gradient, and the real strains in critical points along the depth of asphaltic layers. The results of the computer simulations can be used as a guide for the design and analysis of the pavement structure and pavement performance.

5.2. Pavement Responses: Dynamic Analysis

The use of computational modeling to predict the response of asphalt mixtures have been highlighted when compared to the experimental and empirical models. This mechanistic approach needs the fundamental viscoelastic properties of asphalt concrete to predict the remaining life of pavement. The asphalt mixture has a complex deformation behavior that is dependent on time, temperature, stress state, mode of loading, aging and moisture content. Thus, the stiffness of asphalt concrete is an important material property to predict the performance of flexible pavements by mechanistic analysis (KIM, 2009).

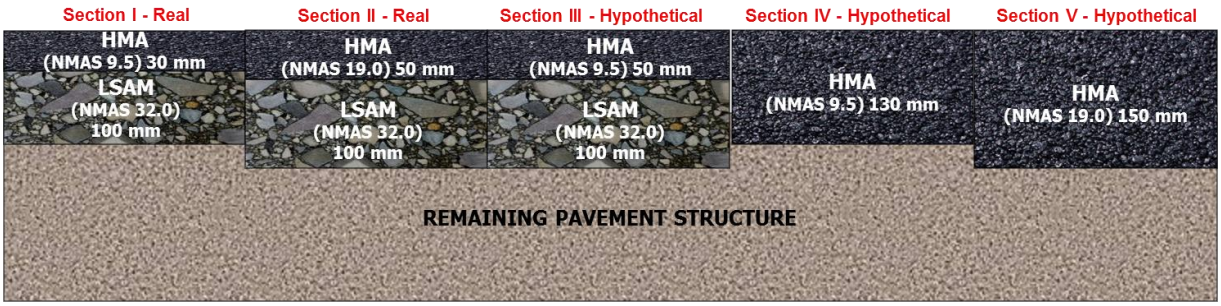
The dynamic modulus is a valuable way to measure the viscoelastic behavior of asphalt concrete which is considered by the application of a sigmoidal axial load at different temperatures and frequencies, which can be considered as an input for dynamic analysis of asphalt pavements. The 3D-Move software uses the finite-layer approach to predict dynamic pavement responses treating the pavement layer as continuum. It has been proposed by Siddharthan, Krishnamenon and Sebaaly (2000) to enable the calculation of stresses distribution and pavement deformations at various extensions and depths when the pavement is subject to a moving or static load with uniform or nonuniform contact stress distribution.

The software also allows the use of linear viscoelasticity characterization of asphalt materials for the analysis of pavement responses. At this scenario, the loading time (frequency) is determined as a function of the loading configuration, the vehicle speed and the pavement structure. Pavement simulation considered the passing of one single axle with dual tire (load of 8 ton) and it was done for the two rehabilitated test sections in Brazil, which construction was described in Chapter 4. The project previously described consisted in milling of previous asphalt layer in two different depths (13 cm and 15 cm), followed by the application of 10 cm of LSAM with NMAS 32 mm as leveling course and different HMAs as surface course, with NMAS of 9.5 and 19 mm, in 3 cm and 5 cm layer thickness, respectively.

The simulation of other three hypothetical pavement structures was considered to enable the evaluation of the different materials in same layer thickness. One used the

same layers' thickness for the two rehabilitated test sections (5 cm of HMA and 10 cm of LSAM 32), the others considered the replacement of LSAM 32 by a continuous layer of conventional HMAs (HMA 9.5 and HMA 19). Figure 41 illustrates the different pavement structures simulated in the 3D-Move.

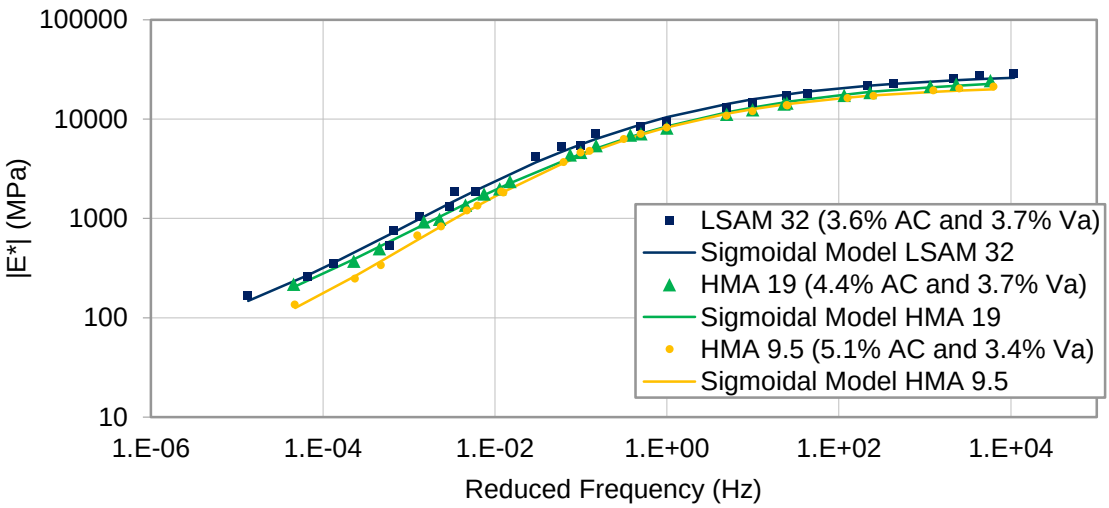
Figure 41 - Pavement structures proposed for simulation



Source: Author

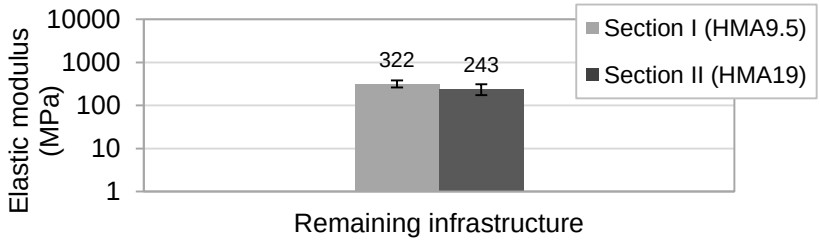
The analysis done for this study characterized the asphalt materials as linear viscoelastic by means of mixtures dynamic modulus (presented again in this chapter, Figure 42 and APPENDIX A). The remaining structure (after pavement milling during rehabilitation) was treated as a linear elastic material behavior using the backcalculated elastic modulus as input (Figure 43). The software BAKFAA 2.0 was used to backcalculate the layer modulus from FWD data. Because of the heterogeneity of the remaining infrastructure over the 600 meters of the test section, it was considered a single semi-infinite layer with the equivalent modulus value.

Figure 42 - Dynamic modulus of asphalt materials



Source: Author

Figure 43 - Backcalculation remaining infrastructure moduli

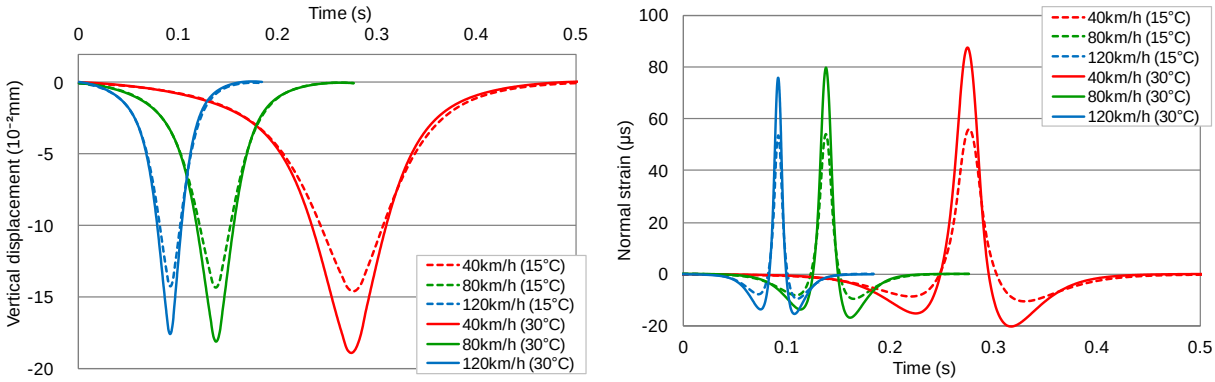


Source: Author

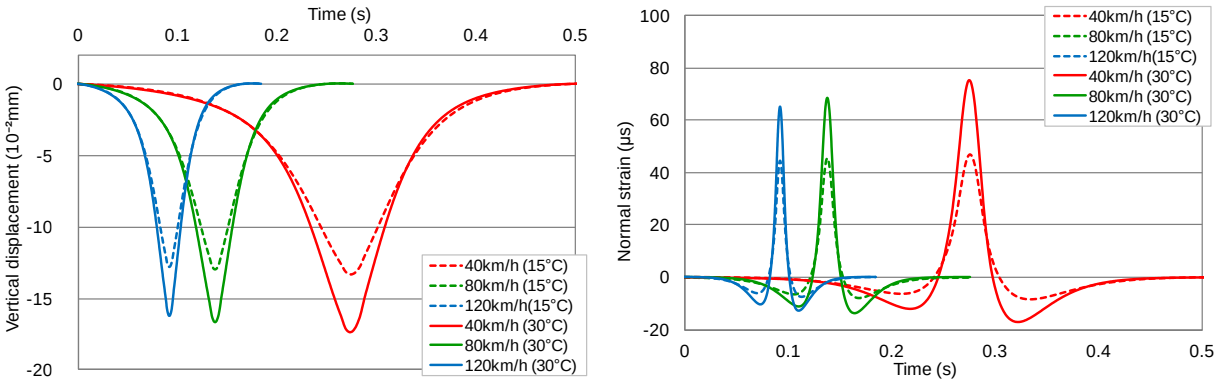
5.2.1. Effect of Traffic Speed and Temperature

The stresses and strains were calculated at critical locations over the pavement structure considering the two rehabilitated test sections with LSAM. There were considered combinations between temperatures (15ºC and 30ºC) and vehicle speeds (40 km/h, 80 km/h and 120 km/h) for the structural analysis. Figure 44 presents the illustration for: (i) the displacement at the top of the surface course (HMA 9.5 and HMA 19), and (ii) the horizontal strain at the bottom of the leveling course (LSAM 32). The maximum values are also reported at Table 20.

Figure 44 - Pavement responses at different vehicle speed



(a) Section I (HMA 9.5 and LSAM 32)



(b) Section II (HMA 19 and LSAM 32)

Source: Author

Table 20 - Maximum vertical displacement and normal strain

Vehicle speed	Vertical displacement (10^{-2} mm)			
	Section I (HMA 9.5 and LSAM 32)		Section II (HMA 19 and LSAM 32)	
	30°C	15°C	30°C	15°C
40 km/h	18.87	14.60	17.35	13.35
80 km/h	18.13	14.31	16.65	12.95
120 km/h	17.62	14.22	16.21	12.84
Vehicle speed	Normal strain (µm/m)			
	Section I (HMA 9.5 and LSAM 32)		Section II (HMA 19 and LSAM 32)	
	30°C	15°C	30°C	15°C
40 km/h	87.51	55.96	75.29	46.83
80 km/h	79.85	54.12	68.59	45.35
120 km/h	76.02	53.37	65.19	44.61

Source: Author

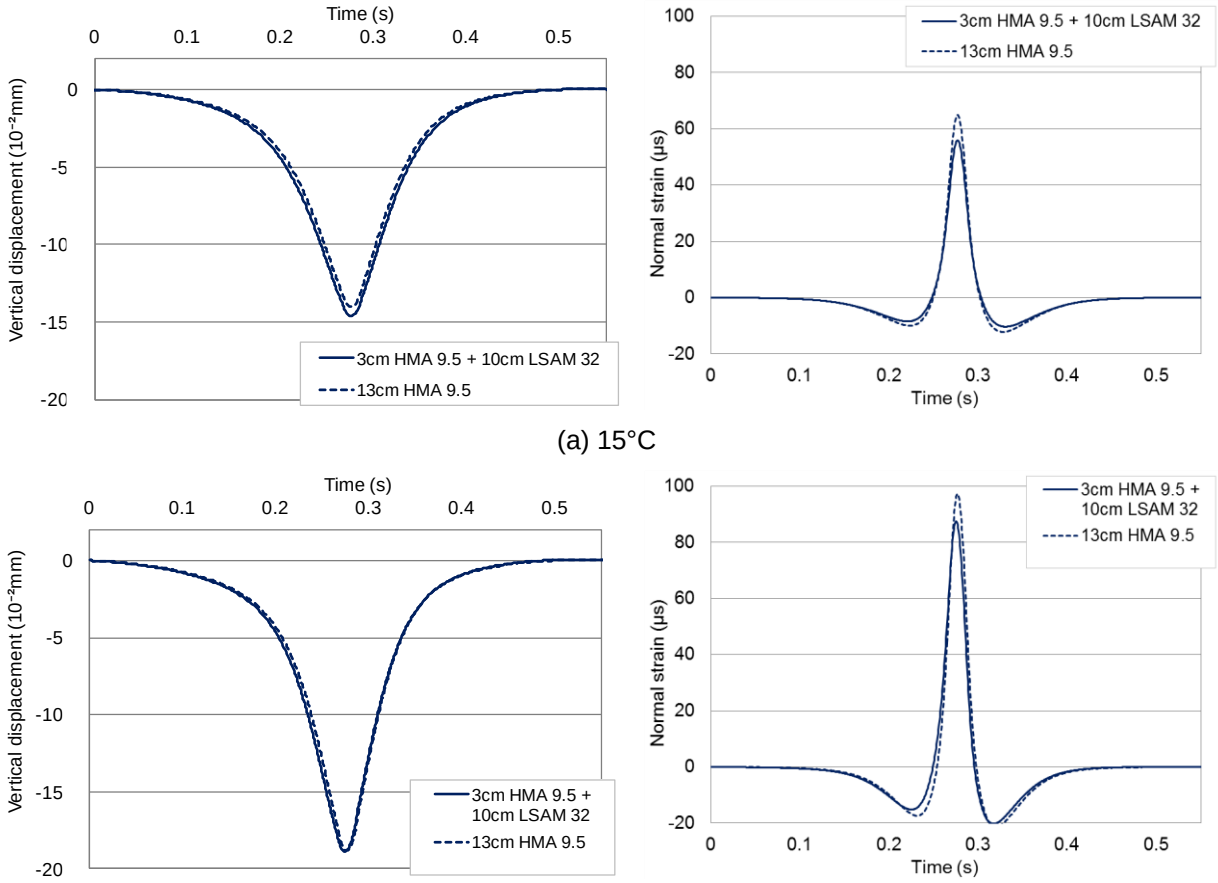
The displacement at the top of the surface course (HMA 9.5 or HMA 19) is very sensitive to the temperature analyses (from 15ºC to 30ºC) which present about 23% of increase with the temperature increase. The change in vehicle speed also presented an influence in pavement responses. The vehicle speed corresponds to the loading rate of the pavement material where the increase of speed causes lower loading time and, consequently, lower deformation compared to the pavement simulation for 40 km/h. The effect of vehicle speed is intensified when the temperature increases, making clearly the difference between displacement values.

For the bottom of the leveling course (LSAM 32), the variation in temperature and vehicle speed of a linear viscoelastic analysis also results in significant change of tensile strain values. For these scenarios, the asphalt material presented higher stiffness at lower temperatures which reduced the tensile stresses and strains.

5.2.2. Effect of Pavement Structure

The first comparison of structures was done between the real section I (3 cm of HMA 9.5 + 10 cm of LSAM 32) and the hypothetical section IV (13 cm of HMA 9.5) which the main objective was to compare the pavement responses for the different scenarios. Figure 45 shows the displacement at the top of surface course and the strain at the bottom of 13 cm depth for vehicle speed of 40 km/h.

Figure 45 - Pavement responses at vehicle speed of 40 km/h - Section I versus Section IV



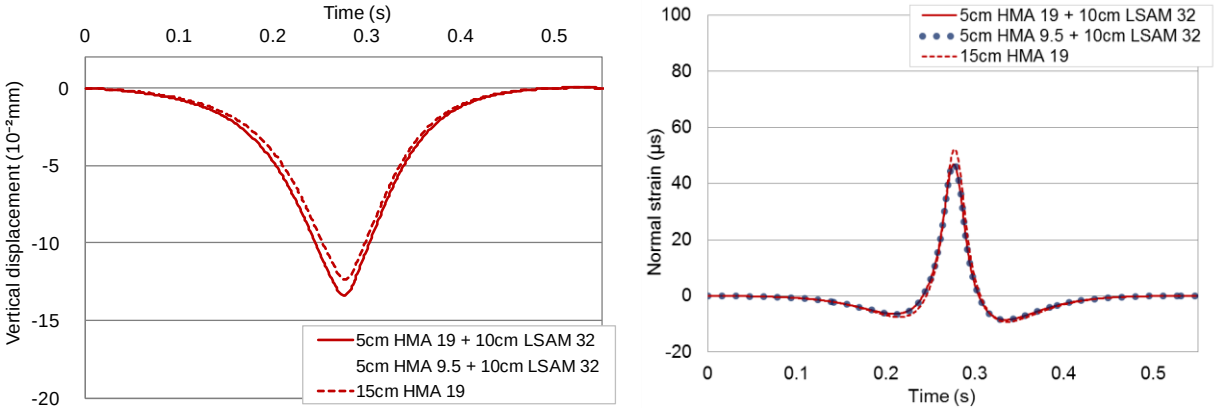
Source: Author

During the sensitivity analysis of the structure effect, the replacement of LSAM 32 from the real section I by a continuous layer of conventional HMA 9.5 resulted in higher tensile strain, about 16% at 15ºC and 11% at 30ºC. However, the displacement at the surface course, with very low variation, decreased about 4% at 15ºC and 1% at 30ºC which was not expected since that pavement with LSAM tends to have higher stiffness. This can be explained by the use of a structure of two asphalt layers with different stiffness (section I and section II) against the equivalent thickness of only one continuous layer (section IV and section V). Also, the temperature increases resulted in pavement responses less sensitivity to the change in the pavement structure, considering the same asphalt layers' depth.

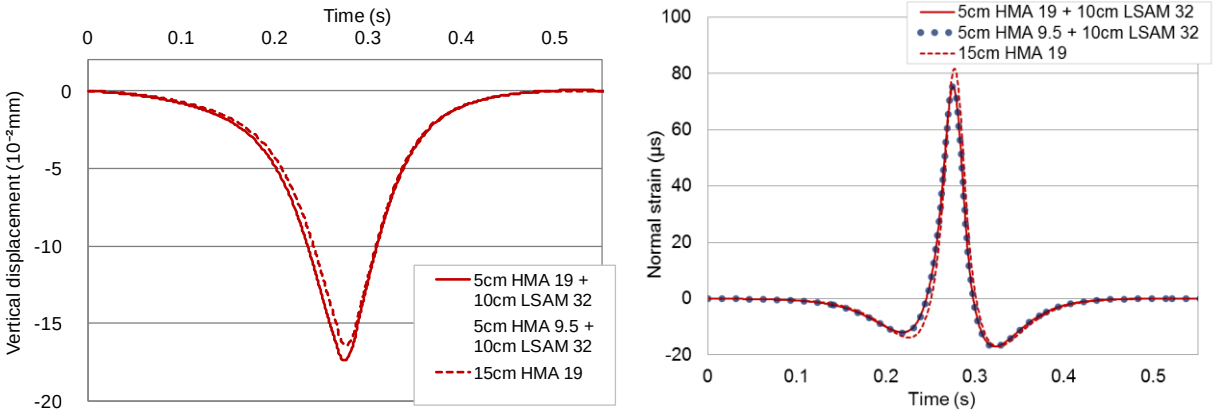
The second comparison considered the real section II (5 cm of HMA 19 + 10 cm of LSAM 32), the hypothetical sections III (5 cm HMA 9.5 + 10 cm of LSAM 32) and V (15 cm of HMA 19). Figure 46 shows the displacement at the top of the surface

course and the normal strain at the bottom of 15 cm depth for vehicle speed of 40 km/h.

Figure 46 - Pavement responses at vehicle speed of 40 km/h - Section II (HMA 19 and LSAM 32)



(a) 15°C



(b) 30°C

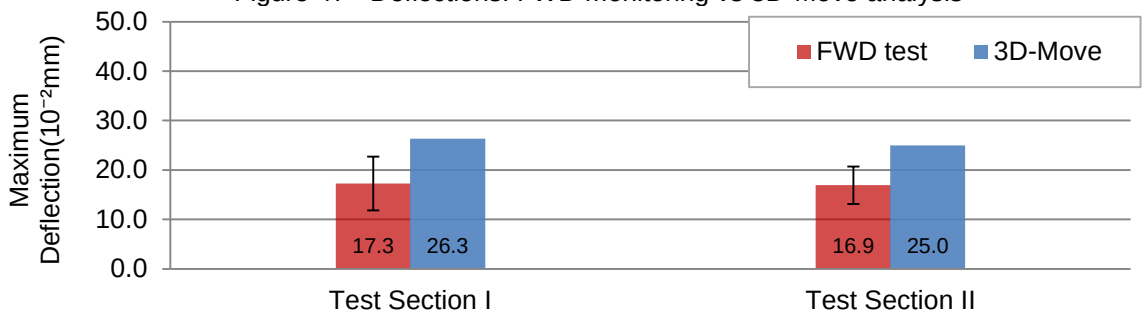
Source: Author

The simulation of equal layer thickness between the rehabilitated test sections presented close pavement responses at each temperature. This occurs due to the similar stiffness of HMA 9.5 and HMA 19, proving that different layer thickness is the main reason for different pavement responses (section I and section II) which will influence in pavement service life. As the first comparison, the replacement of LSAM 32 from the real section II by a continuous layer of conventional HMA 19 also resulted in higher tensile strain (12% at 15ºC and 9% at 30ºC) and lower displacement (7% at 15ºC and 5% at 30ºC).

5.2.3. Pavement Deflection: FWD Monitoring versus 3D-Move Analysis

As described in Chapter 4, FWD monitoring was performed in experimental test site during the first year of service life. These data enable a general and simple comparison between the deflections from FWD tests, and the calculated deflections by means of 3D-Move analysis, considering the same load level and configuration as the FWD equipment. According to Loulizi et al. (2002), the FWD pulse is closer to a haversine with duration of 0.03 s which turns into an estimated speed of 210 km/h (LEIVA-VILLACORTA and TIMM, 2013). Therefore, simulate a moving load with 210 km/h an uniform and circle loaded area with radius of 150 mm was necessary to make it comparable to the FWD impact load, considering the temperature average from the first FWD survey. Figure 47 presents the measured and calculated deflections.

Figure 47 - Deflections: FWD monitoring vs 3D-Move analysis



Source: Author

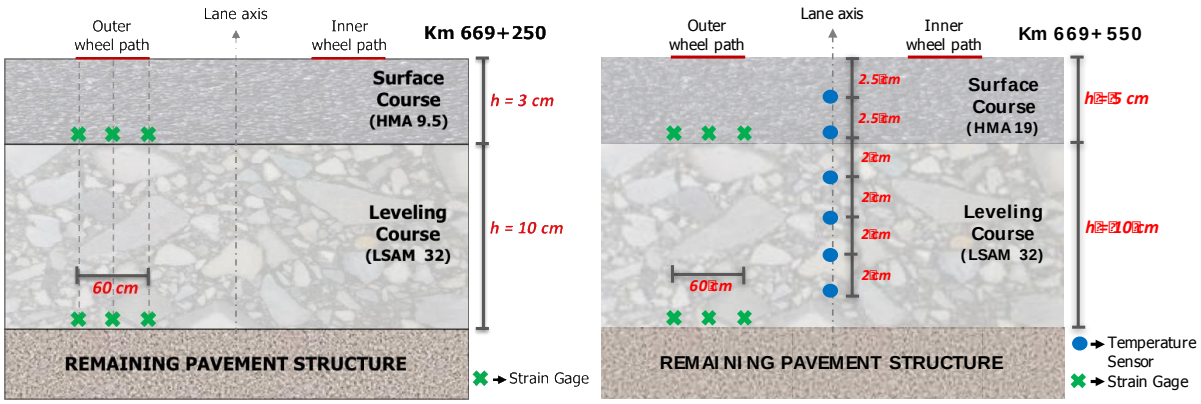
The deflections in field after rehabilitation were lower than the ones obtained by the pavement simulation for the both test sections, which varied almost 50%. Low deflections were observed before the pavement rehabilitation, suggesting that there was no structural problem. Since that the temperature gradient of the LSAM pavement was not obtained on time, it is possible that the remaining pavement structure presents higher stiffness than the average modulus value considered for pavement simulation. Also, this remaining structure is very heterogeneous over the test site presenting layers of totally different materials, which brings low reliability to the backcalculated modulus. However, the maximum deflections from both FWD test and pavement simulation show the same trend when compared the two test sections, where the section I has higher deflections than section II.

5.3. Pavement Responses: Experimental Test Site Instrumentation

The temperature and moisture condition influence the mechanical characteristics of the pavement layers, which can impact the pavement responses to traffic loads and, consequently, the service life of the flexible pavements. Pavement instrumentation is an important tool for the accurate prediction of flexible pavements performance, once it can give measurements in field of pavement responses and temperature at different layer depths. The time and temperature dependency of asphaltic materials brings the need for advanced materials characterization and mechanistic approaches for a realistic performance prediction. The vertical deflections, and the compressive stresses and strains in pavement layer subjected to dynamic loading are key elements to cause accumulated damage inducing surface rutting and fatigue distress. Moreover, the fatigue cracking is consequence of tensile stress and strain under loading repetition.

The two LSAM rehabilitation test sections, described in Chapter 4, were instrumented with sixteen strain gages installed in critical points of the two rehabilitated test sections. The two test sections received, at the same transversal cross section, strain gages at the bottom of surface course and at the bottom of LSAM leveling course. Also, six temperature sensors were placed in the test section II at the same vertical plan in different depths. The Figure 48 shows the location of the strain gages and temperature sensors in both cross sections (installation performed in January of 2018).

Figure 48 - Strain gages and temperature sensors locations



(a) Section I (HMA 9.5) (b) Section II (HMA 19)

Source: Author

Strain gages were installed along the wheel path, one at the center and the other two with 30 centimeters offset to each edge of the wheel path. The main purpose was to increase the probability of vehicle load pass as close as possible to one of the sensors during the data collection. The installation was accomplished by longitudinal and transversal saw cuts as showed in Figure 49.

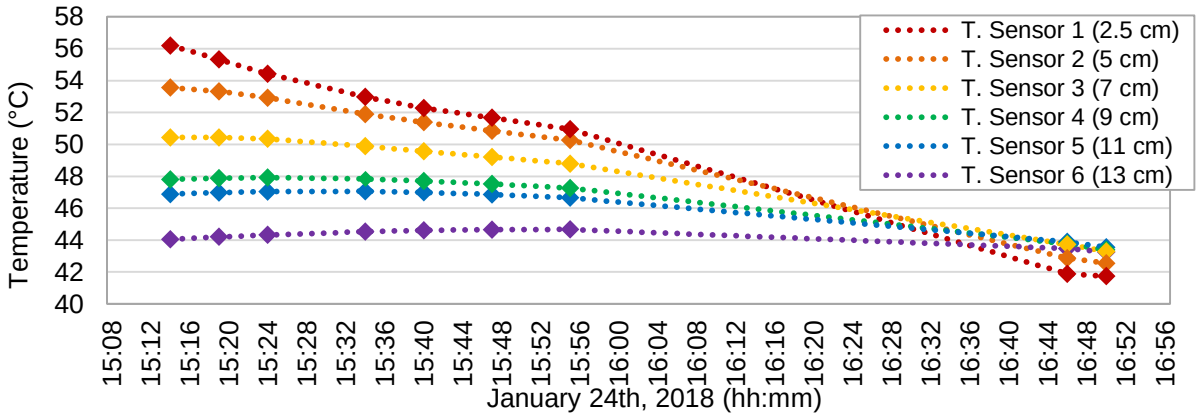
Figure 49 - Pavement saw cuts for instrumentation

Source: Author

5.3.1. Temperature Gradient

The sensors were used to record the temperature at different pavement depths during some surveys along the day, when the air temperature was close to 30°C. Figure 50 illustrates the temperature gradient in one of the test structure used for rehabilitation, the section I with HMA 19 mm and LSAM 32 mm.

Figure 50 - Temperature gradient



Source: Author

Temperature decreases as the sensor depth increases from the pavement surface, which is expected once the first surveys were done during sunny hours of the day. However, the two last surveys had an inverse trend for the two sensors closer to the pavement surface, located at HMA 19 mm surface course, that might be explained by a rainfall that may have cooled the surface with influenced in the other pavement depths (leveling course of LSAM 32).

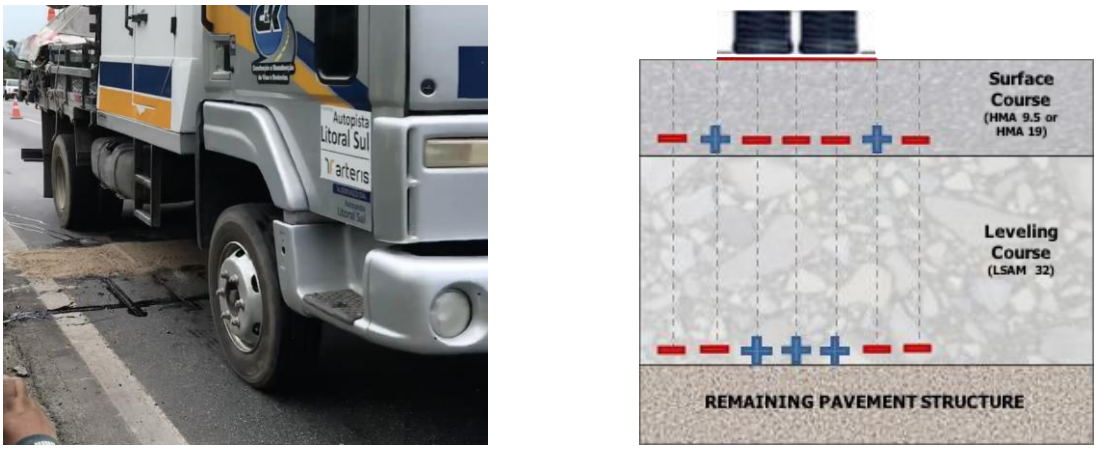
The temperature along the middle of surface course and the bottom of LSAM maintained between 44°C to 56°C, which means a pavement with temperature gradient of 12°C during the short interval recorded in the summer season.

5.3.2. Strain at the Asphalt Layers

The strain measurements were done for a truck with one single axle with single tire (load of 6.6 ton) and one single axle with dual tire (load of 11 ton) and also for free traffic operation to analyze the spectrum of strains over the pavement structure. One concern is to know the location of the strain gages compared to the loading position. Successive load passing at different distances from the sensors, with traffic video recording, and the use of sand to print the contact area of truck tire (Figure 51(a)) were monitored.

In order to previously evaluate how the asphalt layers will perform with load application, pavement simulations using the 3D-Move software were considered to evaluate the stress state at critical locations (where the strain gages were installed). Figure 51(b) shows the expected pavement responses regarding the tension (+) compression (-) strains.

Figure 51 - Instrumentated pavement: (a) traffic load and (b) simulated stress state

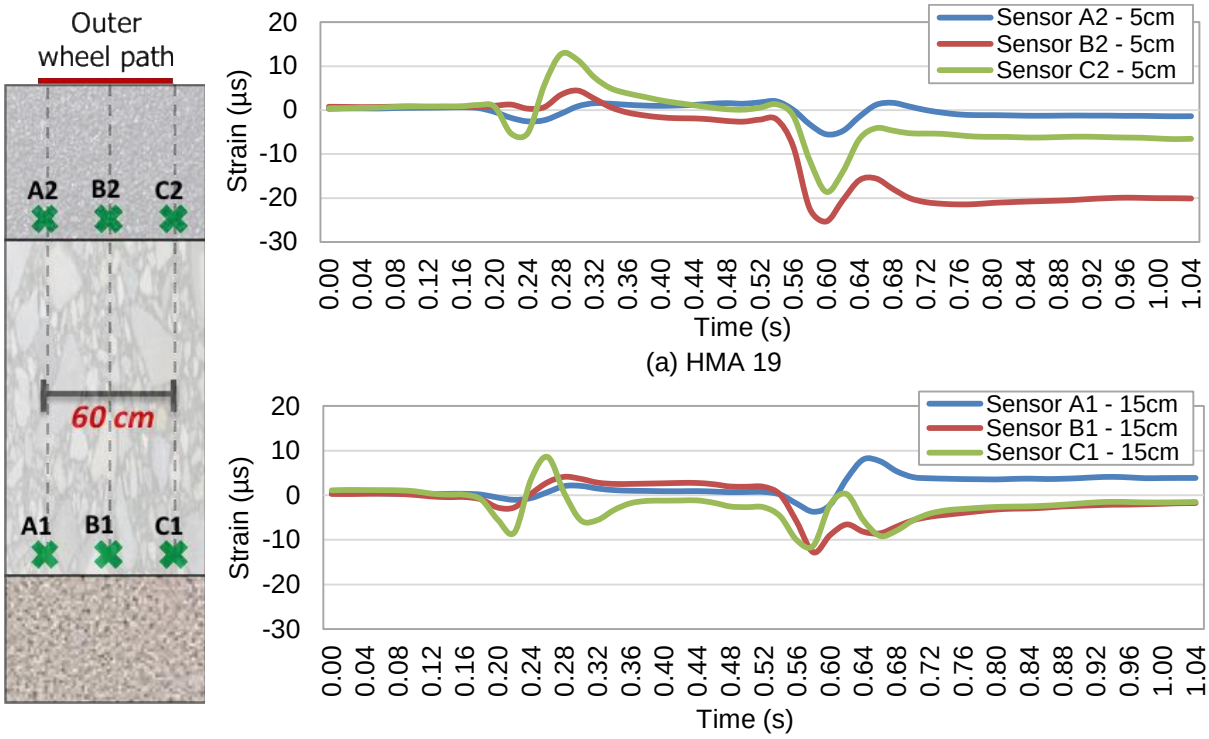


(a) (b)

Source: Author

For the known axle load, low strains were recorded considering the vehicle speed of 40 km/h. The maximum strain amplitudes correspond to compression at the bottom of HMA (5 cm) and to tension at the bottom of LSAM (15 cm) as show in Figure 52.

Figure 52 - Measured strains at (a) 5 cm depth and (b) 15 cm depth



(a) HMA 19

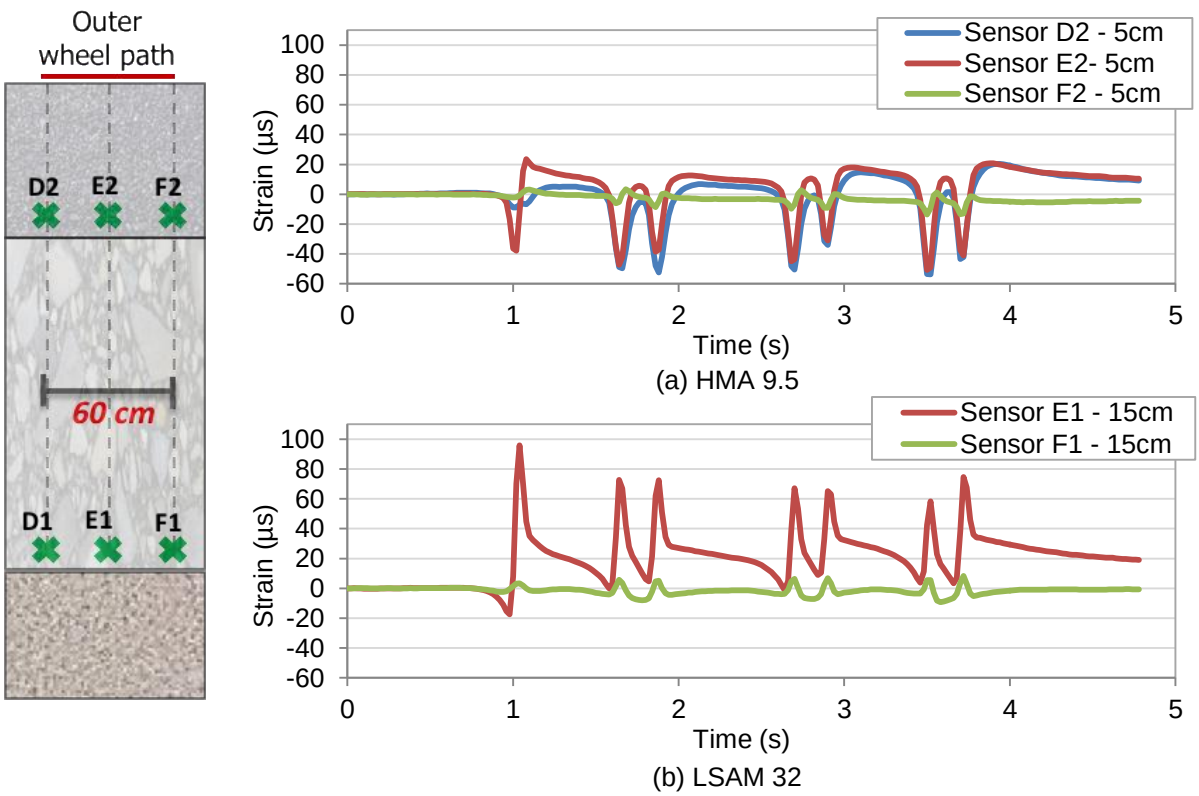
(b) LSAM 32

Source: Author

The magnitude of strain in different depths varied from 10 µs (tension strain at the bottom of LSAM 32 layer) to 25 µs (compressive strain at the bottom of s HMA 19 surface course) when an axle load of 11 ton passed with vehicle speed of 40 km/h at the middle of outer wheel path. The surface course is working mainly in compression. The three strain gages captured this behavior for the single axle with double tire.

The strains at the bottom of LSAM 32 (15 cm depth) presented strain amplitudes varying from tension to compression, probably due to the distance from the load, and also to its proximity to neutral line. The existing of 7 to 9 cm of asphalt layers at the remaining infrastructure can influence the neutral line location which can be closer to the bottom of LSAM 32. Other strain pulses were collected during surveys, presenting values of tension strain varying from 50 to 400 µs. Figure 53 and Figure 54 show an example of strains recorded by the stain gauges located at the bottom of the surface course (HMA 9.5), and at the bottom of leveling course (LSAM 32) of the rehabilitated test section I. Table 21 and Table 22 show the maximum values for tension and compression strain at these different depths.

Figure 53 - Strain pulses for a passing vehicle I



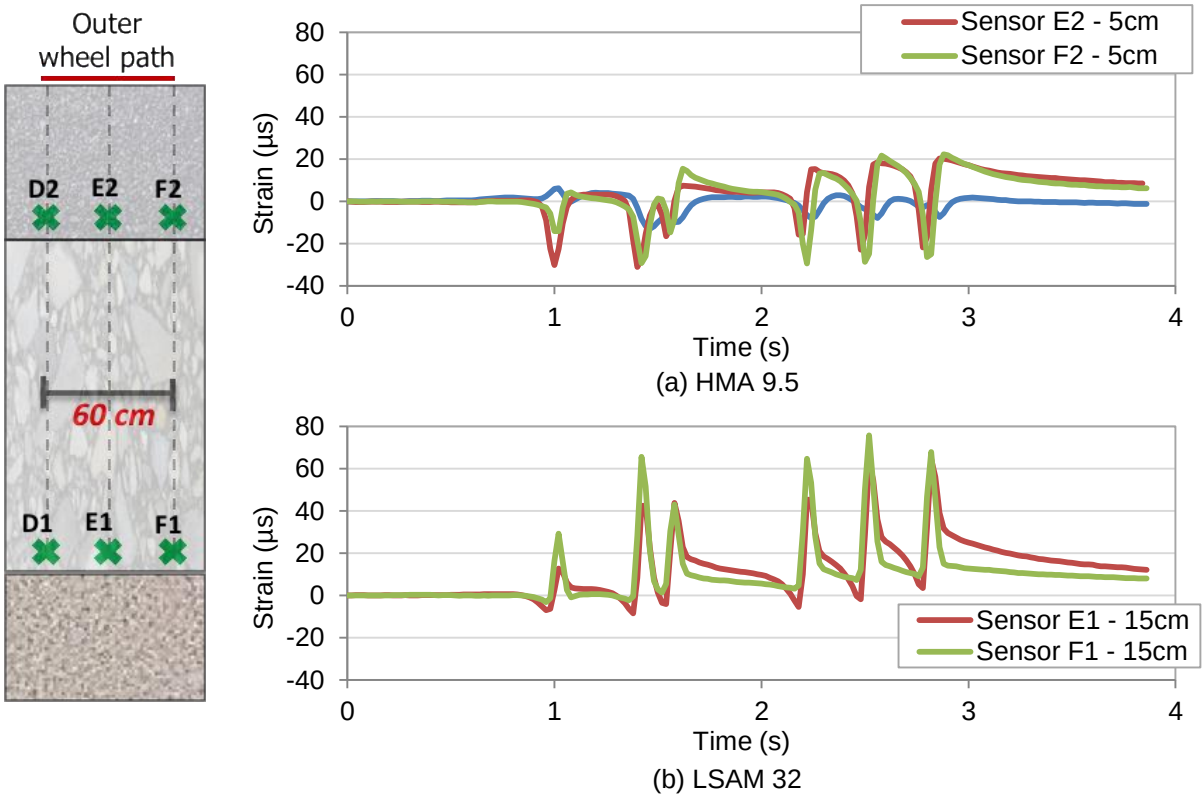
Source: Author

Table 21 - Maximum strains from vehicle I

Maximum strains	Tension (µs)	Compression (µs)
Depth 3 cm (HMA 9.5)	23.6	-53.6
Depth 13 cm (LSAM 32)	95.9	-17.4

Source: Author

Figure 54 - Strain pulses for a passing vehicle II



Source: Author

Table 22 - Maximum strains from vehicle II

Maximum strains	Tension (µs)	Compression (µs)
Depth 3 cm (HMA 9.5)	22.4	-30.9
Depth 13 cm (LSAM 32)	75.8	-8.4

Source: Author

The configuration of truck axles is presented at the continuous measurement of strains, where each tire provides different strain values depending of its load. The data confirm the tension-compression distribution over the pavement, where the surface layer is mainly working in compression, even with some small tensile strain

during the load approximation. The LSAM leveling course is mainly working in tension, as expected, which confirms that the neutral line is at the LSAM layer.

6. SUMMARY AND CONCLUSIONS

The LSAM (nominal maximum aggregate size above 25 mm) is a solution to resist the permanent deformation, proposed for the rehabilitation of pavements subjected to slow and heavy traffic. The experimental test site described at this study was the first experience with LSAM with Brazilian materials, which needed an intensive laboratory evaluation. Different mix design techniques for LSAM were considered at different compaction energies. The design evaluations showed:

- The Marshall compaction does not allow the correct accommodation of coarse aggregates into the specimen mold (limited to 100 mm of diameter), with insufficient compaction for the LSAM design;
- The rolling compaction was considered unappropriated for LSAM design due to the large amount of material necessary to prepare the slabs for design purpose, but should be considered for mechanical behavior analysis;
- For the Superpave gyratory compaction, the locking point seems to be an additional tool for LSAM design analysis, with the compaction at 100 gyrations as the suggested one, assuring enough densification and less propensity do aggregate breakage during compaction.

The mechanical analysis of designed asphalt mixtures with different NMAS was considered in this study to evaluate the LSAM behavior. The laboratory tests showed that:

- The mixtures with different nominal maximum aggregate sizes (9.5 mm, 19 mm, and 32mm) resulted in crescent dynamic modulus results according to NMAS increasing, what was initially expected due to the lower binder content of LSAM when compared to the conventional HMAs;
- The possible presence of air voids of larger dimensions in LSAM may harm its fatigue life due to the high stress concentration at these points. Also, the specimen dimensions at fatigue tests is contested for LSAM with NMAS of 32 mm, once it could not be representative;

- An increase in mixture NMAS also influences the permanent deformation, increasing the rutting resistance probable due to the interlock of the aggregate skeleton with direct stone-on-stone contact (to be confirmed with the 3D image analysis).

The research also allowed the evaluation of large stone asphalt leveling course performance using two rehabilitation test sections constructed in Brazil. Thus far, the use of LSAM in the rehabilitated test sections was successful when compared to other techniques previously used at the same experimental test site (MOURA, 2010; NEGRÃO, 2012), showing almost no permanent deformation after almost two years of service life. The rutting measurements at 18 months were lower than 5 mm and is in accordance to the maximum limit of 7 mm from technical guidelines for Brazilian highway concessions, specified by the National Land Transportation Agency (ANTT).

The dynamic simulation of rehabilitated pavements showed the asphalt layer responses considering its linear viscoelasticity behavior. The reduction in vehicle speed significantly changed the strain values which were intensified by the increase in the pavement temperature. This more severe scenario should be considered for the test site where the LSAM mixture was applied (heavy and slow traffic on the third lane). Also, stress state in dynamic simulations got worst with the replacement of the LSAM layer by a continuous layer of conventional HMA with the same thickness. The results obtained from pavement simulation showed the benefits of the rehabilitation strategy using LSAM. This study can be used as a reference for the design and performance analysis of large stone pavements.

6.1. Suggestions for Future Studies

This study presented the evaluation of LSAM, starting from the design and mechanical behavior to the test sites construction and field monitoring. Even though the results show a successful pavement performance during 18 months of surveys when compared to the previous rehabilitations, more tests and analysis are still need for mixtures with large aggregates. The following suggestions for further studies are made according to Brazilian experience implemented in the state of Paraná:

- Investigate the influence of aggregate form and mineralogy on LSAM properties;
- Evaluate a different laboratory fatigue test which can use representative LSAM specimen dimensions;
- Evaluate the fatigue characterization of asphalt mixtures with different NMAS by Viscoelastic Continuum Damage (VECD) theory;
- Analyze the air voids (size and distribution) in laboratory compacted LSAM samples, comparing with conventional asphalt mixtures;
- Field monitoring of FWD deflections with different load applications to evaluate the linear or non-linearity behavior using the backcalculation of LSAM layer modulus;
- Continuous rutting monitoring of LSAM in the field to evaluate its life cycle compared to laboratory tests;
- Calculate the LSAM pavement responses to vehicle dynamic load considering the viscoelasticity of asphalt materials by means of finite-element analysis.

REFERENCES

AASHTO - AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS. AASHTO M 323/04: Standard Specification for Superpave Volumetric Mix Design. USA, 2004.

AASHTO - AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS. AASHTO T. 342-11: Standard Method of Test for Determining Dynamic Modulus of Hot-Mix Asphalt Concrete Mixtures. Washington, D.C., 2011.

AASHTO - AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS. Mechanistic-Empirical Pavement Design Guide - A Manual of Practice. AAoSHaT Officials, Editor, 2008.

AASHTO - AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS. AASHTO M. 323-13. Standard specification for Superpave volumetric mix design, 2013.

AHLRICH, R. C. Influence of Aggregate Gradation and Particle Shape/Texture on Permanent Deformation of Hot Mix Asphalt Pavements. U.S. Army Corps of Engineers, Waterways Experiment Station, 1996.

AIREY,G. D.; COLLOP, A. C. Mechanical and Structural Assessment of Laboratory- and field-Compacted Asphalt Mixtures. Nottingham Transportation Engineering Centre, University of Nottingham, Nottingham, UK, 2014.

AL-QADI, Imad L.; XIE, Wei; ELSEIFI, Mostafa A. Frequency determination from vehicular loading time pulse to predict appropriate complex modulus in MEPDG. Asphalt Paving Technology-Proceedings, v. 77, p. 739, 2008.

AL-QADI, I. L.; ABAUWAD, I. M.; DHASMANA, H.; COENEN, A. R. Effects of various asphalt binder additives/modifiers on moisture-susceptible asphaltic mixtures. Illinois Center for Transportation, 2014.

ALVAREZ-LUGO, A. E.; CARVAJAL-MUÑOZ, J. S.; WALUBITA, L. F. Comparison of the air voids characteristics of different hot mix asphalt (HMA) mixture types. Ingeniare. Revista chilena de ingeniería, v. 22, n. 1, 2014.

ANWAR, S. Studies on Marshall and Modified Marshall Specimens by Using Crmb. 2014.

ASPHALT INSTITUTE. Superpave Mix Design: Superpave series SP-2. 3. Ed. 2001.

ASTM D2872-12. Standard test method for effect of heat and air on a moving film of asphalt (rolling thin-film Oven test). West Conshohocken, PA: American Society for Testing and Materials, 2012.

ASTM D7405-15. Standard Test Method for Multiple Stress Creep and Recovery (MSCR) of Asphalt Binder Using a Dynamic Shear Rheometer. American Society for Testing and Materials International. Vol. 04.03. West Conshohocken, PA, 2015.

ASTM D7460. Standard Test Method for Determining Fatigue Failure of Compacted Asphalt Concrete Subjected to Repeated Flexural Bending, p. 14, 2010.

BALBO, J. T. Pavimentação Asfáltica: Projetos e Restauração. Oficina de Textos, São Paulo, SP, 2007.

BEJA, I. A.; FLORENCIO, H. A.; CHAVES, J. M.; VASCONCELOS, K. L.; BERNUCCI, L. L. B.; MASCARENHAS, Z. M. G. Large Stone Asphalt Mixture (25mm): Avaliação Do Procedimento De Dosagem. In: XVIII Congresso Ibero Latinoamericano del Asfalto 2015, Bariloche. 2015.

BERNUCCI, L. B.; MOTTA, L.; CERATTI, J.; SOARES, J. Pavimentação Asfáltica: Formação Básica para Engenheiros. 2010.

BROWN, E. R.; BASSETT, C. E. Effects of maximum aggregate size on rutting potential and other properties of asphalt-aggregate mixtures. Transportation Research Record, v. 1259, p. 107-119, 1990.

BUCHANAN, M.; BROWN, E. Effect of superpave gyratory compactor type on compacted hot-mix asphalt density. Transportation Research Record: Journal of the Transportation Research Board, n. 1761, p. 50-60, 2001.

CAO, W. D.; YAO, Z. Y.; SHANG, Q. S.; LI, Y. Y.; YANG, Y. S.. Performance Evaluation of Large Stone Porous Asphalt-Rubber Mixture. School of Civil Engineering, Shandong University, China, 2011.

CNT. Anuário CNT do Transporte - Estatísticas Consolidadas 2016. p. 12, 2017.

COLPO, G. B. Análise de fadiga de misturas asfálticas através do ensaio de deflexão em viga de quatro pontos. Dissertação de Mestrado. Universidade Federaldo Rio Grande do Sul, Porto Alegre, RS, Brasil, 2014.

COMINSKY, R.; LEAHY, R. B.; HARRIGAN, E. T. Level one mix design: Materials selection, compaction, and conditioning. No. SHRP-A-408. 1994.

CONSUEGRA, A.; LITTLE, D. N.; QUINTAS, H. V.; and BURATI, J. Comparative evaluation of laboratory compaction devices based on their ability to produce mixtures with engineering properties similar to those produced in the field. Transportation Research Record, 1228, 80 – 87. 1989.

DANIEL, J. S and KIM, Y. R.. Relatioships among Rate-Dependent Stiffness of Asphalt Concrete Using Laboratory and Field Test Methods. Transportation Research Record 1630, 3-9. TRB, National Research Coucil, Washington, D.C, 1998.

DER/SP - DEPARTAMENTO DE ESTRADAS E ROLAGEM DO ESTADO DE SÃO PAULO. Especificação Técnica ET – DE– POO/26. Pré-Misturado a Quente, São Paulo, SP, 2006.

DNER - DEPARTAMENTO NACIONAL DE ESTRADAS DE RODAGEM. DNER-ME 043/95. Norma técnica. Misturas betuminosas à quente – Ensaio Marshall, Rio de Janeiro, RJ, 1995.

DNIT - DEPARTAMENTO NACIONAL DE INFRAESTRUTURA DE TRANSPORTES. Diretoria Executiva. Instituto de Pesquisas Rodoviárias. Manual de gerência de pavimentos. - Rio de Janeiro, Brasil, 2011.

DNIT - DEPARTAMENTO NACIONAL DE INFRAESTRUTURA DE TRANSPORTES. ES-031/2004. Pavimentos flexíveis – Concreto asfáltico – Especificação de serviço. Rio de Janeiro, 2004.

DONGRE, R., MYERS, L., and D'ANGELO, J. Conversion of Testing Frequency to Loading Time: Impact on Performance Predictions Obtained from the M-E Pavement Design Guide. Paper No. 06-2394 Presented at the 85th Transportation Research Board Annual Meeting, 2006.

DOUGAN, C. E.; STEPHENS, J. E.; MAHONEY, J.; & HANSEN, G. E*-dynamic modulus: test protocol-problems and solutions (No. CT-SPR-0003084-F-03-3). 2003.

EMERY, S. J. Large aggregate mixes in bases. Group Technical Director. Colas Southern Africa Pty Ltd, Isando, South Africa, 1996.

EUROPEAN COMMITTEE FOR STANDARDIZATION. EN, N. F. 12697-22: Mélanges bitumineux: Méthodesd'essai pour mélange hydrocarboné à chaud: Partie 22: Essaid'orniérage. Version française. Bruxelles, 2004.

EUROPEAN COMMITTEE FOR STANDARDIZATION. EN 12697-33: Mélange bitumineux: Méthodes d'essai pour mélange hydrocarboné à chaud: Partie 33: Confection d'éprouvettes au compacteur de plaque. Version française. Bruxelles, 2003.

FENG, X. J.; YE, S.; HAO, P. W. Study on the Rule of Temperature Descent for Large Stone Asphalt Mixtures. In: Advanced Materials Research. Trans Tech Publications, p.1829-1832. 2012.

FERNANDO, E. G.; BUTTON, J. W.; CROCKFORD, W. W. Rut susceptibility of large stone mixtures. Journal of Transportation Engineering, v. 123, n. 1, p. 51-59, 1997.

GATCHALIAN, Dennis. Characterization of aggregate resistance to degradation in stone matrix asphalt mixtures. Dissertation. Texas A&M University. 2006.

GELDENHUYS, J. R. Developing a large stone asphalt to improve the performance of asphalt surfacing. Master dissertation. Pretoria, Republic of South Africa, 2011.

GEORGIOU, P.; LOIZOS, A.; LEVENTIS, A. Influence of compaction method on stiffness performance of asphalt specimens. Bituminous Mixtures and Pavements VI, p. 401, 2015.

GUDIMETTLA, J. M.; COOLEY, L. A.; BROWN, E. R.. Workability of hot mix asphalt. NCAT, 2003.

HARVEY, J.; TSAI, B. Effects of asphalt content and air void content on mix fatigue and stiffness. Transportation Research Record: Journal of the Transportation Research Board, n. 1543, p. 38-45, 1996.

HINGLEY, C. E.; PEATTIE, K. R.; POWELL, W. D. French experience with Grave-bitume, a dense bituminous road base. TRRL Report SR 242 Monograph, p. 16, 1976.

HUANG, B. Fundamental Characterization and Numerical Simulation of Large Stone Asphalt Mixtures. 2000.

HUGO, F.; RUST, F.C.; MYBURGH, P.A.; GROBLER, J. Large stone asphalt mix design for heavy duty asphalt pavements, 1990.

KANDHAL, P. S. Large Stone Asphalt Mixes: Design and Construction. National Center for Asphalt Technology. 1990.

KANDHAL, P. S. Testing and evaluation of large stone mixes using Marshall mix design procedures. NCAT Report 89-04. 45p. National Center for Asphalt Technology, Auburn University, Alabama, 1989.

KIM, Y. Richard. Modeling of asphalt concrete. 2009.

LEE, J.; NOROUZI, A.; KIM, Y. R.. Determining specimen geometry of cylindrical specimens for direct tension fatigue testing of asphalt concrete. Journal of Testing and Evaluation, v. 45, n. 2, p. 613-623, 2016.

LEIVA-VILLACORTA, F.; TIMM, D. Falling weight deflectometer loading pulse duration and its effect on predicted pavement responses. In: Transportation Research Board 92nd Annual Meeting. 2013.

LITTLE, D. N.; EPPS, J. A. The benefits of Hydrated Lime in Hot Mix Asphalt. National Lime Association, 2001.

LOULIZI, A.; AL-QADI, I.; LAHOUAR, S.; FREEMAN, T. Measurement of vertical compressive stress pulse in flexible pavements: representation for dynamic loading tests. Transportation Research Record: Journal of the Transportation Research Board, (1816), 125-136. 2002.

LOULIZI, A.; FLINTSCH, G.; AL-QADI, I.; MOKAREM, D. Comparing resilient modulus and dynamic modulus of hot-mix asphalt as material properties for flexible pavement design. Transportation Research Record: Journal of the Transportation Research Board, (1970), 161-170. 2006.

LPC. Manuel LPC d'aide à la formulation desenrobés. Groupe de Travail RST ''Formulation des enrobés''. Laboratoire Central des Ponts et Chaussées. Paris, France, 2007.

MAHBOUB, K. Large-stone Mixes for Reducing Rutting. Southeastern Consortium of University Transportation Centers (SECUTC), Kentucky Transportation Center, University of Kentucky, 1990.

MAHBOUB, K.; WILLIAMS, E. G. Construction of Large-Stone Asphalt Mixes (LSAMs) in Kentucky. Transportation Research Record, v. 1259, p. 41, 1990.

MASAD, E. et al. Characterization of air void distribution in asphalt mixes using X-ray computed tomography. Journal of materials in civil engineering, v. 14, n. 2, p. 122-129, 2002.

MOHAMMAD, L.; KIM, M.; RAGHAVENDRA, A.; OBULAREDDY, S. Characterization of Louisiana Asphalt Mixtures Using Simple Performance Tests and MEPDG (No. FHWA/LA. 11/499). Louisiana Transportation Research Center.2014.

MOHSENI, A. LTPP seasonal asphalt concrete (AC) pavement temperature models. (No. FHWA-RD-97-103). Federal Highway Administration. 1998.

MOURA, E. Estudo de deformação permanente em trilha de roda de misturas asfálticas em pista e em laboratório. Tese de Doutorado. Universidade de São Paulo, 2010.

NAPA - National Asphalt Pavement Association. Design, Construction, and Performance of Heavy Duty Mixes, Quality Improvement Series 123. National Asphalt Pavement Association. Landham, MD. 2002.

NASCIMENTO, L. A. H. "Nova abordagem da dosagem de misturas asfálticas densas com uso do compactador giratório e foco na deformação permanente". Dissertação de Mestrado. Instituto Alberto Luiz Coimbra de Pós-Graduaçao e Pesquisa de Engenharia, Universidade Federal do Rio de Janeiro, Rio de Janeiro, 2008.

NASCIMENTO, L. A. H. Implementation and validation of the viscoelastic continuum damage theory for asphalt mixture and pavement analysis in Brazil. Dissertation. North Carolina State University, 2015.

NCHRP - National Cooperative Highway Research Program, Report 386: Design and Evaluation of Large-Stone Asphalt Mixes. Transportation Research Board, National Research Council, Washington, 1997.

NCHRP - National Cooperative Highway Research Program. NCHRP Report 573: Superpave mix design: verifying gyrations levels in the Ndesign table. Transportation Research Board, National Research Council, Washington, 2007.

NEGRÃO, D. P. Contribuição para calibração de curva de evolução de afundamentos em trilha de roda de revestimentos asfálticos com utilização de

resultados obtidos de simulador de tráfego em escala real. Tese de Doutorado. Universidade de São Paulo, 2012.

NEWCOMB, D. E.; WEI, Z.; STROUP-GARDINER, M. Investigation of Large-Stone Mixtures.Civil& Mineral Engineering Department, University of Minnesota, 1993.

OLIVEIRA FILHO, C. M. S. Estudo do Efeito de Diferentes Granulometrias no Comportamento Mecânico de Misturas Asfálticas Densas Descontínuas Tipo Stone Matrix Asphalt (SMA). Dissertação de Mestrado, Programa de Mestrado em Engenharia de Transportes, Universidade Federal do Ceará, Fortaleza, CE, 122 fl, 2007.

PELLINEN, T. K.; WITCZAK, M. W.; BONAQUIST, R. F. Asphalt mix master curve construction using sigmoidal fitting function with non-linear least squares optimization. In: Recent advances in materials characterization and modeling of pavement systems. 2004.

PRADHAN, M. M. Permanent deformation characteristics of asphalt-aggregate mixtures using varied materials and molding procedures with Marshall method. Tese de Doutorado. Montana State University-Bozeman, College of Engineering. 1995.

PRICE, D. A.; ASCHENBRENER, T. Large-Stone Hot Mix Asphalt Pavements. Research Report 1994-5, Colorado Department of Transportation, Denver, Colorado, 1994.

ROBERTS, F. L.; KANDHAL, P. S.; BROWN, E. R.; LEE, D.; KENNEDY, T. W.; Hot mix asphalt materials, mixture design, and construction. National Asphalt Pavement Association Education Foundation 2 ED Maryland, 1996.

SIDDHARTHAN, R.V.; KRISHNAMENON, N.; SEBAALY, P.E. Pavement Response Evaluation using Finite-Layer Approach. Transportation Research Record No. 1709, TRB, 2000.

SWIERTZ, D.; MAHMOUD, E.; BAHIA, H. Asphalt mixture compaction and aggregate structure analysis techniques: state of the art report, in: RILEM Technical Committee 206-ATB Meeting, TRB, Washington, D.C., 2010.

TANGELLA, S. R.; CRAUS, J.; DEACON, J. A.; & MONISMITH, C. L. Summary report on fatigue response of asphalt mixtures (No. SHRP-A-312). 1990.

THE CAMBRIDGE TRIBUNE.A Saturday Morning Record of Cambridge and Harvard Affairs, Volume XXIX, nº 29.Cambridge, Massachusetts, 16th September of 1916.

USACE. Hot-Mix Asphalt Paving Handbook. AC 150/5370-14A. US Army Corps of Engineers, Washington, DC, 2000.

VASCONCELOS, K. L. Comportamento Mecânico de Misturas Asfálticas a Quente Dosadas pelas Metodologias Marshall e Superpave com Diferentes Granulometrias. Dissertação de Mestrado em Engenharia de Transportes, Centro de Tecnologia, Universidade Federal do Ceará, Fortaleza, 2004.

VAVRIK, W. R. “Asphalt mixture design concepts to develop aggregate interlock”. Thesis (Doctor of Philosophy) Civil Engineering in The Graduate College of University of Illinois at Urbana-Champaign, 2000.

VAVRIK, W.R.; CARPENTER, S.H. Calculating air voids at specified number of gyrations in Superpave Gyratory Compactor. Transportation Research Record: Journal of the Transportation Research Board, No. 1630, 1998.

VERHAEGHE, B. M.; RUST, F. C.; AIREY, G. D.; VOS, R. M. Fatigue and Deformation Characteristics of Large-Aggregate Mixes for Bases. In: Proceedings of the 6th Conference on Asphalt Pavements for Southern Africa, Capsa'94, Held Cape Town October 1994. Vol 1. 1994.

VON QUINTUS, H. L. Asphalt-aggregate mixture analysis system, AAMAS. Transportation Research Board, 1991.

WARREN BROTHERS COMPANY. The Bitulithic Pavement. The Best by Every Test Built from Experience. 59 Temple Place, Boston, Massachusetts. 1912.

WATSON, D.; MOORE, J.; HEARTSILL, J.; JARED, D.; WU, P. Verification of Superpave number of design gyration compaction levels for Georgia. Transportation Research Record: Journal of the Transportation Research Board, n. 2057, p. 75-82, 2008.

WEI, J. C.; LI, J. Dynamic Modulus Characteristic of Large Stone Permeable Asphalt Mixtures. In: Applied Mechanics and Materials. Trans Tech Publications, 2013.

YE, Y. L.; ZHUANG, C. Y.; ZHANG, R. F. A Method for Temperature Correction of HMA Dynamic Modulus. In: Applied Mechanics and Materials. Trans Tech Publications, p. 1615-1618. 2012.

YUFENG, B.; SONNEN, W.; HUBER, G. Research and applications of new pavement structure based on large stone porous asphalt mixture. In: Bearing Capacity of Roads, Railways and Airfields. 8th International Conference (BCR2A'09), 2009.

ZANIEWSKI, J.P.; NALLAMOTHU, S.H. Evaluation of Binder Grades on Rutting Performance. Asphalt Technology Program, Department of Civil and Environmental Engineering, West Virginia University, 2003.

ZHANG, Z.; BAI, Q. Laboratory Study of Fatigue Limit for LSM Base Mixture. In: Plan, Build, and Manage Transportation Infrastructure in China, 2008.

ZHAO, Y.; HUANG, X. Design method and performance for large stone porous asphalt mixtures. Journal of Wuhan University of Technology--Materials Science Edition, v. 25, n. 5, p. 871-876, 2010.

APPENDIX A - Dynamic modulus test

Table 23 - Dynamic modulus and phase angle for analyzed asphalt mixtures

Temperature (°C)	Frequency (Hz)	LSAM 32 mm		HMA 19 mm		HMA 9.5 mm	
		Phase angle (°)	|E*| (MPa)	Phase angle (°)	|E*| (MPa)	Phase angle (°)	|E*| (MPa)
4.4	25	5.60	28387.20	6.40	24250.79	5.88	21102.95
	10	6.10	26869.84	7.10	22295.19	6.72	20131.90
	5	6.56	25523.08	7.57	21169.87	6.99	19217.51
	1	8.08	22616.43	9.01	18441.35	8.15	17005.01
	0.5	9.87	21334.61	11.14	17244.90	9.74	16118.28
	0.1	12.87	17875.35	14.68	14096.09	12.42	13639.74
21.1	25	12.52	17026.18	12.90	14384.36	13.06	13712.72
	10	14.56	14530.68	14.67	12386.02	14.62	11934.14
	5	15.24	12944.24	15.44	11096.53	15.52	10759.76
	1	19.00	9430.95	19.08	8069.20	18.61	8137.11
	0.5	25.02	8333.16	24.95	7096.90	23.27	7018.25
	0.1	30.13	5394.63	30.47	4595.94	27.97	4558.24
37.8	25	25.84	7156.97	23.20	6869.96	24.95	6250.91
	10	28.09	5313.11	25.31	5377.20	27.18	4745.95
	5	30.44	4112.60	27.08	4352.61	28.86	3661.76
	1	36.97	1854.69	32.39	2345.45	32.97	1806.20
	0.5	38.38	1310.92	34.96	1753.89	36.06	1328.70
	0.1	37.90	528.39	33.32	919.01	34.93	666.81
54.0	25	37.72	1844.82	36.50	1982.34	36.55	1845.19
	10	40.24	1041.15	35.02	1357.40	36.96	1188.81
	5	37.99	745.30	35.06	975.33	37.26	823.83
	1	34.36	345.10	32.12	494.67	34.12	335.17
	0.5	31.61	257.98	31.27	368.27	32.56	245.61
	0.1	24.45	164.54	25.29	217.23	27.79	134.47

Table 24 - Mastercurves and shift factor function fitting coefficients

Asphalt mixture	Sigmoidal function coefficients				Shift factor polynomial function coefficients		
	a	b	d	g	α_1	α_2	α_3
LSAM 32	1.3956	3.0661	1.7751	0.5871	0.0008	0.1811	3.4260
HMA 19	1.2894	3.1371	1.6595	0.5505	0.0007	0.1568	3.0275
HMA 9.5	1.1162	3.2262	1.8851	0.6270	0.0008	0.1647	3.1094