Laboratory characterization of bitumen treated full depth reclamation materials

by

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LABORATORY CHARACTERIZATION OF BITUMEN TREATED FULL DEPTH RECLAMATION MATERIALS

Apparao GANDI

RÉSUMÉ

À la suite du vieillissement du réseau d'infrastructures routières, les interventions visant à rétablir l'état de la chaussée, à accroître le confort de conduite de l'utilisateur et / ou à améliorer la sécurité routière sont très courantes au Canada. Depuis les années 1970, les traitements de réadaptation, par opposition aux nouvelles constructions, gagnent du terrain. En raison de la crise économique, l'augmentation du coût des matériaux comme les granulats vierges, les liants, etc., et le désir de préserver un réseau routier efficace et durable ont ravivé le recyclage du revêtement existant comme une des options principales. En particulier, le recyclage à froid en place de l'enrobé seulement (CIR) et de l'enrobé et d'une partie de la couche granulaire (FDR) des chaussées sont des alternatives prometteuses pour la réhabilitation des routes.

Bien que des matériaux d'enrobés recyclés à froid avec des matériaux stabilisés au bitume aient été appliqués avec succès, il subsiste certains problèmes critiques dans cette technologie. L'un des domaines où un travail important est nécessaire est la compréhension et la modélisation du comportement de ces matériaux à jeune âge. Ces deux matériaux, la plupart du temps lorsqu'ils sont traités avec une émulsion de bitume, ont des caractéristiques évolutives dans le temps, qui sont principalement liées à leur teneur en eau. La conception du mélange pour les matériaux CIR et FDR a été principalement basée sur la procédure de conception de mélange Marshall qui est empirique et qui rend difficile son utilisation dans d'autres applications.

L'objectif principal de ce projet de recherche est d'améliorer les performances à court terme et à long terme des matériaux d'enrobés recyclés à froid traités avec une émulsion de bitume ou avec de la mousse de bitume. Pour ce faire, la température de cure a été variée, et il a été démontré que les basses températures ont un impact négatif sur le comportement mécanique des enrobés recyclés à froid (CRM). Les performances ont été évaluées de différentes manières, dont le module complexe qui a permis l'évaluation du comportement viscoélastique de ces matériaux. Les essais de module complexe ont entre autre été effectué avec une pression de confinement ce qui a permis de voir que le comportement des CRM se situe entre un granulat et un matériau bitumineux à jeune âge. Finalement, il a été possible de définir la combinaison optimale d'émulsion et de mousse de bitume en les faisant varier afin d'améliorer la performance mécanique des CRM.

Les résultats de la recherche contribueront à une meilleure compréhension de la méthode de conception du mélange basée sur la presse à cisaillement giratoire et des propriétés

rhéologiques des matériaux d'asphalte recyclés à froid avec de l'émulsion de bitume ou de la mousse de bitume.

Keywords: Matériaux bitumineux recyclés à froid, conception du mélange, cure, émulsion et mousse de bitume, et module dynamique.

LABORATORY CHARACTERIZATION OF BITUMEN TREATED FULL DEPTH RECLAMATION MATERIALS

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ABSTRACT

Because of an aging road infrastructure network, interventions to restore the pavement condition, increase the user's riding comfort, and/or improve the road safety are very common in Canada. Since the 1970s, rehabilitation treatments as opposed to new constructions have been gaining momentum. Because of the economic crisis, increased cost of materials like virgin aggregate, binder, etc., and a strong desire to preserve effective and sustainable roadway system have fueled the popularity of recycling existing pavement as a primary option. Particularly, Cold In-place Recycling (CIR) and Full-Depth Reclamation (FDR) of asphalt pavements are promising alternatives for road rehabilitation.

Although Cold Recycled bituminous Materials with bitumen-stabilized materials have been successfully applied, there are still some critical problems existing in this technology. One of the areas where much work is needed is in the understanding and the modeling of the behavior of those materials at younger age. Those two materials, mostly when treated with asphalt emulsion, have evolutive characteristics with time, which is mostly linked to their water content. The mix design for CIR and FDR materials has been mostly based on Marshall mix design procedure which is empirical that makes it difficult to use in further application.

The main objective of this research project is to enhance the short term and long-term performance of Cold Recycled bituminous Materials (CRM) treated with asphalt emulsion and/or with foamed asphalt. To that end, the curing temperature of CRM was varied, and it showed that cold temperature do have a negative impact on their mechanical performances. Those performances were evaluated with different tests, including complex modulus measurement that enabled us to describe their viscoelastic behavior. Complex modulus tests were also performed at young age with different confining pressure, which showed that that the behavior is between bituminous and granular material. Finally, it was possible to define the optimum binder combination by testing different double coating combinations of emulsion and/or foam in order to increase the CRM's performances.

The results from the research will contribute to a better understanding of the mix design method based on the Superpave gyratory compactor, and rheological properties of the Cold Recycled bituminous Materials with foamed asphalt and emulsified asphalt.

Keywords: Cold Recycled bituminous Materials, Mix design, Curing, foamed asphalt, emulsified asphalt and Complex modulus.

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LIST OF ABREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
AC	Asphalt Content
ASTM	American Society for Testing and Materials
BSM	Bitumen stabilized materials
CSS1P	Cationic Slow Setting 1 with Polymer
CSS1S	Cationic Slow-Setting with soft bitumen emulsion
CIR	Cold In-Place Recycling
CRM	Cold Recycled asphalt/bituminous Materials
EA	Emulsified asphalt
ESAL	Equivalent Single Axle Load
FA	Foamed asphalt
FCM	Federation of Canadian Municipalities
FHWA	Federal Highway Administration
FDR	Full depth reclamation
HMA	Hot mix asphalt
ITS	Indirect tensile strength
ITSM	Indirect Tensile Stiffness Modulus
MTQ	Ministry of transportation Quebec
OPSS	Ontario Provincial Standard Specifications
PCC	Portland cement Content
PG	Penetration grade
RAP	Recycled asphalt pavement
SGC	Superpave gyratory compactor
TTSP	Time Temperature Superposition Principle
TSR	Tensile Strength Ratio
VA	Virgin aggregate

INTRODUCTION

Because of an aging road infrastructure network, interventions to restore the pavement condition, enhance the user's riding comfort, and/or improve the road safety are very common in Canada. Since the 1970s, rehabilitation treatments as opposed to new constructions have been gaining momentum. Because of the economic crisis, increased cost of materials like virgin aggregate, binder, etc., and a strong desire to preserve effective and sustainable roadway system have fueled a reviving of recycling existing pavement as a primary option.

In other words, when the roadway network was rapidly expanding, the initial construction cost was the most important issue, with little or no attention being paid to the ongoing maintenance costs. Since funding for preventive maintenance, preservation, rehabilitation, and reconstruction of roadways will have to compete with other demands on the public purse, innovation is required in order to do more with less. Asphalt recycling is one way of increasing the effectiveness of existing budgets in order to maintain, preserve, rehabilitate and reconstruct more miles (kilometers) of roadway for each dollar spent (ARRA, 2001).

There are several methods to recycle asphalt pavements. All over the world, the experience and the choice of technology for In-place recycling varies broadly. Particularly, different methods of recycling and reclamations are applicable to different types, levels, and severity of distresses, and hence different periods in the pavement life as shown in Figure 0.1. Typically, Hot In-place Recycling (HIR) is used when the majority of the pavement distresses are minimal and limited to the upper few inches of the surface of the roadway with no evidence of structural problems (i.e., longitudinal cracking in wheel path, alligator cracking, and edge cracking). Cold In-place Recycling (CIR) is used when there is a higher number, type, and severity of non-load-related distresses that may extend farther down from the surface. CIR with an overlay can be used to address some load-related distresses. Full Depth Reclamation (FDR) is an in-place rehabilitation process that can be used for reconstruction, lane widening, minor profile improvements, and increased structural capacity by addressing the full range of pavement distresses. The anticipated depths of the distresses, combined with the overall existing asphalt pavement thickness, are used to identify the type of in-place recycling process (es); that can be expected to extend the life of the pavement most economically. Mainly CIR and FDR with the addition of Bitumen Stabilized Materials (BSM) like foamed or emulsified asphalt.

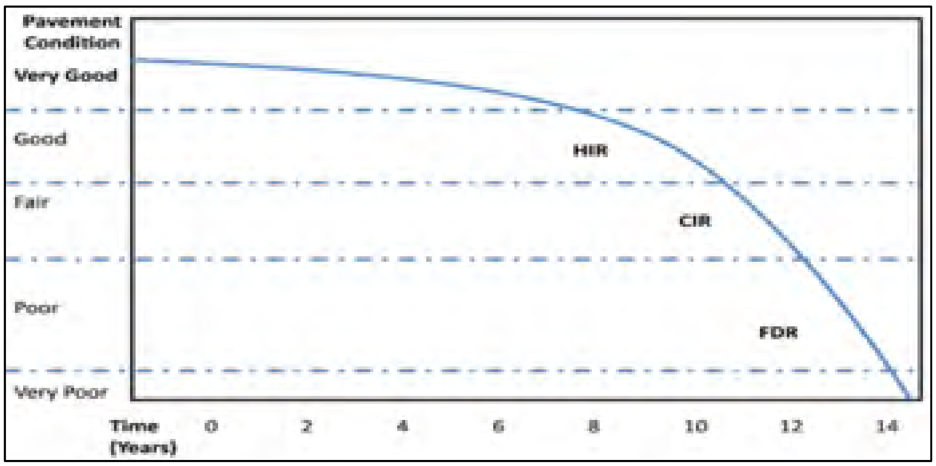

Figure 0.1-1 Pavement condition and type of In-place recycling
Taken from the Stroup-Gardiner (2011)

CIR is a Cold In-Place Recycling method, in which only the existing bituminous materials are recycled which is considered as 100% RAP. In this method bitumen is added, as an emulsion or foam, and makes a good base material that needs to be covered with a layer of Hot Mix Asphalt (HMA) or a surface treatment. CIR can be accomplished as full depth reclamation or partial depth recycling. In partial depth recycling a portion of asphalt (Base) layer in the pavement, normally it is performed at the depth of 50mm to 100mm (2 to 4in), and it is more frequently used to create a base course, in most cases low-to-medium traffic volume highways (ARRA, 2001; Kandhal & Mallick, 1998). On the other hand, in FDR, both asphalt layer and part of the granular base are recycled (50% RAP and 50% virgin aggregate) at the same time and reconstructed with or without the addition of bitumen. FDR materials also need to be covered with a layer of HMA or surface treatment. It is usually done for depth between 100 mm to 300 mm (4 to 12 in.) (ARRA, 2001; Epps & Allen, 1990).

A typical view of CIR and FDR construction process as illustrated in Figure 0.2. CIR and FDR together hereafter referred to as Cold Recycled bituminous Materials (CRM) in this thesis.

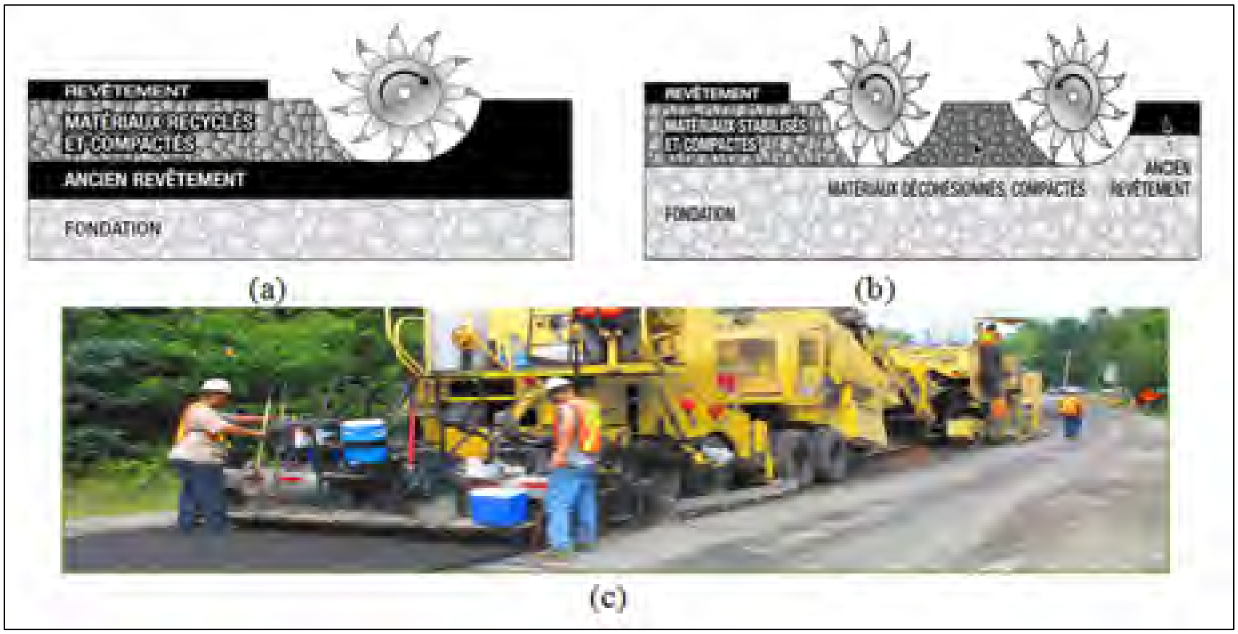

Figure 0.1-2 Cold recycled bituminous materials process
(a) Cold in-place recycling process (CIR), (b) Full-depth reclamation process (FDR)
(c) Surface treatment with hot mix asphalt for CIR and FDR process
Taken from Bitume Quebec (2018)

The behavior of Bitumen-Stabilized Materials (BSMs) is uniquely different from all other materials used to construct road pavements. Unlike asphalt, where the bitumen as a continuum binds all the aggregate particles together, the bitumen in a BSM is dispersed selectively amongst only the finer particles, regardless of whether bitumen emulsion or foamed bitumen is used as the stabilizing agent (Jenkins, 2012). Firstly, Bitumen Emulsion can be defined as, it is comprises bitumen emulsified in water. The bitumen is dispersed in the water in the form of an oil-in-water type bitumen emulsion. An emulsifying agent holds the bitumen in suspension. The emulsifying agent determines the charge of the bitumen emulsion. Cationic bitumen emulsions have a positive charge and anionic bitumen emulsion have a negative charge. Secondly, foamed bitumen is produced by injecting water into hot bitumen, resulting in spontaneous foaming. The physical properties of the bitumen are temporarily altered when the injected water, on contact with the hot bitumen, is turned into vapor, which is trapped in thousands of tiny bitumen bubbles. Particularly, CIR and FDR of asphalt pavements are famous design alternatives for road rehabilitation. These techniques has shown huge potential saving money and material, and allowing a convenient way of fixing proper grade and cross slope to highway. However, even if many successful projects were completed with these techniques, there are still some problems are not addressed by the researchers on the subject.

The mix designs are performed to determine the desirable amount of the different materials used in the composition of the Cold Recycled bituminous Materials (CRM) and to ensure that these materials meet the desired quality requirements. As far as, the method of Cold recycled asphalt mixture design based on the Hveem stabilometer, Marshall stability test results, air voids, resilient modulus test, most of the times based on experiences, sometimes field trials, and visual condition of samples to establish the optimum binder content. A standard national method is not available. However, A review of practice in the North America, Epps et al. (1990) showed that the Marshall mix procedure was used in over 60% of the agencies that used mix design procedures. Other agencies used the Hveem resilient modulus and indirect tensile test. However, 25% of the agencies did not use a formal design procedure at all, rather relying on field workability and experience for determining the binder content. There is also research underway to adopt Superpave technology to CIR mixtures (Emery, 1993; Tia, Castedo, Wood, & Iida, 1983). Furthermore, in 2014, as reported by task group (TG6) of RILEM TC-237 SIB (Tebaldi et al., 2014) ″Currently a universally accepted specimen preparation procedure for cold-recycled mixtures is not available, therefore universities, research centres and road administration agencies are developing specific methods based on their laboratory tests and field experiences″. The CIR pavement design process involves testing of representative specimens of foamed and emulsified treated materials as means to evaluate pavement performance over time. To adequately acquire representative specimens, it is necessary to condition the materials in a process called Curing (K. J. Jenkins & Moloto, 2008). Although curing procedures have been adopted in many countries, the curing protocols are varied and an accepted procedure is currently not available. The lack of representation is due to complex process of curing simulation, like type and content for composition materials, climatic conditions, mechanical properties of the materials, etc. (K. J. Jenkins & Moloto, 2008). Finally, all the various mix design methods provide an approach for selecting the type and the percentage of asphalt binder; however, the methods of compaction, curing and testing differ.

At early stages, the behavior of FDR materials is similar to a granular material, but after the curing phase ends, the behavior is close to a HMA. Therefore, it has been suggested that the FDR materials treated with asphalt binder like emulsion or foam, have a time dependent behavior (Pérez, Medina, & del Val, 2013). Hence, they can be considered, at some point, to be in between a purely granular material and a HMA. Due to time dependent behavior inherent to CRM's, Carter et al. (2013) acknowledged the challenge related to measuring the stiffness of CRM, given the variation that occurs depending on the considered curing protocol. Stiffness values of FDR mixes can be determined through a resilient (M_R) or a dynamic modulus test. Santagata et al. (Santagata, Chiappinelli, Riviera, & Baglieri, 2010b) investigated the short term stiffness of FDR mixes by using a triaxle cell. A very limited amount of research has been done on the influence of confining pressure on CRM's with dynamic modulus. Due to a lack of research results in CRM's, the current mechanistic-empirical pavement design guide (M-E PDG) and design method treat CIR mixtures the same as HMA base course. However, they significantly vary in terms of stress-strain behavior where that of CIR materials is influenced by stabilizing agents and the quality of RAP materials. Todd and Richard (Thomas & May, 2007) reported that the dynamic modulus of Full-Depth Reclamation with emulsified asphalt (FDR-emulsion) was influenced by the mix composition, quantity of RAP, and binder type.

However, a review of research studies on cold recycled bituminous materials reveals that, this may be attributed the fact that the mix design procedure for CIR and FDR materials has been mostly based on Marshall mix design procedure which is empirical, which makes it difficult to use in further application. It should be noted that, much research has been undertaken to better understanding the emulsion and foaming technologies on cold recycled bituminous materials separately. Furthermore, based on previous research on cold recycled materials, the early stage strength development, the impact of the mixing, compaction and curing temperature on the mechanical properties, double coating of cold recycled bituminous materials and modeling of these materials has not been completely studied. This manuscript based PhD thesis aims to address those problems.

CHAPTER 1

RESEARCH PROBLEM AND OBJECTIVES

1.1 Research Problem

In Québec, over the past twenty years, Cold In-place Recycling (CIR) and Full-Depth Reclamation (FDR) have been reliable rehabilitation techniques; restoring pavement condition at affordable cost with a lower footprint on the environment. Lot of research is being undertaken on many aspects of pavement maintenance and rehabilitation in order to increase their lifespan. However, limited amount of research exists in the literature even though many successful projects were completed with CIR and FDR. This may be attributed to the fact that the mix design procedure for CIR and FDR has been mostly experience based which makes it difficult to use these materials. In addition, to decrease the environmental impact, a major advantage of Cold Recycled bituminous Materials (CRM) over hot recycling asphalt techniques is the possibility to reuse higher percentages of Recycled Asphalt Pavement (RAP). In hot in-plant recycling of asphalt mixtures, a maximum of 40% RAP is generally accepted in the base layers, and this amount is reduced to 15% or even prohibited in the surface layers. In CRM, the usage of RAP can be as high as 100%, but this generally results in a loss of mechanical properties and durability (Stimilli, Ferrotti, Graziani, & Canestrari, 2013). Therefore, it is necessary to study and determine the correct mixture proportion for CRM through a mix design procedure, which consider RAP percentage is the most important parameter.

CIR is a cold recycling method in which only the existing bituminous materials are recycled. In this method, bitumen is added in the form of an emulsified asphalt or foamed asphalt to serve as a good base material that is covered with a layer of Hot Mix Asphalt (HMA) or a surface treatment. On the other hand, in FDR, both asphalt layer and part of the granular base are recycled at the same time and reconstructed with or without the addition of bitumen. As with CIR, FDR materials need to be covered. It should be noted that CIR and FDR characteristics change over time. For instance, the stability of these materials is low at younger age and it increases with time. Different knowledge gaps exist, and one of the areas where much work is needed is in understanding and modeling the behavior of these materials at young age. These materials, when treated with asphalt emulsion, have evolving characteristics with time, which is linked, partially, to their water content. Considering the time dependency of the behavior of CRM, it is important to study the impact of curing conditions on CRM.

In addition to time, in most countries, there is a low-temperature limit to lay down HMA because if it is too cold, it becomes impossible to get proper compaction. All over the world, various studies have been done to understand the compaction behavior of hot mix asphalt materials. However, limited research is available on low-temperature compaction of CRM. The compaction of CRM with emulsified asphalt or foamed asphalt is a very important factor to get good mechanical characteristics. It helps to position the particles of the material and redistribute the binder from separate globules to continuous films (Needham, 1996). The compaction quality has an impact on air voids of the CRM (Kassem, 2008; Lauter, 1998; Pellinen & Witczak, 1998). Not only the quantity, but the level of uniformity of the air voids distribution considerably affects the behavior of the mixture (Xu, Chang, Gallivan, & Horan, 2012 Castillo and Caro 2013). However, too much compaction can also be detrimental. Quick and Guthrie (2011) stated that the severity level of compaction impacts strength development in emulsified asphalt mixture. Compaction can contribute to the initial damage of the emulsified asphalt, but also worsen the curing period within these mixtures. Subsequently, it is needed to understand the influence of curing temperature and compaction on the mechanical properties of FDR materials treated with emulsified asphalt and foamed asphalt respectively.

Furthermore, FDR with emulsified asphalt (EA) and FDR with foamed asphalt (FA) technologies are now fully consolidated in practice and witness of numerous studies (Y. Kim & Lee, 2006), and developments over the years. Very little information is available in the literature and in practice on the combined usage of these two techniques. It is believed that with emulsified asphalt, most particles are well coated, which is not the case with foamed asphalt. However, foamed asphalt does work as a binding agent in CRM. As of now, there have been no precise mix design specifications to understand the double coating (combined) technology. Consequently, this can achieved through using the proper approach to develop the mix design and validate the possibility of using both emulsified asphalt and/or foamed asphalt on FDR materials to have better mechanical characteristics. Another aspect of these materials that needs to be studied is their rheological behavior (complex modulus).

1.2 Objectives

The main objective of this research project is to enhance the short term and long-term performance of the Cold Recycled bituminous Materials with asphalt emulsion and/or with foamed asphalt.

The specific objectives of the study are listed below:

- to evaluate the impact of the addition of four different percentages of RAP (50%, 75%, 85%, and 100%), on the rheological behavior of the cold recycled emulsified asphalt materials;
- to establish a performance testing protocol for Cold Recycled bituminous Materials based on the age of the mixture (at the early age of the mix);
- to determine the influence of confinement pressure on the complex modulus of FDR mixtures;
- to evaluate the effect of compaction and curing temperature and duration, on the degree of compaction and on the Marshall stability and indirect tensile strength (ITS) of CRM;
- to compare the behavior of Cold Recycled bituminous Materials treated with foamed asphalt (CRM-foam) and Cold Recycled bituminous Materials treated with emulsified asphalt (CRM-emulsion);
- to determine the mix design procedure for double coating full depth reclamation materials with the addition of four different combinations of the binders (foam and emulsion);

- to evaluate the complex modulus of the double coated full depth reclamation materials with the addition of four different combinations of the binders (foam and emulsion).

1.3 Outline of the Thesis

This Ph.D. thesis is manuscript based, which means that most chapters are published or submitted papers. The outline of this dissertation is as follow:

chapter 1 - Introduction of research problem and objectives;

chapter 2 - Literature Review;

chapter 3 - presents the first published article of this Ph.D. program. The article is titled: Rheological behavior of cold recycled asphalt materials with different contents of recycled asphalt pavements;

chapter 4 - presents the second paper that covers the study of the use of confining pressure when measuring the complex modulus of full-depth reclamation materials;

chapter 5 - presents the third article of this Ph.D. program. The article is titled: study of the impact of the compaction and curing temperature on the behavior of cold bituminous recycled materials;

chapter 6 - presents the fourth paper of this Ph.D. program. The article is titled: effect of binder type on full depth reclamation material behaviour;

conclusions and Recommendations.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

The literature review comprised of many areas of Cold In-place Recycling (CIR) rehabilitation technique, starting from the Reclaimed Asphalt Pavement (RAP) i.e., basic material handling in recycling. This chapter also explains the viscoelastic behavior of bitumen, current practices for cement based treated materials with cold asphalt mixtures, influences of curing time and moisture content on rheological properties of the cold mixtures and mix design procedures for cold mixes using bitumen stabilized materials based on gyratory compacter, and impacts of air voids on characteristics of asphalt mixtures. It includes mechanical behavior and performance evaluation of CIR and FDR mixtures using bitumen stabilized materials.

2.2 Reclaimed Asphalt Pavement (RAP)

Asphalt recycling is not a new concept. The technique was initially developed in 1915, but it started gaining popularity since 1975 because it offers reduced costs; geometric preservation; and conservation of aggregates, binders, and energy (Epps & Allen, 1990).

Sullivan (1996) provided an executive summary of the state of the practice of recycled HMA in 1996. The report reveals that about 45 million tons of recycled asphalt pavement (RAP) are generated each year and 80% of the RAP is reused in highway applications. This makes RAP the most recycled product in the United States, both in tonnage (73 million tons) and in percentage of product recycled (80% of RAP is recycled) (Beyond Roads, 2014). Asphalt rehabilitation projects produce about 100 million tons of RAP per year from millings, presenting a major solid waste concern (Alam, Abdelrahman, & Schram, 2010).

There are two common sources of reclaimed asphalt: reclaimed asphalt concrete (RAP) and recycled asphalt shingles (RAS). RAP comes in the form of lumps and millings. RAS can be obtained as construction waste or manufactured ends. Any of these materials can be crushed and blended, with or without the addition of virgin aggregate, to create blends. Reclaimed asphalt is used as a source of two materials: aggregate and bitumen (Widger. A., 2012). Old asphalt materials can be recycled using cold, warm or hot production methods, and the addition of new binder, asphalt mixture, water or mineral aggregate in the old asphalt can be performed either in the plant or on site. Recycled asphalt can be used for wearing courses, base courses or road bases. Cold and warm methods are mainly intended for roads with low or medium traffic volumes, while hot recycling is also suitable for roads with high traffic volumes (Jacobson, 2002).

From the sustainability point of view, recycling reuses the existing aggregates and RAP binder, thus reducing the need for new materials and the energy it takes to produce them. In addition, recycling can reduce transportation costs and expenses associated with landfilling or storage of the milled material. There are additional environmental and societal benefits of reusing existing resources that are difficult to quantify (McDaniel, Kowalski, & Shah, 2012).

2.2.1 RAP Content in Bituminous Mixtures

Under the current economy, there is an increased interest in using higher amounts of RAP in more applications. As a result, some states are considering expanding and revising their specifications regarding RAP usage. Recently, for example, the Indiana DOT began to allow the use of RAP in surface mixes. The initial allowance for RAP in surface courses permitted the use of 15% RAP in surface courses on roadways with a design traffic level of less than 3,000,000 equivalent standard axle loads (ESALs). In 2010, the specifications were expanded to allow up to 15% by weight of the total mixture for higher traffic categories (over 3,000,000 ESALs). Finally, in the 2012 specifications, the allowable RAP content is expressed in terms of binder replacement (percent of recycled binder as a percentage of total binder in the mix); up to 40% of the total binder can now come from recycled materials

(RAP and shingles) for traffic volumes below 3,000,000 ESALs and 15% for traffic volumes greater than 3,000,000 (McDaniel et al., 2012).

Federal Highway Administration (FHWA) was conducted the survey in 2011; the majority of State highway agencies (more than 40) allow more than 30 percent RAP in the typical hot mix asphalt (HMA) mixture. Considering the cost of pavement materials, it was found that the incorporation of RAP into HMA pavement provides a saving ranging from 14 to 34 percent when the RAP content varied between 20 to 50 percent (Kandhal & Mallick, 1998).

According to Federation of Canadian Municipalities (FCM), (2005), the maximum amount of RAP permitted in hot mix asphalt (HMA) varies somewhat from province to province in Canada. All provinces except Nova Scotia and Prince Edward Island permit RAP to be use in HMA, provided that testing is completed to ensure the quality (penetration/viscosity, or performance grading for Superpave mixture or the asphalt cement) and uniformity of the RAP source and that the reclaimed hot mix (RHM) meets all specification requirement for asphalt concrete. Ontario (Ontario Provincial Standards Specifications (OPSS) 1150) limits the amount of RAP in surface course HMA to 15 percent maximum with 30 percent in conventional binder course mixes and up to 50 percent in certain situations subject to confirmatory testing. Newfoundland allows 10 percent RAP in leveling course only, whereas Québec accepts up to 15 percent RAP in RHM. Alberta and New Brunswick permit higher RAP addition levels (30 percent and 40 percent (±5 percent), respectively). British Columbia, Saskatchewan and Manitoba do not limit the amount of RAP that can be added to HMA.

According to the FHWA RAP Expert Task Group (n.d.) the following table 2.1 provides a list of the locations of high RAP field projects in HMA, the percent RAP used in each project and the dates of construction held in United States of America (USA). In cold and warm recycling, the proportion of recycled asphalt may be up to 100%, while in heated plant recycling the proportion may be 10-40% (Jacobson, 2002).

Table 2.1 High RAP field projects

Location	% RAP	Date of Construction
North Carolina	40%	September 20017
South Carolina	30 and 50%	October 2007
Wisconsin	25%	November 2007
Florida	45%	December 2007
Kansas	30 to 40%	May 2008
Delaware	35%	Summer 2008
Minnesota (MnROAD)	30%	2008
Illinois	10 to 50% allowed	2008

2.2.2 Performance of RAP in Bituminous Mixtures

Many studies are available on performance evaluation with conventional asphalt mixes (mix without RAP). Some studies indicates the performance of pavements with properly prepared recycled asphalt in terms of fatigue, rutting, thermal resistance and durability proved to be satisfactory (Imad L., Elseifi. Mostafa, 2007). Some researchers found that recycled mixes have good resistance to moisture damage at low RAP percentages whereas there is no significant increase in resistance to moisture damage with increase in RAP percentage in mix (Huang, Shu, & Vukosavljevic, 2011) and some studies state that resistance to moisture damage significantly decreases with presence of RAP (Huang et al., 2011). Some researchers found that presence of RAP increases the stiffness of the mix (Aravind & Das, 2007) and decreases in some studies(Huang et al., 2011). Similarly fatigue life increases (Aravind & Das, 2007) and decreases (Rebbechi & Green, 2005) and vary according to the temperature (Puttagunta, Oloo, & Bergan, 1997) Tensile strength increases (Puttagunta et al. 1997 & Watson et al. 2008) or similar to virgin mixes (Huang et al., 2011). Some researcher's state that based on the laboratory testing work carried out on virgin mixes and mixes with 20 % RAP, it was found that addition of RAP improves all the properties of the bituminous mixes.

This indicates that mixes with 20% RAP would perform better than the virgin mixes under similar conditions (Pradyumna, Mittal, & Jain, 2013).

2.2.3 Uses of RAP in bituminous mixtures

From an environmental perspective, it is essential that construction materials, such as RAP, be recycled where possible. The use of RAP will serve as a supplement to natural aggregates in order to conserve natural resources and keep asphaltic concrete out of landfills (Kandhal & Mallick, 1998). To accomplish this, almost all agencies allow the use of reclaimed pavement materials in some form.

2.3 Mix Design for Cold Recycling Mixtures – A Review

All over the world increase in use of Bitumen Stabilized Material(s) (BSM) mixtures in road construction and rehabilitation has created a need for sound guidelines to be established for the laboratory mix design procedures for Cold Recycled bituminous Mixtures (CRM).

2.3.1 Role of Bitumen Stabilized Materials in CRM

From the early 1960s, Bowering (1970); Martin (1976); Acott (1979); Lee (1981); Ruckel et al. (1983) studied the expanded (foamed) asphalt mixtures using virgin materials. Since then, foamed asphalt has begun to be implemented in the FDR process of old asphalt pavement. Other researchers, Hicks (1988); Wood (1982); Al (1983); T. & Wood (1983); v W. & Wood (1983); and Al (1985) have been researched the design procedure and the performance of FDR foam. Maccarrone et al. (1994) introduced a FDR-foam process called FOAMSTAB with advantages such as a cost effective, good fatigue property and rapid curing. In 2002, foamed asphalt was used as a stabilizing agent in full depth reclamation of Route 8 in Belgrade (Marquis, Bradbury, & Colson, 2003). However, there have been limited CIR projects were constructed using foamed asphalt as a stabilizing agent.

In the 1990s, due to the advancement of CIR methods and equipment, the CIR technique has been implemented more frequently using a foamed asphalt process. However, there is currently no nationally accepted method for CIR-emulsion mix design process (ARRA, 2001) and CIR-foam projects (Y. Kim & Lee, 2012). AIPCR and PIARC (2002) published a draft report on CIR of pavements with emulsion (CIR emulsion) or foamed bitumen (CIR foam). However, this report was not intended as a specification; it rather provided information on the applications in different countries. The western United States uses emulsified recycling agents proposed by the Pacific Coast User-Producer Group, among other types of binders according to AASHTO method T-59 as shown in Table 2.2 (Pacific Coast User-Producer Conference, 1989). Cutbacks and soft asphalt cements are used by some agencies (L. E. Wood, White, & Nelson, 1988). The type and amount of diluent should be known by the engineer before any of these liquid asphalts is used (Epps & Allen, 1990).

The amount of BSM places a major role in mix design for CRM, generally the percent of binder for CIR ranges from 0.5% to 3.0% emulsion, with 0.5% to 1.8% suggested by Oregon (Allen, 1988; Allen, Nelson, Thirston, Wilson, & Boyle, 1986) and 1.2 to 1.5 percent in Pennsylvania (Jester, 1987) as starting points for mixture design. Similarly, quantities in the range from 0.5 to 1.5 percent are used for the partial-depth operations, whereas qualities in the range from 1.5 to 3 percent are used for the full-depth operations (Epps & Allen, 1990). Another researcher states that, the typical emulsion content for standard CIR ranges from 1.5 to 2.2 percent (Croteau & Lee, 1997), 0.5 to 2.5 percent (Murphy & Emery, 1996), and 1.0 to 3.0 percent for CIR-foam (Kim, Lee, & Heitzman, 2007).

The addition of virgin aggregate recycled pavement appears to be a widespread standard practice. The reasons cited by different agencies (Epps & Allen, 1990) for adding virgin aggregate include providing additional thickness, correcting gradation (The Asphalt Institute, 1983), and providing for acceptance of additional binder or increase the stability of the recycled mix (Recycling Manual, 1982), sometimes may be to modify RAP characteristics (Widger. A., 2012). The amount of new aggregate ranges from 15 to 50 percent (Jester, R., 1987, Wood et al. 1988) and 27 to 28 percent for CRM (Yao, Li, Xie, Dan, & Yang, 2011). It has been shown that, for CIR processes, the addition of 20 to 25 percent virgin aggregate decreases porosity and improves stability (Murphy & Emery, 1996). Scholz et al. (1990) states that new aggregate typically not used on Oregon projects and not directly considered in the Oregon method of mixture design. The addition of new aggregates may not be necessary in some projects(Jahren & Chen, 2007) in other hand, based on the screening results of the RAP, new aggregates need to be added to CRM (Yao et al., 2011).

Table 2.2 Emulsified recycling agents of the Pacific Coast User-Producer Group (According to AASHTO method T-59)

Test ID	**Test**	**ERA 5**	**ERA 25**	**ERA 75**	**CMS-2RA**	**HFE-200**
T-59	Viscosity at 77^0F	15-100	15-100	15-100	-	-
T-59	Viscosity at 77^0F	-	-	-	50-450	50 min
T-59	Sieve	0.1 max	0.1 max	0.1 max	0.1 max	0.1 max
T-59	One-day storage stability (%)	1 max	1 max	1 max	1 max	1 max
T-59	Residue at 500^0F (%)	60 min	60 min	60 min	60 min	60 min
T-59	Oil Distillate (%)	-	-	-	5-15	0-7
T-59	Charge	+ Pass	+ Pass	+ Pass	+ Pass	- Pass
	Residue Tests					
T-202	Viscosity at 60^0C	200-800	1,000-4,000	1,000-4,000	-	-
T-49	Pen	-	-	-	100-250	200-350
ASTM D4124	Saturate (%)	30 max	30 max	30 max	-	-
T-50	Float Sec 140^0F	-	-	-	-	1200 min
T-44	Solubility (%)	97.5 min	97.5 min	97.5 min	97.5 min	97.5 min
T-240	RFTO Ratio	2.5 max	2.5 max	2.5 max	-	-

2.3.2 Emerging Mix Design for CRM

The method of mixture design based on the Hveem stabilometer, Marshall stability test results, air voids, resilient modulus test, most of the times based on experiences, sometimes field trials, and visual condition of samples to establish the optimum binder content. A standard national method is not available; however, certain basic steps are normally included in the mix design process. These include (Epps & Allen, 1990):

Various steps involved in Mix design:

- Obtaining representative field samples from the pavement or from stockpiles of reclaimed materials;
- Processing of field samples for use in mix design;
- Evaluation of RAP: Asphalt content, Asphalt physical properties (Penetration, viscosity), Aggregate gradation;
- Selection of amount and gradation of new aggregate;
- Estimate the asphalt demand;
- Selection of type and amount of recycling agent;
- Mixture, compaction, and testing of trail mixture, Initial cure properties, Final cure properties, Water sensitivity;
- Establishment job mix formula;
- Adjustment in filed.

Similarly, Figure 2.1 shows the key steps in the mix design of an asphalt mixture (Mallick & Tahar, 2013). Recently, many mix design methods have emerged in an effort to improve the CIR process as a viable method for pavement rehabilitation.

Methods proposed by different agencies and groups that appear to have the most developed mix design procedures for CIR (Epps & Allen, 1990):

- California Test 378;

- Chevron USA, INC. Mix Design Method;
- Corps of Engineers;
- Nevada;
- Oregon Mix Design;
- New Mexico;
- Pennsylvania Mix Design Method.

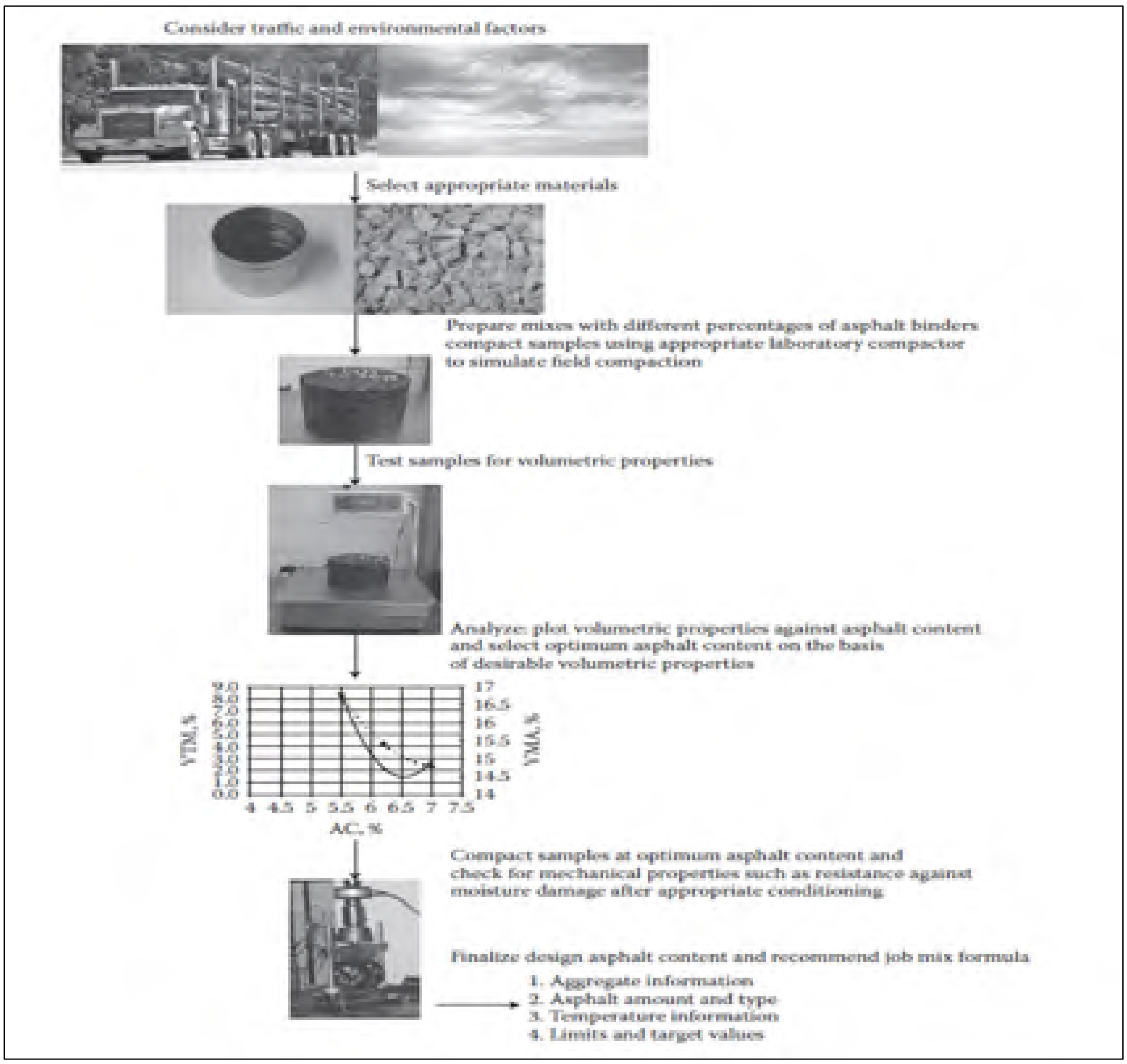

Figure 2-1 key steps in the asphalt mix design

Taken from (Mallick & Tahar, 2013)

- Purdue;
- Texas;
- Indiana (Tia et al., 1983);
- The United Kingdom (Stock, A. F., 1987);
- Ontario (Emery, 1993);
- Israel (Cohen, Sidess, & Zoltan, 1989).

Appendix I summaries the mix design procedures, and the sampling and testing techniques used by some of these organizations (Oqueli, 1997). The methods are generally very similar. All but one uses the Marshall and Hveem tests. Some use kneading or gyratory compaction, while others use the Marshall method. The main differences are in the addition of new aggregate, and in curing time and temperatures.

The Asphalt Institute has recommended the modified Marshall Mix Design procedure for the design of CIR mixes (ASTM D - 1559). Such a design initially involves obtaining samples of the candidate pavement to determine the gradation of the aggregate, the asphalt content, and the penetration and viscosity of the asphalt binder (Kearney & Huffman, 1999). American Association of State Highway and Transportation Officials/the Associated General Contractors of America/American Road and Transportation Builders Association (AASHTO-AGC-ARTBA) Joint Committee Task Force 38 Report (AASHTO Task Force No.38, 1998) contain modified mix design procedures for both Marshall (ASTM D- 1559 or AASHTO T-245) and Hveem (ASTM D-1560 and D-1561 or AASHTO T-246 and T-247) test methods. There is also research underway to adopt Superpave technology to CIR mixtures (ARRA, 2001).

CIR mix design serves as an initial job mix formula, the same as in hot mix asphalt (HMA) construction. Adjustments are generally required for workability, coating, and stability (ARRA, 2001). Most mix design methods for CIR mixes involve the application of asphalt emulsions, emulsified recycling agents or cutbacks as the recycling additive although foamed asphalt and chemical recycling additives have also been used (Acott, 1979b; Mallick & Tahar, 2013).

Three basic theories have been proposed for designing CIR mixes with these recycling additives (AASHTO Task Force No.38, 1998). The first theory assumes that the RAP will act as a black aggregate and the mix design consists of determining a recycling additive content to coat the aggregate. The second theory evaluates the physical and chemical characteristics of the recovered asphalt binder and adds a recycling agent to restore the asphalt binder to its original consistency. The assumption is that complete softening of the old asphalt binder occurs. The third and most prevalent theory is a combination of the first two, where some softening of the old asphalt binder occurs. This theory is referred to as the effective asphalt theory, where the recycling additive and the softened aged asphalt binder form an effective asphalt layer. The degree of softening is related to the properties of the old asphalt binder, recycling additive, and environmental conditions. Because the degree of softening is difficult to quantify, it is recommended that mechanical tests on the CIR mix be a part of all mix designs (Engelbrecht, Roberts, & Kennedy, 1985; Y. Kim, Lee, & Heitzman, 2008).

2.4 Curing Mechanism of CRM

This section is explores some of the few critical findings relative to the curing as published by various researchers around the globe. Due to the vast amount of challenges on curing of bitumen stabilized material (BSM)'s, focus has been applied to aspects that are widely accepted as important parameters to investigate when addressing curing.

The Cold In-place Recycling (CIR) pavement design process involves testing of representative specimens of foamed and emulsified treated materials as means to evaluate pavement performance over time. To adequately acquire representative specimens, it is necessary to condition the materials in a process called Curing (K. J. Jenkins & Moloto, 2008). The Asphalt Institute (1997) reported that inadequate curing can produce high retained moisture contents that would increase the possibility of asphalt stripping and slow the rate of strength development after an HMA overlay is placed. The curing process can be fairly rapid in favorable weather conditions, but high humidity, low temperature, or rainfall soon after CIR placement can increase the curing period significantly.

The Association Mondiale de la Route (AIPCR) and World Road Association (PIARC) (2002) recommend that the application of the HMA overlay should be delayed until the residual water has largely evaporated. This duration should not only depend on the climatic conditions following CIR construction, but also on the traffic level that the CIR layer could support after the completion of pavement construction. Figure 2.2 represent the moisture condition of CIR pavement after surfacing.

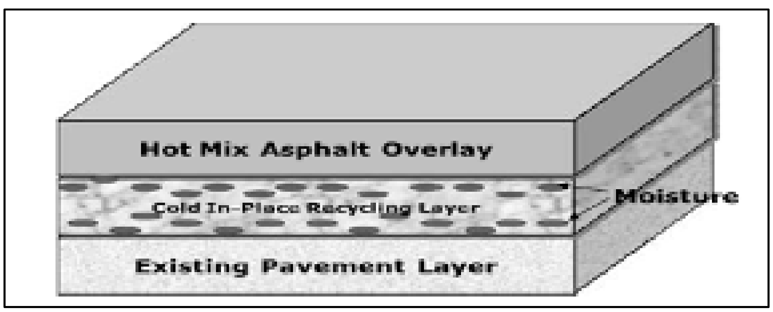

Figure 2-2 Cold in-place recycling pavements in the field
Taken from Kim et al (2011)

Although curing procedures have been adopted in many countries, the curing protocols are varied and an accepted procedure is currently not available.

The lack of representation is due to complex process of curing simulation, as emphasized by the following challenges (K. J. Jenkins & Moloto, 2008):

The complex composition and types of cold mixes to be conditioned in terms of:

- binder type and content;
- active filler type and content;

- aggregate grading and type (porosity, parent rock, petrography);
- binder dispersion within the mix;
- moisture content after compaction;
- voids in the mix and particle orientation (linked to compaction method);
- climate in the area of application (temperature, evaporation and relative humidity conditions);
- mechanical properties;
- time duration since construction that is being simulated;
- service environment: Traffic effects and position of cold mix layer in the pavement structure.

2.4.1 Overview on Curing Procedures of CRM

Following recent research, a certain time period is necessary to allow the recycled mixture to cure and build up some internal cohesion before being covered with a wearing course (AASHTO Task Force No.38, 1998; ARRA, 2001; Bergeson & Barnes, 1998). As a result, curing is a process whereby bitumen stabilized materials gain strength over time accompanied by a reduction in the moisture content. Current practices for accelerated laboratory curing are extremely vast and tend to vary significantly between diverse institutions.

In the aim to address the problem, Marais & Tait (1989) recognized that the material properties of emulsion mixes changed seasonally with significant variation in the first 6 months to 2 years. Another researcher Leech (1994) conducted different studies by Chevron Research Company in California concluded that full curing of cold bituminous mixtures on site may occur between 2 and 24 months depending on the weather conditions. Similarly, a time period of 14 days is typically specified by Croteau & Lee (1997); Committee & No. (1998); Kandhal & Mallick (1998).

The most significant contributions relative to accelerated curing were made by Lee (1981) when he highlighted the following key points:

a) A recommendation that due to the effect of curing on the strength development of foamed mixes, mix design of foamed mixes should be locally based, using information obtained from trial sections;
b) Both curing temperature and the presence or absence of a mould during curing have a direct impact on moisture content of the specimen, which invariable affects mix behavior, particularly the Marshall Stability values.

Lee highlighted the importance of moisture considerations when selecting a curing procedure. Most researchers and mix designers in the period up to the year 2000 had ignored the importance of moisture content of cold mix during curing simulations. Residual moisture contents of less than 0.5% after oven curing at 60ºC were common. Lee's findings mainly highlighted the need to link laboratory curing procedure with a mix property. Consequently, the effects of curing are material property dependent.

The 1999 to 2004 era marked an improvement towards curing procedures. The noticeable curing improvements were mainly driven by CIPR projects including the South African Bitumen Association (Sabita), Germany, USA and Europe. Following Lee's findings, an improvement towards curing temperatures of cold recycled mixes followed, with temperatures of 60ºC being considered too high. The 60ºC curing temperature is above the softening point temperature of the base binder and may cause visual redistribution and dispersion of the bitumen.

Subsequently, the most noticeable improvement followed when a target moisture content equivalent to field equilibrium moisture content (EMC) of the cold mix after curing for a specified period was established (KJ Jenkins, 2000). A summary of the revised curing protocols is presented in Table 2.3.

The influence of active fillers was incorporated in the (SABITA, 1999) guideline where stipulations were made for non-elevated temperature curing. In the case of using cement for emulsion mixes, a 7 day cure at ambient temperature was proposed whilst for no cement mixes a 28 day ambient temperature cure was suggested. In Europe, both Brown & Needham (2000) particularly investigated the influence of cement in emulsion mixes. Findings from their research concluded that, although cement dramatically increases mix stiffness, it does not necessarily repel moisture from the mix.

In 2003, Thanaya (2003) conducted laboratory curing of the cold mix was carried out in an oven set at 40°C. Full curing conditions were achieved when the specimens, following repeated weighing, maintained a constant mass at 40°C. Typically, full curing conditions were achieved within 18–21 days for samples with air void values in the range 8–9%.

Table 2.3 Amended Curing Procedures for Cold Mixes from 1999 to 2004

Curing Method	Equivalent Field Cure	Reference
24 hrs @ ambient + 48 hrs @ 40ºC (OMC<8%) 45 hrs @ 60ºC (OMC>8%)	Emulsion mixes, medium term (1 year field cure?)	(SABITA, 1999)
7 days @ ambient & 28 days @ ambient	Emulsion + cement Emulsion + no cement	(SABITA, 1999)
24hrs @ ambient in mould + 3 days @ 40ºC (sealed)	6 months field cure (foam)	(Asphalt Academy, 2003)
24 hrs @ 40ºC (sealed) + 48 hrs @ 40ºC ambient (unsealed)	Medium term cure (foam and emulsion)	(Robroch S., 2002)
6 hrs @ 60°C (hot summer day) +24 hrs @ 25°C (cool summer night)	short-term and long-term curing, respectively	(K. W. Lee, Brayton, & Harrington, 2003)
24 hrs @ ambient 25ºC (unsealed) + 48 hrs @ 40ºC (sealed)	Long term foamed mix cure (1 to 2 years)	(Houston & Long, 2004)
24 hrs @ ambient (unsealed) + 48 hrs @ 40ºC (sealed) + 3 hrs cooling @ ambient (unsealed)	Medium term cure (foam and emulsion)	(Wirtgen, 2004)
20 hrs @ 30ºC (unsealed) + 2x24 hrs @ 40ºC (sealed – change bag midway)	Medium term cure (foam and emulsion)	Stellenbosch University (2004)

As observed in Table 2.3, almost curing temperatures of 40ºC were commonly used as means to retain field moisture conditions at the end of curing. Low moisture content may be a criterion to evaluate the curing of the mixture. However, such a criterion may be misleading because the moisture content is increased by rain. The material may have built up adequate internal cohesion, but rainfalls may have maintained the moisture content at a high level, incorrectly suggesting that the mixture has not sufficiently cured. As a rule of thumb, whenever a complete core can be extracted from the mat relatively easily, the material has built up enough internal cohesion to be covered (AASHTO Task Force No.38, 1998; ARRA, 2001; Kandhal & Mallick, 1998). Kim et al. (2008) prepared the same CIR-foam specimens for simple performance test and cured in the oven at 40°C for three days. The cured specimens were allowed to cool to a room temperature for 24 hours before testing. According to the test method LC 26-002, an acceptable mix, by Transport Quebec's standards, has a dry Marshall stability of at least 8,000N and a retained stability of at least 60 percent. In this study, all mixes meet those requirements after 2 days of curing except the CIR without cement. The CIR mix without cement was still not acceptable after 5 days. However, the FDR mixes with foamed asphalt failed to meet the requirements at five days, except for the mix with cement (Lachance, Carter, & Tate, 2012).

2.4.2 Recent Developments in Curing Procedure of CRM with Emulsion

Following recent trends in various curing procedures, the need for unified curing procedure method became increasingly apparent. Sebaaly et al. (2004) recommended that the design process should evaluate the early stability of the designed CIR-emulsion mixture by using Hveem stability and resilient modulus.

They evaluated the CIR-emulsion mixtures at three different curing stages: (1) initial curing; (2) final curing; and (3) long-term curing as follows:

- initial curing: Specimens are cured in the mold at 25°C for 15 h;
- final curing: Specimens are extruded out of the mold and are cured in an oven at 60°C for three days;
- long-term curing: Specimens are extruded out of the mold and cured in an oven at 60°C for 30 days.

There exists some variation in the curing temperature and curing time adopted for CIR-emulsion mix design processes in the laboratory (K. W. Lee, Brayton, & Huston, 2002). Cross (2003) adopted two stages of curing for CIR-emulsion mixtures: (1) an initial curing stage; and (2) a final curing stage. Initially, samples were cured for 0 h, 0.5 h, 1 h, and 2 h after mixing. After the initial curing time, they were compacted by using a Superpave gyratory compactor. The compacted specimens were then extracted from the mold and cured in a 60°C oven for 48 h. As mentioned in Table 2.4., Lee et al. (2003) recommended curing periods of 6 and 24 h to simulate short-term and long-term curing, respectively. Curing temperatures of 60°C and 25°C were adopted to represent typical pavement temperatures during a hot summer day and a cool summer night, respectively. Because only the surface of CIR pavement is directly exposed to air in the field, Batista & Antunes (2003) covered all but the tops of some CIR-emulsion specimens with a plastic film to allow water to evaporate through the top surface only. They reported that water content evolution in the field would be between laboratory specimens with and without plastic films. They obtained cores from the site after one year of traffic loading and tested them for resilient modulus. The resilient modulus of CIR-emulsion specimens cured at room temperature for four months (two months with a lateral filmstrip and two months without it) exhibited resilient modulus between 2,000 MPa and 2,500 MPa, which were similar to those of the cores. The specimens cured in the oven at 60°C for three days, however, had lower resilient modulus than the cores. On the basis of the limited laboratory test results, Carter et al. (2007) concluded that accelerated curing at 60°C in an oven for 24 hours seems to be sufficient to achieve a consistent Marshall stability. CIR materials show good rutting and thermal cracking resistance when cured in the oven at 60°C for 48 hours. They reported that overall performance of suggested accelerated curing procedure showed increase in mechanical properties of about 10 to 15 percent. Carter et al. (2008) recognized that for less than 6 hours of air cure for CIR-emulsion samples with cement, the ruts are too deep. If those results were representative of field results, this would mean that a minimum of 6 hours curing period would be needed before opening it to traffic. Feisthauer et al. (2013) reported that the all specimens were oven cured at 38°C for 24 hours. They did this way in order to lose water and thus achieve the early-life field like conditions after the curing and hardening process subsequent to placement and compaction.

Finally, the curing process can be fairly rapid in favorable weather conditions, but high humidity, low temperature, or rainfall soon after CIR placement can increase the curing period significantly(Y. Kim, Im, & Lee, 2011). As mentioned earlier the curing duration should not only depend on the climatic conditions following CIR construction, but also on the traffic level that the CIR layer could support after the completion of pavement construction.

2.4.3 Need for accelerated curing

According to Serfass et al. (2004), in the field, the cold mixes reach their mature level of properties only after a period of time. In temperate climate and under medium traffic, at least one complete cycle of seasons is necessary for the mix to attain such stable condition. The curing time may be longer if the climate is cooler or more humid, the traffic lighter - and conversely. Evaluating cured cold mixes in the laboratory is clearly necessary, but reproducing exactly field curing conditions is too complicated and, above all, time-consuming. An accelerated curing method is necessary.

The requirements are:

- The curing procedure(s) should be as short as possible;
- It must produce materials in a state as close as possible to their in-place mature state;
- It must not cause any significant ageing of the bituminous binder;
- The laboratory equipment should not be too sophisticated.

Mechanisms of curing relate to well defined factors governing curing of bitumen stabilized materials(K. J. Jenkins & Moloto, 2008). As noted in this portion of literature review, most factors driving curing are usually material specific and environmentally linked.

CHAPTER 3

RHEOLOGICAL BEHAVIOR OF COLD RECYCLED ASPHALT MATERIALS WITH DIFFERENT CONTENTS OF RECYCLED ASPHALT PAVEMENTS

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3.1 Abstract

In Quebec, for more than 20 years, cold in-place recycling (CIR) and full-depth reclamation (FDR) have been reliable rehabilitation techniques; restoring pavement condition at an affordable cost with a lower footprint on the environment. Experience reveals that CIR and FDR interventions effectively address the issues of reflective cracking and respect Quebec's Ministry of transportation rutting threshold values. However, despite their commendable performance in the field, the cold recycled emulsified asphalt materials (CRM) has yet to be adequately characterized with respect to their rheological properties. This study was undertaken to evaluate the rheological behavior of the CRM with four different combinations of RAP (50, 75, 85, and 100%). The scope of work for this study consisted of preparing the laboratory compacted CRM specimens, determining the complex modulus (E*) of compacted specimens at various testing temperatures and loading frequencies, analyzing the experimental data with the help of 2S2P1D (2S: two springs, 2P: two parabolic elements, 1D: one dashpot) model and finally, validating the results with pavement design. It was concluded that 100% RAP mixture exhibits extremely high stiffness value at high frequency and low temperature. The results revealed that all four mixtures respect the time–temperature superposition principle with respect to the complex modulus. From a pavement design perspective, the moduli measured in this study do have a big impact. However, since

different pavement structure are achieved with those different materials, the stiffest material, the CIR, ended up giving the least performant structure.

3.2 Introduction

When road networks were rapidly expanding, the initial construction cost was the most important issue, with little or no attention being paid to the ongoing maintenance costs. Since funding for preventive maintenance, preservation, rehabilitation, and reconstruction of roadways will have to compete with other demands on the public purse, innovation is required in order to do more with less. Asphalt recycling is one way of increasing the effectiveness of existing budgets in order to maintain, preserve, rehabilitate and reconstruct more miles of roadway for each dollar spent (ARRA, 2001).There are several methods to recycle asphalt pavements. All over the world, the experience and the choice of technology for In-place recycling vary broadly, mainly Cold in-place recycling (CIR), and Full-depth reclamation (FDR) with the addition of Bitumen Stabilized Materials (BSM) like foamed or emulsified asphalt. Because of the oil crisis of 1973, increased cost of materials like virgin aggregate, asphalt, etc., and a strong desire to preserve effective and sustainable roadway system have fueled a reviving of recycling existing pavement as a primary option.

CIR is a recycling method in which only the existing bituminous materials are recycled. In this method, bitumen is added as an emulsion or foamed, and makes a good base material that needs to be covered with a layer of hot mix asphalt (HMA) or a surface treatment.

CIR is normally performed at a depth of 50mm to 100mm, and it is more frequently used to create a base course, in most cases low-to-medium traffic volume highways (Carter, Feisthauer, Lacroix, & Perraton, 2010; Kandhal & Mallick, 1998; Salomon & Newcomb, 2000). On the other hand, in FDR, both asphalt layer and part of the granular base are recycled at the same time and reconstructed with or without the addition of bitumen. As with CIR, FDR materials need to be covered. It is usually done for depth between 100 mm to 300 mm (ARRA, 2001; Kandhal & Mallick, 1998). CIR and FDR do not have the same mechanical properties because of the different percentages of the constituents in the mix design. A classification was defined by The Bureau de Normalisation du Québec(BNQ) for the different mixes according to the amount of aggregate, reclaimed asphalt pavement (RAP), and cement as illustrated in the (Figure 3.1) (Carter et al., 2010).With that classification, CIR materials are called MR7, and FDR materials are identified as MR5. In this study, the Quebec classification for the Cold Recycled emulsified asphalt Materials (CRM) is used.

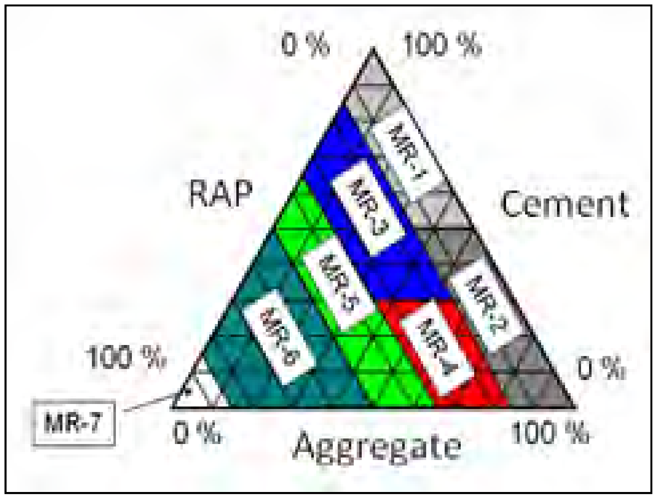

Figure 3-1 Classification of recycled asphalt materials in Quebec
Taken from Carter et al. (2010)

The structural performance of flexible pavement is significantly influenced by the modulus of the asphalt mix layers. Generally, the modulus is affected by the mixture characteristics, the rate of loading frequencies, and pavement temperature. It is also an important constituent in the mechanistic-empirical pavement design (Witczak & Fonseca, 1996).At early stages, the behavior of FDR materials is similar to a granular material, but after the curing phase ends, the behavior is close to a Hot Mix Asphalt (HMA). Therefore, it has been suggested that the FDR materials treated with asphalt binders like emulsion or foam have a time-dependent behavior (Pérez et al., 2013). Hence they can be considered, at some point, to be in between a purely granular material and an HMA. In fact, the binder plays a major role in its structural stability, but the level of air voids are close to the one found in a granular material, around 14 percent (Carter et al., 2008). In addition, Carter et al. (2008) noticed during their study that when a low amplitude compression (2.5 μdef) was applied to a CIR sample, the latter neither compressed nor bounced back into place, in other words, the material did not show purely elastic behavior.

In addition, to decrease the environmental impact, a major advantage of CRM over hot recycling asphalt techniques is the possibility to reuse higher percentages of RAP. In hot-recycling asphalt mixtures, a maximum of 40% RAP is generally accepted in the base layers, and this amount is reduced to 15% or even prohibited in the surface layers. In CRM, the usage of RAP can be as high as 100%, but this generally results in a loss of mechanical properties and durability (Stimilli et al., 2013). Carter et al. (2013) studied and modeled the complex modulus results, related to FDR (50 percent of RAP) and CIR mixes with respect to the 2S2P1D (2S: two Springs, 2P: two Parabolic elements, 1D: one Dashpot) model. Results obtained from the modeling fit on a single curve in the Cole-Cole plan, as predicted. The tested mixes were cured 2 weeks at room temperature (before coring) and an additional two weeks after coring. At 10^{0}C, with respect to higher frequencies, dynamic modulus values were slightly above 10,000 MPa. Also, values of 4415, 3920 and 5565 MPa were obtained for three different FDR materials, respectively that were tested at 21^{0}C and a frequency of 10 Hz after a curing period of 72 hours at 40^{0}C (May, 2008; T. Thomas & Kadrmas, 2003). Stimilli et al. (2013) mentioned that the values of the complex modulus norm measured at medium and high reduced frequencies (medium and low temperatures) showed that cold recycled materials stiffness was considerably lower compared with the conventional hot mix asphalt concrete, reflecting their higher air voids content. Gandi et al. (2015) stated that influence of confining pressure on the complex modulus of the FDR mixtures was mainly on the elastic component. Twagira et al. (2006), studied the flexural dynamic modulus tests were performed on bituminous materials containing 75 percent of RAP. This bending test on an asphalt beam gave a modulus of around 1,500 MPa at 20ºC and 10 Hz. Godenzoni et al., (2015) investigated the cold-recycled mixtures, treated with 2% cement and 3.0% bituminous emulsion and different RAP (0%, 50%, and 80%) contents respectively. They concluded that, the complex modulus values, highlighted that mixtures containing RAP exhibited an asphalt-like behavior (i.e. frequency-dependent and thermo-dependent), whereas the frequency- and thermo-dependence of the mixture containing only virgin aggregate was almost negligible.

3.3 Complex Modulus (E*)

The complex modulus (E*) test is performed to determine the linear viscoelastic (LVE) behavior of asphalt mixtures at various temperatures and different frequencies with respect to a changing phase angle (φ) (Carter & Perraton, 2002; Di Benedetto & De La Roche, 1998). Hence, an asphalt base material, HMA or CRM treated with foam or emulsion, for example, with proven linear viscoelastic behavior, can be characterised by both the phase angle and the corresponding complex modulus. By definition, the complex modulus is the proportionality coefficient between the sinusoidal complex amplitude of the stress, for a given frequency ω, and the sinusoidal amplitude of the strain ε (Carter & Perraton, 2002).

The complex modulus is measured through a direct tension-compression test performed in a loading cell. It has the advantage of being a homogenous test, in other words, the loading applied to the tested sample, results in a uniform distribution of the stress through the entire material and therefore rheological properties can be deducted by measuring the strain. The results obtained from the test are analysed through the 2S2P1D model (Di Benedetto, Olard, Sauzéat, & Delaporte, 2004). It is extensively used to model the LVE unidimensional or tridimensional behavior of bituminous materials which includes binders, mastics, and mixes (Mangiafico et al., 2014; Olard & Di Benedetto, 2003; Tapsoba, Sauzéat, Di Benedetto, Baaj, & Ech, 2013). The 2S2P1D analytical expression of the Complex Young's Modulus, at a specific temperature, as expressed by (Equation 3.1):

$$E^*(i\omega\varpi\tau) = E_0 + \frac{E_\infty - E_0}{1 + \delta(i\varpi\omega\tau)^{-k} + (i\varpi\omega\tau)^{-h} + (i\varpi\omega\beta\tau)^{-1}} \tag{3.1}$$

Where,

- i : complex number defined by $i^2 = -1$;
- ϖ : The angular frequency, $\varpi = 2\pi f$ (f is the frequency);

- h, k: Parameters (constants) parabolic elements of the model ($0 < k < h < 1$);
- δ : dimensionless constant;
- E_0 : the static modulus when ($\omega \rightarrow 0$);
- E_∞ : the glassy modulus when ($\omega \rightarrow \infty$) ;
- β: Parameter linked with η, the Newtonian viscosity of the dashpot,

  $\eta = (E_\infty - E_0)\beta\tau$ When $\omega \rightarrow 0$.
- τ_E and τ_V are characteristic time values, which are the only parameters dependent on the temperature, and have a similar evolution as expressed in (Equation 3.2):

$$\tau_E(T) = a_T(T) \times \tau_{0E} \tag{3.2}$$

Where, $a_{Tref}(T)$ is the shift factor at temperature T and $\tau_E = \tau_{0E}$ at reference temperature T_{ref}. Seven constants (E_{00}, E_0, δ, k, h, β and τ_{0E}) are required to completely characterise the linear viscoelastic properties of the tested material at a given temperature. The evolutions of τ_E were approximated by the William-Landel-Ferry (WLF) model (Ferry, 1980) (Equation 3.3). τ_{0E} was determined at the chosen reference temperature T_{ref}. When the temperature effect is considered, the number of constants becomes nine, including the two WLF constants (C_1 and C_2 calculated at the reference temperature).

$$\log(a_T) = \frac{-C_1(T - T_{ref})}{C_2 + T - T_{ref}} \tag{3.3}$$

If the material has linear viscoelastic behavior, as anticipated, all the results fit on a single curve in the Cole-Cole plan of the model. Also, for a given temperature, known as the reference temperature, with considerations to the principle of time and temperature equivalency, master curves are obtained from the test results and highlight the evolution of the dynamic modulus with respect to a constant reference temperature and a changing frequency.

3.4 Pavement design in Quebec

In Québec, flexible pavement design is mainly done with the CHAUSSÉE 2 program, which is a modified version of the American Association of State Highway and Transportation Officials (AASHTO) 1993 method (Carter et al., 2008). This program uses AASHTO structural design equations, but frost protection variables were added in order to have a pavement that will resist the particular climatic conditions of Québec.

In CHAUSSÉE 2, default values of structural coefficients for commonly used pavement materials can be used. Those structural coefficients were back calculated from Falling Weight Deflectometer (FWD) results obtained in the field. The only two treated recycled materials that are available are the MR5 (*50% of RAP, 50% of VA)* (FDR) with or without cement, and the CIR with emulsion and cement. MR5 is a cold recycled material containing 50 percent of milled asphalt and 50 percent of reused aggregate base.

In most cases, MR7 *(100% of RAP)* (CIR) is a 100 mm thick base layer covered with 50 mm of normal HMA. There is no real structural calculation done in order to evaluate the traffic that new structure will withstand; it's more of an experience based design than a calculated design.

3.5 Objectives

The main objectives of the present study are:

- to evaluate the rheological behavior of the cold recycled emulsified asphalt materials (CRM) with four different percentages of RAP (50%, 75%, 85%, and 100%);
- to evaluate the impact of the measured modulus on pavement design.

3.6 Scope

The scope of work for this study consisted of preparing the laboratory compacted CRM specimens, determining the complex modulus (E*) of compacted specimens at various testing temperatures and loading frequencies, analyzing the experimental data with the help of 2S2P1D model and finally validation with AASHTO Pavement design in Quebec province conditions.

3.7 Test Plan

The samples of CRM tested in this research were prepared in the laboratory using RAP, virgin aggregates (VA), asphalt emulsion, Portland cement, and water. To study the different combinations of cold recycled emulsified asphalt materials, the following four mixes were studied: MR5 (50% of RAP, 50% of VA); MR6 – 75% (75% of RAP, 25% of VA); MR6 – 85% (85% of RAP, 15% of VA); and MR7 (100% of RAP).

The RAP (0-10mm size) used in the laboratory study was obtained from a stockpile in the Montreal area and contained around 3 percent of binder measured in accordance with ASTM D6307-10, (2010). The RAP was homogenized and separated to ensure that all mixes had a similar gradation. The virgin aggregate was an MG20, which is the nominal maximum aggregate size (NMAS) of 20 mm, commonly used as a base material in the construction of flexible pavements in Quebec. The mix design was done in according to MTQ's method LC 26-002 (MTQ, 2001). Samples were compacted using a Superpave Gyratory Compactor (SGC) with the target air void content of 13 percent ±1 percent. After compaction, specimens were extracted from the mould and cured for 10 days at 38 ± 2°C. At the end of the curing phase, samples of 75 mm diameter were cored in the thickness of the specimen's perpendicular to the top surface of the compaction, and saw cut to a length of 120 mm (if required). The gradations and other properties of the mixes used in the tests are summarized in (Table 3.1). Asphalt content is kept constant while ensuring an almost constant RAP gradation across all the samples.

3.8 Complex Modulus Testing

The main objective of this research was to investigate and compare the four different percentages of RAP with CRM. The complex modulus was evaluated using haversine compression loading (stress controlled), with a servo-hydraulic testing system (MTS 810, TestStar II) having a maximum load capacity of 100 kN. The testing setup was equipped with three extensometers, placed 120^0 apart (Figure 3.2), with a measuring base of 50 mm and temperature sensors to monitor strain and temperature variations.

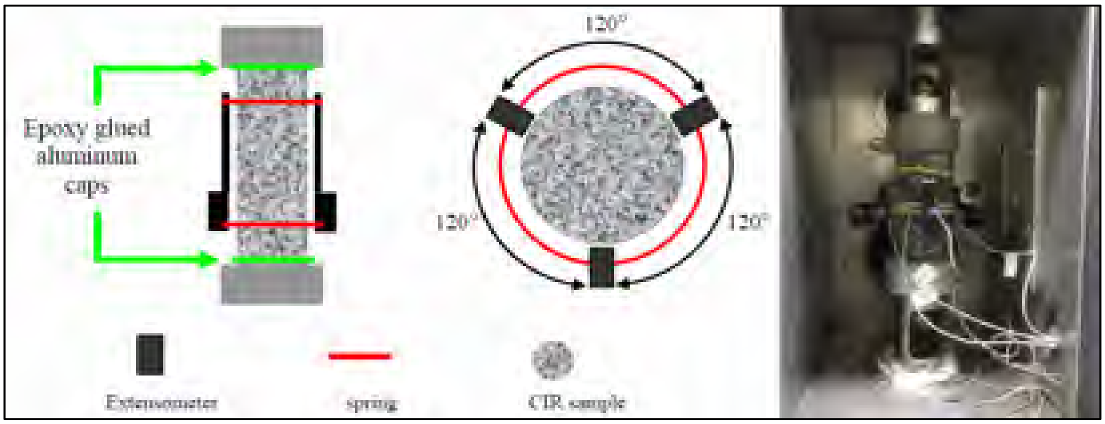

Figure 3-2 Complex modulus test setup

The tests were performed at six different temperatures and (-25^0C,-15^0C,-5^0C, 5^0C, 15^0C, and 25^0C in that order) and at each temperature, a frequency sweep of six different frequencies starting with the slowest one (0.01, 0.03, 0.10, 0.30, 1.00, and 3.00 Hz) was done. For each loading frequency and temperature, the stress level was selected in order to obtain steady-state strain amplitude ranging from 30 to 50 microstrain in compression only. The number of cycles used for the calculation of the modulus and the phase angle changes according to the frequency. A conditioning period of 6 hours was applied before loading after each temperature change.

Table 3.1 Mix Gradation and mix properties

	% of Passing sieve			
Sieve Size (mm)	**MR 7**	**MR6-85%**	**MR6-75%**	**MR5**
20	100	100	100	95
14	100	98	97	89
10	99	96	94	74
5	70	67	66	48
2.5	48	46	45	29
1.25	33	32	31	23
0.630	20	20	20	12
0.315	9.8	11	11	6.4
0.16	4.8	6.2	7.0	3.7
0.080	3.2	4.2	5.0	2.3
% of residual binder in RAP		3.0		
Asphalt Emulsion CSS1P (AC %)		67.4		
Type of Compaction		Superpave gyratory Compaction		
Curing Condition (days)		10 days at 38 ± 2°C		
Added AC (%)	1.8	1.8	1.8	1.8
PCC (%)	1	1	1	1
Water content (%)	5	6.5	6.5	6.5
Total AC (%)	4.8	4.3	4.0	3.3
Gmm	2.531	2.484	2.491	2.535
Gsb	2.167	2.213	2.260	2.311
V_a (%)	14.4	10.9	9.3	8.9

Note: Gmm=maximum theoretical specific gravity of mix; Gsb= bulk specific gravity of mix; V_a= Air voids of the mix; AC=Asphalt Content; PCC=Portland Cement Content; CSS1P= Cationic Slow Setting 1 with Polymer.

3.9 Results and Analysis

This study is aimed at determining the influence of RAP content on the complex modulus of cold recycled emulsion treated asphalt materials with cement. This study was conducted in laboratory prepared samples with 6 different temperatures and 6 different frequencies as mentioned before.The results obtained from the laboratory investigation are analysed through the 2S2P1D rheological model.

3.9.1 Master curve of the tested asphalt mixtures

The complex modulus test results can be plotted a master curve. The master curves were plotted as a function of the equivalent frequency based on the assumption that the asphalt mixtures exhibit the Time–Temperature Superposition Principle (TTSP). Initially, the reference temperature is selected (T_{ref} = 5^{0}C), and then the data at different temperatures are shifted with respect to time in order to obtain a single smooth master curve. The shift factor at temperature T, named $a_T(T)$, used for the construction of the master curve can be determined by means of Equation 3.3. However, both the master curve and the shift factor $a_T(T)$ are needed for a complete depiction of the rate and temperature effects (Singh, 2011). In Figure 3.3, the master curves (complex modulus norm as a function of a frequency) of the four mixtures at a reference temperature (T_{ref} = 5^{0}C) are shown.

In Figure 3.3, the top right portion of the |E*| master curves at higher frequency approaches asymptotically to a maximum value which describes a maximum stiffness value obtained with the MR7 asphalt mixtures. On the other hand, the bottom left apportion of the |E*| master curves at lower frequency approaches a minimum value, which describes a minimum stiffness value, corresponding to the MR5 asphalt mixtures. In addition to this, at a lower frequency and higher temperature, the other two (MR6 - 75% and MR6 - 85%) mixtures exhibit the maximum stiffness value. The higher stiffness of MR7 at high frequency and low temperature may be due to the fact that it contains more RAP binder than the other mixes. This needs to be studied in more details since that mixture does not contain the maximum total binder content. At lower frequencies and higher temperatures, the stiffness value differences between MR7 and MR6 mixtures significantly increases, which possibly depends on the aggregate skeleton. Since the MR5 asphalt mixture has a slightly different gradation than the other mixes, the gradation could explain the difference in modulus value.

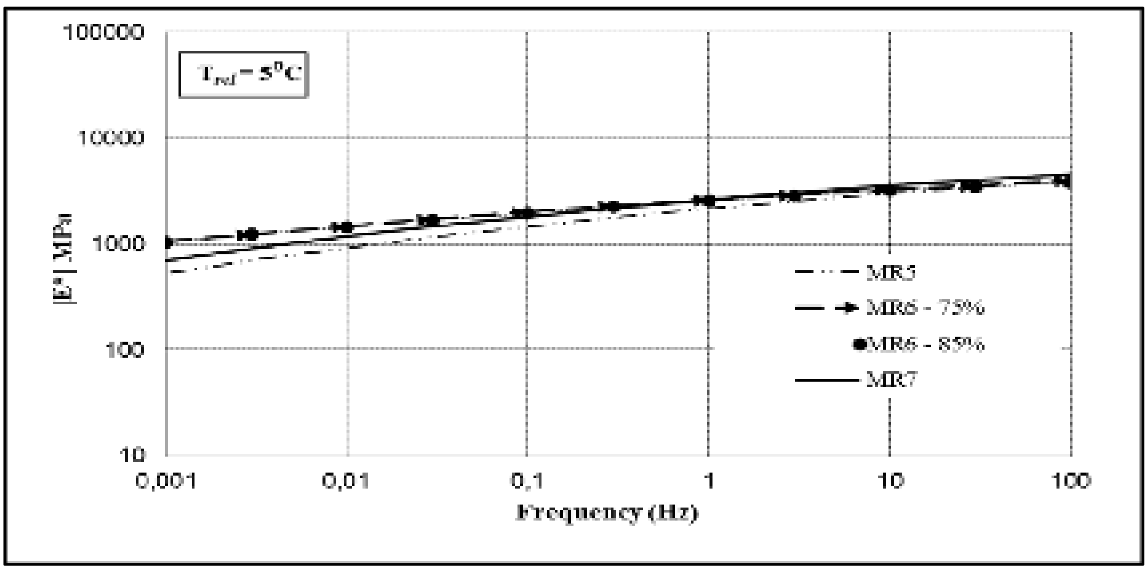

Figure 3-3 |E*| Master curves of the tested asphalt mixtures at Tref = 5°C

3.10 The Cole-Cole plane and Black Space diagram with 2S2P1D model

The 2S2P1D model is generally used to describe the behavior of the asphalt mixtures as well as the binder behavior (Olard & Di Benedetto, 2003). The complex modulus tests were performed on different asphalt mixtures at various temperatures and frequencies to determine the modeling parameters (E_0, E_∞, k, h, β, δ, C1, and C2) included in the 2S2P1D model that characterize the asphalt mixture response in the linear viscoelastic domain. The modeling parameters are presented in Table 3.2. These parameters are determined by obtaining the best-fit curve for the measured complex modulus values plotted in the Cole – Cole and Black space diagrams of the 2S2P1D models are shown in Figure 3.4 and Figure 3.5 respectively. The k, h, δ and β parameters are related to the binder rheology (Di Benedetto, Partl, Francken, & Saint André, 2001; Godenzoni, Graziani, & Perraton, 2016). These parameters are nearly same for all CRM's except β, which means RAP percentage could modify the binder rheology. Regarding the other parameters, E_0 is the static modulus (E when $\omega \rightarrow 0$), and E_∞ is the glassy modulus (E when $\omega \rightarrow \infty$), which is associated with the air void content and aggregate skeleton (Nguyen, Pouget, Di Benedetto, & Sauzéat, 2009). It can be considered that the RAP percentage had an impact on the glassy modulus, which is moderately higher for MR7. This may be due to higher air voids content and higher RAP binder content than the remaining three CRM's.

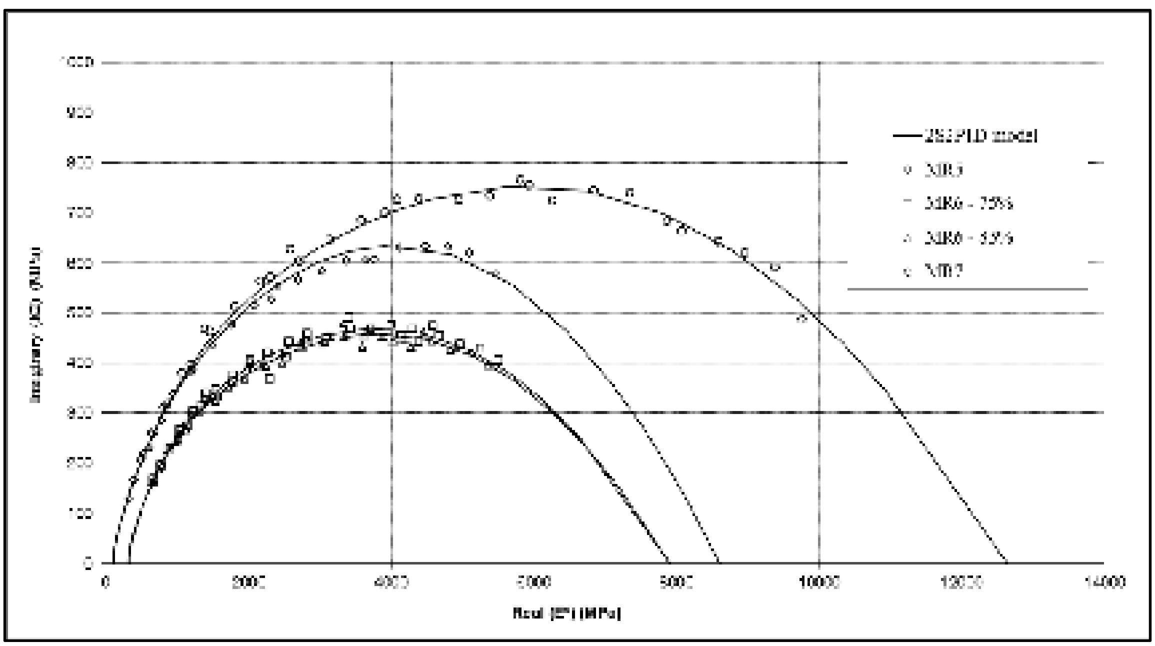

Figure 3-4 Complex module tested asphalt mixtures represented in the Cole-Cole diagram

Figure 3.5 shows black space diagram of 2S2P1D model, the complex modulus norm in function of the phase angle (φ). As observed from the experimental data, the phase angle alternated between 2.86° (low temperature/high frequency) to 22.9° (high temperature/low frequency). Phase angle is the loss coefficient of the material. The larger the phase angle, the more energy is absorbed by the material. So, the material has high φ value would be very viscoelastic and inclined to absorb more energy in cyclic loading, whereas, with less φ values, it absorbs less energy. Although for all tested CRM mixtures the value of both E_0 and φ are well below those commonly measured on HMA (Di Benedetto, Partl, Francken, & Saint André, 2001; Godenzoni, Graziani, & Perraton, 2016). It can be seen that the combination of MR7 and MR5 have higher φ value, than the MR6 - 75% and MR6 - 85% asphalt mixtures. This can be considered that the MR7 and MR5 asphalt mixtures are more viscoelastic materials and in addition to this RAP percentages do have more impact on elastic response than on viscous response.

Table 3.2 Parameters of the 2S2P1D model for the corresponding mixtures (Tref = 5^0C)

Mixture	**E_0 (MPa)**	**E_∞ (MPa)**	**k**	**h**	**δ**	**β**	**C_1**	**C_2**
MR7	100	12625	0.14	0.41	4.0	1500	52.55	325.95
MR6-85%	300	7900	0.14	0.41	4.0	2000	40.15	308.01
MR6-75%	320	7900	0.14	0.42	3.8	1000	41.86	322.52
MR5	80	8600	0.18	0.45	4.0	2000	13.88	96.69

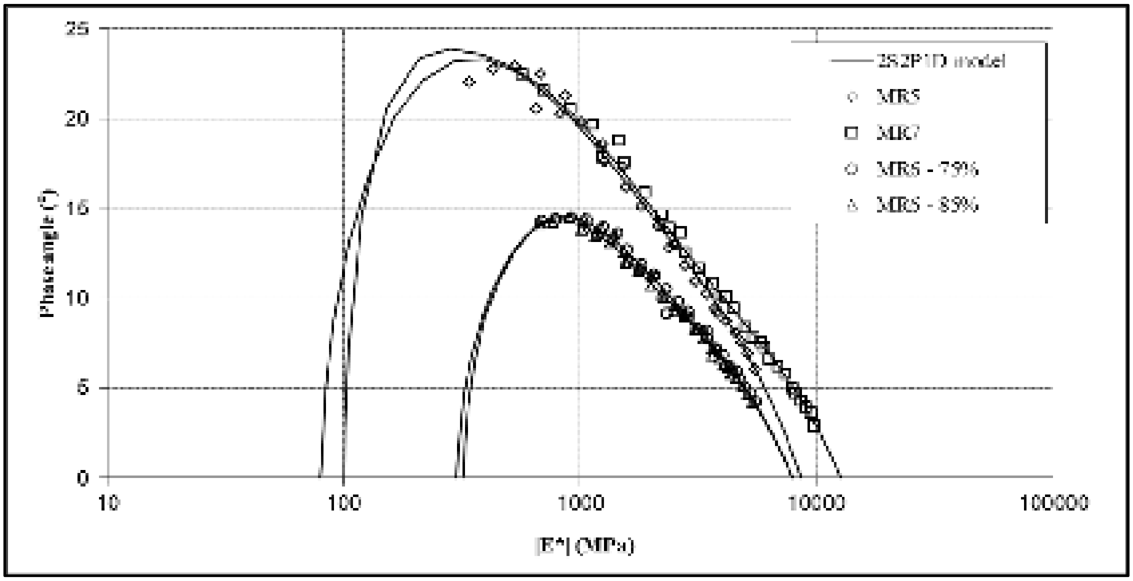

Figure 3-5 |E*| tested asphalt mixtures represented Black space diagram
with 2S2P1D model

3.11 Pavement design

For this part of the study, the program CHAUSSÉE 2 (based on AASHTO 1993) from Transport Québec was used. The goal of this design was to evaluate what kind of traffic increase can be achieved by using CIR. The hypothetical section is located in Montreal and the initial structure is shown in Table 3.3.

With this design, the structure can, according to CHAUSSÉE 2, survive to another 260,000 Equivalent Single Axle Loads (ESALs), compared to the 1,300,000 ESALs for the original design. This was done by applying a factor of 0.6 to the modulus of the HMA layer to take into account the degradation. Unfortunately, with this design, the pavement is still severely cracked and is far from giving a smooth ride. The rehabilitation technique must, at least, eliminate the cracks and, if possible, add structural capacity to the pavement.

Table 3.3 Initial Pavement Design used for Residual Life Assessment

Material	**Thickness (mm)**	**Mr (MPa)**
Cracked Hot Mix Asphalt (HMA)	150	1366
Granular Base (MG 20)	400	198
Soil (GM)	∞	87

The usual rehabilitation with CIR is done with 100 mm of CIR covered with 25 or 50 mm of HMA. For this pavement design, 50 mm of HMA was used. This means that the structure will have a GM soil, a 400 mm granular base, 50 mm of cracked HMA, 100 mm of CIR and 50 mm of new HMA. For the other sections, the recycling thicknesses were chosen according to the percentage of RAP, and 50mm of HMA was added as a surface layer for every section. For example, for the 85% RAP section, the total 150mm of cracked HMA, which is the 85% RAP, was mixed with 26mm of granular base (15%).

The structural coefficients for the four different materials were calculated from the measured complex modulus modelled with 2S2P1D at 20ºC and 10 Hz. With those results, it was possible to do a pavement design with each material. As shown in Figure 3.6, the impact of the modulus, when the pavement design is done with AASHTO, is important. Even if the CIR has the highest modulus, it ended up being the least productive choice because cracked HMA is still present in the pavement. A more complete analysis from a pavement point of view is needed, but with the results obtained here, the best option is the MR6-85%, followed by the MR6-75%, then the MR5 and finally the MR7 (CIR).

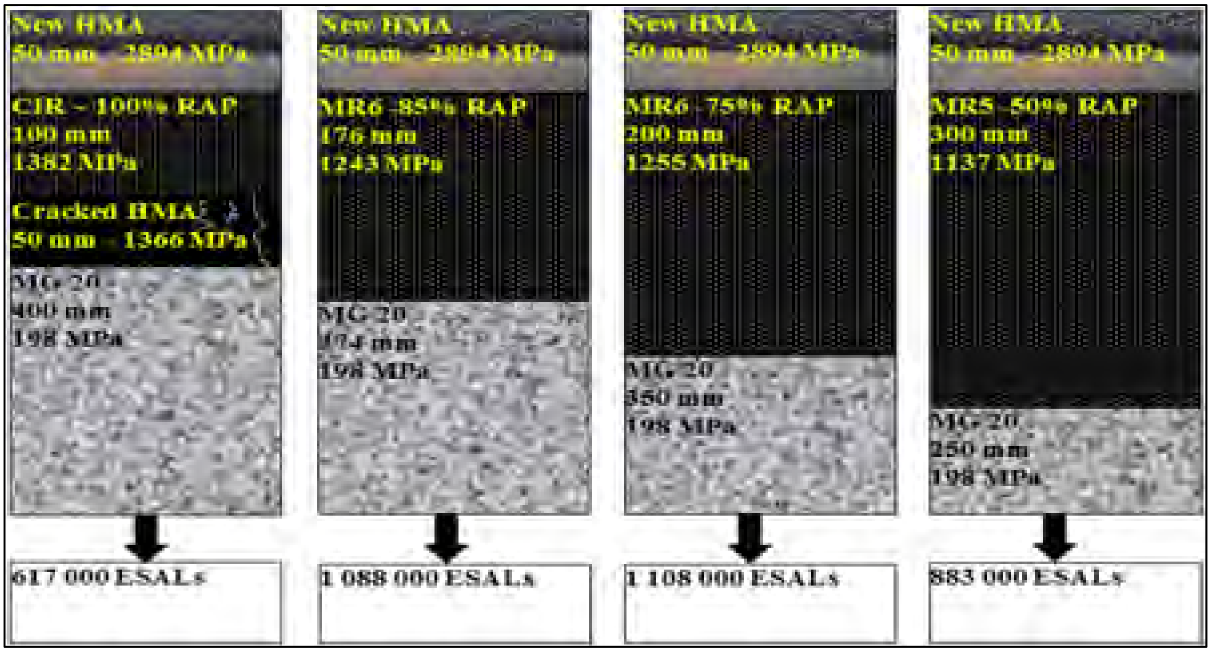

Figure 3-6 AASHTO pavement design with the four different CRM

3.12 Conclusions

The Linear viscoelastic behavior of cold recycled emulsified asphalt mixtures with various percentages of RAP has been analyzed in this study. Complex modulus testing was done on MR5, MR6 - 75%, MR6 - 85%, and MR7 asphalt mixtures with six temperatures and six frequencies respectively. The experimental results considered good since they fit on a single curve on a Cole-Cole plane with 2S2P1D model(Figure 4) as well as on a single master curve plotted at a reference temperature (5°C) using a shifting procedure.

It was observed that MR7 asphalt mixture exhibits high stiffness value at high frequency and low temperature. This can be explained in part by its high total binder content. On the other hand, at lower frequencies and higher temperatures, the stiffness value approaches a limiting value which possibly depends on the aggregate skeleton.

The results revealed that all four mixtures respect the time-temperature superposition principle with respect to the complex modulus. From a consideration of Cole-Cole plane

(Figure 3.4), the RAP percentage had an impact on the glassy modulus is moderately higher for MR7, may be due to higher air voids content and different aggregate gradation than the remaining three CRM's.

However, Black space diagram (Figure 3.5) reveals that the combination of MR7 and MR5 have higher Phase angle (φ) value than the MR6 mixtures. From this consideration, it can be said that the MR7 and MR5 mixtures have higher viscous components than the MR6. This could lead to the conclusion that, contrary to what is found in the literature, the amounts of RAP do not have a strong influence on the phase angle, but more work is needed to support this statement.

From a pavement design standpoint, the moduli measured in this study do have a big impact. However, since different pavement structure are achieved with those different materials, the stiffest material, the CIR, ended up giving the least performant structure. A life cycle cost analysis would be needed to help choose the optimum structure and material. Additional work is needed to do on this aspect.

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CHAPTER 4

STUDY OF THE USE OF CONFINING PRESSURE WHEN MEASURING THE COMPLEX MODULUS OF FULL-DEPTH RECLAMATION MATERIALS

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4.1 Abstract

In Québec, for more than twenty years, Cold In-place Recycling (CIR) and Full-Depth Reclamation (FDR) have been reliable rehabilitation techniques; restoring pavement condition at an affordable cost and a lower footprint on the environment. Experience reveals that CIR and FDR interventions effectively address the issues of reflective cracking and respect Québec’s Ministry of Transportation rutting threshold values. However, despite their commendable performance in the field, Cold Recycled Mixtures (CRM) have yet to be adequately characterised with respect to their rheological properties. In this regard, the study of the mechanical behaviour of a FDR mixture treated with emulsion and containing 50 percent of Reclaimed Asphalt Pavement (RAP) and 50 percent of virgin aggregates were carried out. With respect to the influence of curing on FDR mixtures, samples were subjected to the Indirect Tensile Strength test considering five different curing protocols. Rheological properties were investigated through a complex modulus test conducted at different confining pressures. Because of limited test results, it was concluded that, a longer curing period would produce higher tensile strength and a significant increase in the resistance to moisture damage. The confining pressure shows significant influence on the complex modulus of FDR mixtures.

4.2 Introduction

As a result of an aging road network infrastructure, interventions to restore the pavement condition carried out in order to increase the user's riding comfort and/or the road safety requirements are very common in Canada. Since the 1970s, rehabilitation treatments as opposed to new constructions have been gaining momentum. In fact, increasing prices of asphalt based materials and reduced public resources to fund infrastructure projects drove the evolution of rehabilitation technics in terms of acceptancy, effectiveness and practical understanding in the pavement industry. In addition, lately, growing concerns over human impacts on the environment adds another dimension to rehabilitation considerations. In this regard, rehabilitating structurally unsound flexible pavements through full depth reclamation (FDR) treatment has been a cost effective technic, used in Québec since the 80s. With respect to sustainable development aspects, FDR produces a lower footprint on the environment as the amount of virgin aggregate required is lower than in a reconstruction intervention and the levels of greenhouse gas and energy consumption are substantially reduced. Finally, in terms of effectiveness, FDR interventions have been able to address reflective cracking while rutting performance has been satisfactory as per Québec's Ministry of Transportation threshold values (Bergeron, 2005).

The intervention is performed through what is typically known as the recycling train. The old pavement and a predetermined depth of the granular foundation are milled, corrected to desired grading, mixed with a binder, usually foam or emulsion, and compacted before traffic starts circulating on the rehabilitated pavement structure. To achieve a satisfactory stiffness, moisture content in the FDR mixes needs to be evacuated from their structure, this process is known as the curing phase. Once the curing period is completed, a hot mix asphalt (HMA) layer or surface overlay is placed on top of the compacted FDR material due to their high water sensitivity and protect it from the traffic loads.

However, despite all the benefits of FDR interventions, satisfactory research is yet to be done with regards to gaining further comprehension of their rheology. In fact poor understanding of the mechanical behavior of this type of material has been a source of reluctance to use them as road base materials (Depatie, J. Bilodeau, G., and Gold, 2012). Rheology is the science that studies behavior laws of material by linking their stress and strain for a specified temperature and loading frequency. As of today, attempts to characterize the stiffness of FDR materials through a triaxial test, by measuring the resilient modulus (M_R), or through a complex modulus (E*) test have been undertaken.

4.3 Background

4.3.1 Mechanical behavior of FDR materials

At early stages, the behavior of FDR materials is similar to a granular material, but after the curing phase ends, the behavior is close to a HMA. Therefore, it has been suggested that the FDR materials treated with asphalt binder like emulsion or foam, have a time dependent behavior (Pérez et al., 2013). Hence, they can be considered, at some point, to be in between a purely granular material and a HMA. In fact binder plays a major role in its structural stability, but the level of air voids are close to the one found in a granular material, around 14% (A Carter et al., 2008). In addition, Carter et al. (2008) noticed during their study that when a low amplitude compression (2.5 μdef) was applied to a Cold-In-Place Recycled (CIR) sample, the latter neither compressed nor bounced back into place, in other words, the material did not show a pure elastic behavior.

It has been widely covered that hot mix asphalt (HMA) is a viscoelastic material subjected to temperature and frequency sensitivity (Carter & Perraton, 2002). Whereas, granular material have an elasto-plastic response to loading, independent of temperature or loading frequency. So far, many findings concluded that FDR materials stabilised with a binder like foam or emulsion have a viscoelastic behavior (Carter et al., 2013; Pérez et al., 2013) after curing. Locander, (2009), explained that granular and FDR materials have a distinctive behavior due to the presence of binder, and coating FDR's aggregates. In fact, for a similar level of stress and an adequate compaction, FDR materials (without binder) have a stiffness comparable to a granular material. Hence, it can be inferred that FDR materials with binder should have higher stiffness due to more cohesion, resulting from the binder effect (Depatie, J. Bilodeau, G., and Gold, 2012). Jenkins et al. concluded that, in comparison to an equivalent granular material, inclusion of binder (foam), in cold recycled mixes (CRM), resulted in greater cohesion. Santagata et al. (Santagata et al., 2010b) reported that when properly designed, CRM, in the long-term, can achieve stiffness values comparable to those obtained for a HMA mixture. Therefore, Perez et al. (Pérez et al., 2013) explained that treating FDR materials, which are stabilized with a binder, as a granular material is unrealistic. Also, there is a persistent gap between predicted life to observation in the field with respect to FDR layers in flexible pavement structures.

Due to a time dependent behavior, inherent to CRM materials, Carter et al.(Carter et al., 2013) acknowledged the challenge related to measuring the stiffness of CRM, given the variation that occurs depending on the considered curing protocol. Stiffness values of FDR mixes can be determined through a resilient or a dynamic modulus test. Santagata et al., (2010b) investigated the short term stiffness of FDR mixes by using a triaxle cell. Considering short term curing protocol (1 to 2.5 hours at respectively 20, 40 and 60^{o}C) M_R values obtained through the study are indicated in Table 4.1.

Carter et al., (2013) studied and modeled the complex modulus results, related to FDR and CIR mixes with respect to the 2S2P1D model. Results obtained from the modeling fit on a single curve in the Cole-Cole plan, as predicted. The tested mixes were cured 2 weeks at room temperature (before coring) and an additional two weeks after coring. At 10^{o}C, with respect to higher frequencies, dynamic modulus values were slightly above 10,000 Mpa. Also, values of respectively 4415, 3920 and 5565 MPa were obtained for three different FDR materials tested at 21^{o}C for a frequency of 10 Hz after a curing period of 72 hours at 40^{o}C (May, 2008; Thomas & Kadrmas, 2003).

Table 4.1 Curing Procedures for FDR Mixes
Taken from Santagata et al. (2010b)

Temperature (^{0}C)	**Curing protocol**	**M_R range (MPa)**
20	Short term	177.6 to 625.0
40	Short term	172.4 to 631.2
60	Short term	163.7 to 427.7

4.3.2 Effect of confinement on FDR mixes stiffness

The complex modulus test is performed within low strain and stress values in order to preserve the linear viscoelastic behavior of the material. However, measuring asphalt based material stiffness, whether hot or cold mixes, at low stress and strain does not allow the properties of the granular structure to be taken into account to a full extent (Kim, 2008). With regards to HMA mixes, as an attempt to address these limitations, during his study Kim Y. (2008) conducted dynamic modulus testing on HMA samples and simultaneously considering low and high levels of deviatoric stress for different levels of confinement. He assumed that confinement (138 to 206 kPa) with high levels of vertical stress (up to 552 kPa) would mobilise the internal friction angle. However, results indicated that no improvement with respect to rutting resistance. In their study Shu and Huang (Shu & Huang, 2008) evaluated the dynamic modulus of HMA sample by applying frequencies ranging from 0.1 to 25 Hz, three different temperatures 10, 25 and 54^{0}C and three different confining pressures which are 0, 103.5 and 207 kPa respectively. In their findings, it is shown that for low temperatures, where the binder is very stiff, aggregates were tightly bonded and the effect of confining pressures was noticed to be minimal. Whereas for higher temperatures, as the binder softened hence inducing less cohesion among the aggregates, the confining pressure effect was much more significant, resulting in higher stiffness than predicted.

A very limited amount of research has been done on the influence of confining pressure on FDR mixtures. However, Yan et al.,(2014) performed dynamic modulus testing on Asphalt Emulsion Cold Recycled Mixtures (AECRM) and found a frequency and temperature sensitivity on the dynamic modulus, as mentioned earlier, but none from the three confining pressures considered during the testing (0, 100 and 200 kPa).

4.4 FDR mixes stiffness evaluation test

4.4.1 Complex Modulus E*

It is possible to model a linear viscoelastic material by analyzing the evolution of its complex modulus (E*) with respect to a changing phase angle (φ) (Carter et al., 2008; Di Benedetto & De La Roche, 1998). Hence, an asphalt base material, HMA or FDR treated with foam or emulsion, for example, with a proven linear viscoelastic behavior, can be characterized by both the phase angle and the corresponding complex modulus. By definition, the complex modulus is the proportionality coefficient between the sinusoidal complex amplitude of the stress, for a given frequency ω, and the sinusoidal amplitude of the strain ε (equation 4.1) (Di Benedetto & De La Roche, 1998).

$$E^*(\omega) = \frac{\sigma_0}{\varepsilon_0} e^{i\varphi(\omega)} = |E^*| e^{i\varphi(\omega)} \tag{4.1}$$

The complex modulus can be written as a vector (equation 4.2), representing the elastic component of the complex modulus E_1 and the viscous component of the complex modulus E_2. Derived from that expression, the dynamic modulus is the norm $|E^*|$ of the complex modulus (equation 4.3).

$$E^* = E_1 + iE_2 \tag{4.2}$$

$$|E^*| = \left({E_1}^2 + {E_2}^2\right)^{1/2} \tag{4.3}$$

The complex modulus is measured through a direct tension compression test performed in a loading cell. It has the advantage of being a homogenous test, in other words, the loading, applied to the tested sample, results in a uniform distribution of the stress through the entire material and therefore rheological properties can be deduced by measuring the strain. The results obtained from the test are analysed through the 2S2P1D (2S: two springs, 2P: two parabolic elements, 1D: one dashpot) model. If the material has linear viscoelastic behavior, as anticipated, all the results fit on a single curve in the Cole-Cole plan. Also, for a given temperature, known as the reference temperature, with considerations to the principle of time and temperature equivalency, master curves are deduced from the test results and highlight the evolution of the dynamic modulus with respect to a constant reference temperature and a changing frequency.

4.4.2 Resilient modulus (M_R)

Resilient modulus (M_R) evaluation is usually applied to granular materials, but is also used to characterise the asphalt based materials. Characterizing granular materials is difficult, as a result, Robert, et al., observed that granular materials are simply considered as homogeneous and isotropic. Generally speaking, the use of resilient modulus only applies if the loading conditions remain in the elastic domain (St-Laurent, 2014). Therefore, with respect to characterizing granular or asphalt based materials, with their M_R, unrecoverable portion of the strain (ε_p) resulting from loading is negligible in comparison to the recoverable one (ε_R). For that reason, the elastic analysis is limited to two parameters which are M_R and the resilient Poisson coefficient (ν_R). The resilient modulus is expressed as a ratio of the deviatoric stress σ_d and the resulting recoverable strain (equation 4.4).

$$M_R = \frac{\sigma_d}{\varepsilon_r} \tag{4.4}$$

Materials composing the pavement structure have their respective M_R. According to St-Laurent, (2014), the M_R variations results from the intrinsic characteristics of each material such as the level of compaction, the water content and the applied stress. With respect to asphalt based materials in particular, the M_R value is subjected to temperature and loading duration sensitivity. The M_R test is conducted within a triaxial cell and various modeling approaches apply with respect to the mechanical behavior of each material (St-Laurent, 2014). A summary of the typical models, considered by Québec's Ministry of Transportation design approach is indicated in Table 4.2.

Table 4.2 Typical models for pavement materials
Taken from St-Laurent (2014)

Name of the model	Equation	Explanation
Asphalt based materials	$M_R = 10^{(K1-K2T^{K3})} + K_4$ «+ K_4» only if height of the layer >150 mm	The resilient modulus is expressed as a function of temperature T, and K_1 to K_4 are M_R modeling constants.
K-theta	$M_R = K_1\theta^{K2}$ Where $\theta = \sigma_1 + \sigma_2 + \sigma_3$	The resilient modulus is expressed as a function of the total confining stress θ.

Based on the values suggested by Québec Ministry of Transportation in their pavement design software, *Chaussée 2*, theoretical resilient modulus of a FDR layer can be plotted as a function of the total stress θ considering regression coefficient K_1=16.9, and K_2= 0.6. As indicated on Figure 4.1. The total stress on the sample has an impact on the resilient response of the material.

4.5 Objectives

Considering the time dependency of FDR mixes, the objectives of this study were to obtain a better understanding of the impact of curing on FDR samples and evaluating their rheological properties.

Specifically:

- to evaluate the impact of curing condition on the ITS values and moisture sensitivity of FDR mixtures.
- to determine the influence of confinement pressure on the complex modulus of the FDR mixtures.

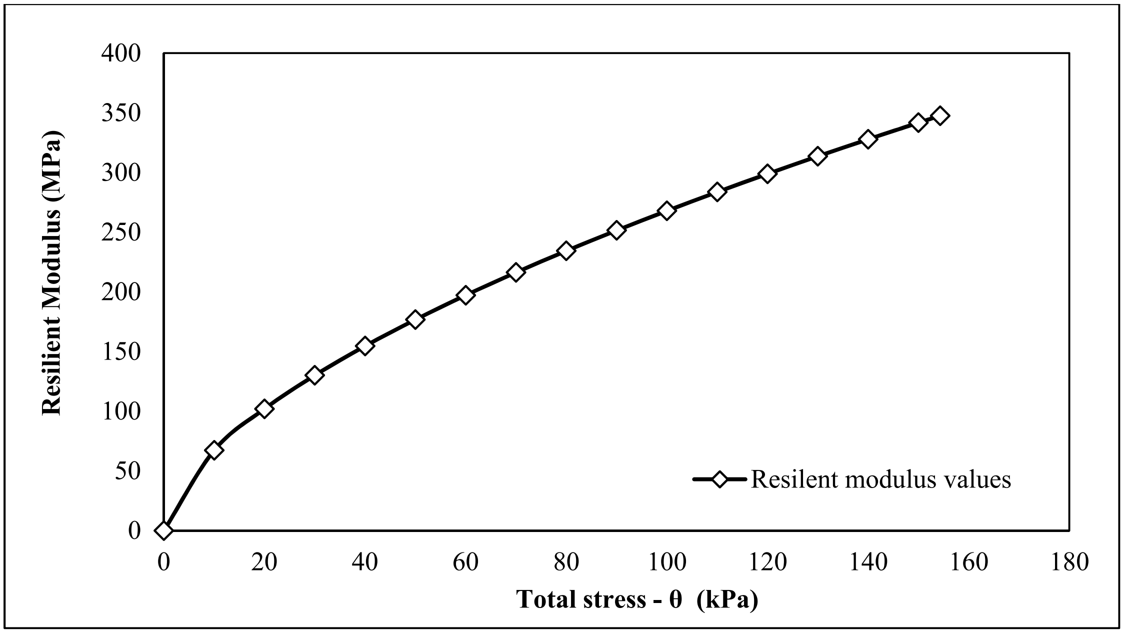

Figure 4-1 Theoretical Resilient Modulus Values of a Full Depth Reclamation Layer

4.6 Methodology

4.6.1 Methodology

Fabricating the samples in the laboratory, which resemble an actual FDR layer in the field, is difficult due to the significant material variability within road sections. The samples of FDR tested in this study were prepared in the laboratory using RAP, virgin aggregates, bituminous emulsion, Portland cement and water. The Reclaimed Asphalt Pavement (RAP) used in the laboratory study was obtained from a RAP stockpile in the Montreal area, Canada, had a maximum size of 14 mm and contained around 4.76 percent of binder measured in accordance with ASTM D6307-10, (2010). The RAP was homogenized and separated to ensure that all mixes have a similar gradation. The virgin aggregate was a MG20 which is the 0 mm to 20 mm aggregate normally used in Quebec as a base material for highways. The gradations of RAP, virgin aggregate (MG20) as well as the FDR mix gradations are shown in Figure 4.2.

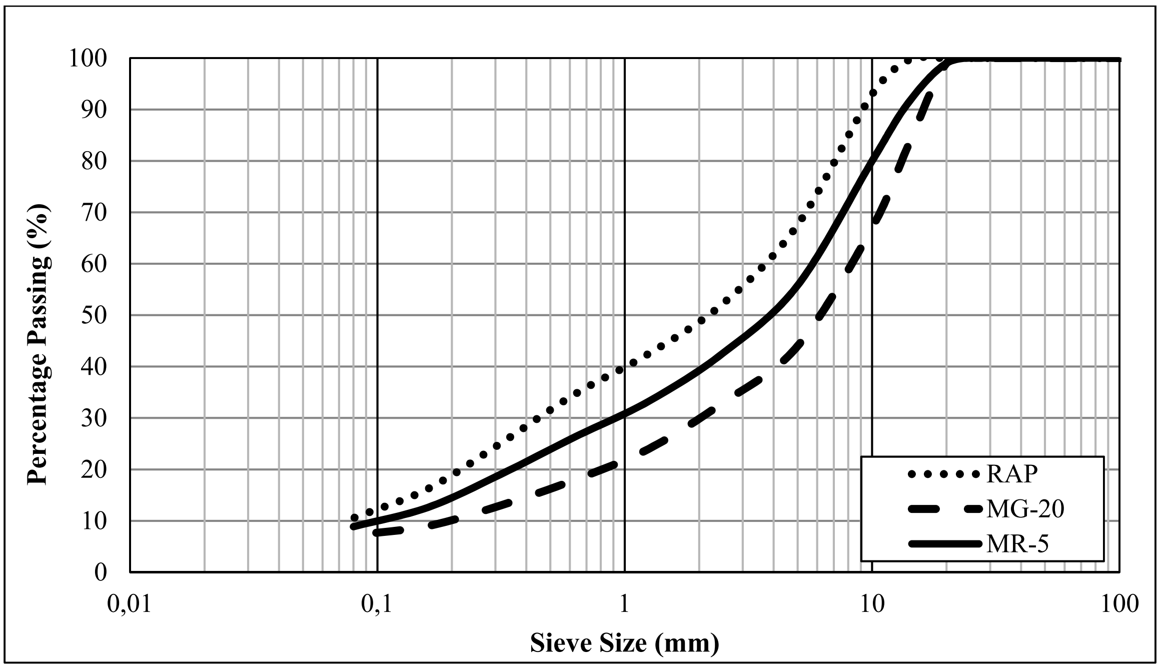

Figure 4-2 Particle size distribution of the Reclaimed Asphalt Pavement (RAP), virgin aggregate (MG-20), and the Full Depth Reclamation (MR-5) mix

The FDR mixtures tested in this study were made of 50 percent of RAP and 50 percent of virgin aggregate (MG20). Cationic Slow-Setting with Polymer asphalt emulsion (CSS1P) was employed. The base asphalt content of the emulsion was 69.2%. The cement used as active additive was Portland cement type 10. For emulsified cold recycled mixtures, 1.0% cement (by dry mass of RAP and MG20 mixtures) and 2.6% of binder was added in the blend. The emulsion mixtures used a pre-mix water method. The pre-mix optimum water content was 6.5% for the weight of the total dry mass including cement. Pre-mix water has several advantages, including higher levels of RAP and virgin aggregate coating, better lubrication of the mixture during compaction, and accelerating the reaction of cement hydration (Yan, Ni, Yang, & Li, 2010). The percentage of asphalt emulsion and optimum water content matched field levels in order to represent the FDR mixtures placed in the field status.

4.6.2 Specimen Preparation

The Mix design was done in according to MTQ's method LC 26-002, Méthode de formulation à froid des matériaux recyclés stabilisés à l'émulsion (MTQ, 2001). For each mix, 30 replicates were compacted for five different curing periods at two different curing conditions (wet and dry respectively for indirect tensile testing). For complex modulus testing, six replicates were compacted for each mix in two different conditions (with and without confinement, respectively). Before mixing and compaction, virgin aggregates and RAP were dried at 110 and 60°C, respectively. The dry virgin aggregate and RAP blend was preliminarily mixed with cement. Afterwards, each blend was thoroughly mixed, eventually adding water and emulsion. After mixing by hand for no more than two minutes, a visual evaluation was made to check for homogeneity and to verify that emulsion did not break. The loose mix was compacted into specimens having 100 and 150 mm diameter, for Indirect Tensile Strength (ITS) and complex modulus testing respectively, using a Superpave Gyratory Compactor (SGC) with a constant pressure of 600 kPa, a gyration speed of 30 rpm and a constant angle of inclination of 1.25 degrees. For each sample, the weight of the loose mixture, about 1.2 and 5.7 kg (for ITS and complex modulus, respectively). And it was adjusted to attain an air void content of 13 percent ±1 percent and a specimen height of 63.5 ± 2.0 mm for ITS, and 140 mm for complex modulus according to LC 26-003, Détermination de l'aptitude au compactage des enrobés à chaud à la presse à cisaillement giratoire (MTQ, 2014). After compaction, specimens were sufficiently stable to allow immediate extrusion.

Once the specimens were compacted, they were cured for 2 days at 60 ± 2^{o}C before being cored for complex modulus testing. Samples of 80 mm diameter were cored in the thickness of the specimen's perpendicular to the top surface of the compaction, and saw cut to a length of 140 mm (if required) as shown in Figure 4.3.

(a) (b)

Figure 4-3 Original Gyratory Specimen (a) and
Cored Specimen of 140 mm x 80 mm (b)

The curing protocol was fixed and controlled to study the evolution of the material properties as a physical consequence. A summary of the curing periods is presented in Table 4.3. Compacted specimens were initially cured in a climatic chamber at 25°C for 24 hours to allow water to evaporate, followed by curing in the oven at different temperatures as per curing protocol. Prior to testing the cored specimens were kept in a sand box at room temperature.

Table 4.3 Curing Procedures for Full Depth Reclamation (FDR) Mixtures

<table>
<tr><th rowspan="2">Protocol Number</th><th colspan="2">Curing Period</th></tr>
<tr><th>Indirect Tensile Strength</th><th>Complex Modulus</th></tr>
<tr><td>1</td><td>1 day at 25^{0}C</td><td rowspan="5">2 days at 60^{0}C ± 2^{0}C</td></tr>
<tr><td>2</td><td>1 day at 25^{0}C and 1 day at 33^{0}C ± 2^{0}C</td></tr>
<tr><td>3</td><td>1 day at 25^{0}C and 2 days at 33^{0}C ± 2^{0}C</td></tr>
<tr><td>4</td><td>16 hrs. at 25^{0}C and 2 days at 60^{0}C ± 2^{0}C</td></tr>
<tr><td>5</td><td>3 days at 60^{0}C ± 2^{0}C</td></tr>
</table>

4.7 Experimental Method

4.7.1 Indirect Tensile Strength test

Asphalt materials may be sensitive to the presence of water in the finished pavement. Water will cause the binder to not adhere to the aggregate. Since the binder is the glue that holds the pavement together, rapid failure of the pavement can be expected if the binder cannot adhere to the aggregate. This is often referred to as stripping. The stripping resistance of asphalt mixtures is evaluated by the decrease in the loss of the ITS according to AASHTO T283 test procedure. In this test, specimens are subjected to compressive load at a constant rate of 51mm/min acting parallel to and along the vertical diametrical plane of the specimen through two opposite loading strips. This type of loading develops a tensile stress which acts perpendicular to the direction of applied load and along the vertical diametrical plane, and the specimen usually fails by splitting along with the loaded plane (Behiry, 2013; Nejad, Azarhoosh, Hamedi, & Azarhoosh, 2012) as shown in Figure 4.4. The recorded maximum compressive strength is divided by appropriate geometrical factors to obtain the ITS using the following equation 4.5 (AASHTO, 2003).

$$ITS = \frac{2000 \times P}{\pi \times H \times D} \tag{4.5}$$

Where,

ITS : Indirect Tensile Strength (kPa);

P : Maximum load (N);

D : Diameter of the specimen (mm); and

H : Average height of the specimen (mm)

The level and the extent of moisture damage, also called moisture susceptibility, depends on environmental, construction, and pavement design factors; internal structure distribution and the quality and type of materials used in the asphalt mixture (Behiry, 2013). Moisture susceptibility of the compacted specimens is evaluated by Tensile Strength Ratio (TSR) which is calculated by dividing the average conditioned ITS result by the average unconditioned dry ITS result (Behiry, 2013).

Figure 4-4 Indirect Tensile Test Loading

4.7.2 Complex modulus test

For testing the complex modulus, the cured specimen was installed in the testing machine at an age of 16 days due to a deficiency with the apparatus. As per Figure 4.5, the testing was performed with a MTS810 press, with a compression loading only for a target strain of 50 μdef. The testing setup was equipped with extensometers and temperature sensors to monitor strain and temperature variations. The process was carried out in two phases at ambient temperature 23 ± 1^{0}C. The two phases consisted of testing the material with respect to two different frequencies of 0.03 and 1 Hz. During each loading condition described, confining pressure of 0, 20, 50, 75, 100 and 115 kPa were applied on the sample.

Confinement pressures were assumed based upon a Boussinesq’s stress distribution model from the middle of the FDR layer (depth of 0.150 m), as achieving constant confinement pressure above 115 kPa was not possible with the testing apparatus. The model pavement structure consisted of a HMA layer of 50 mm and a FDR layer of 200 mm, a tire pressure of 552 kPa, an axle load of 80 kN (20 kN/tire), and a contact area radius of 0.107 m the horizontal stress variation was plotted as a function of depth (Figure 4.6).

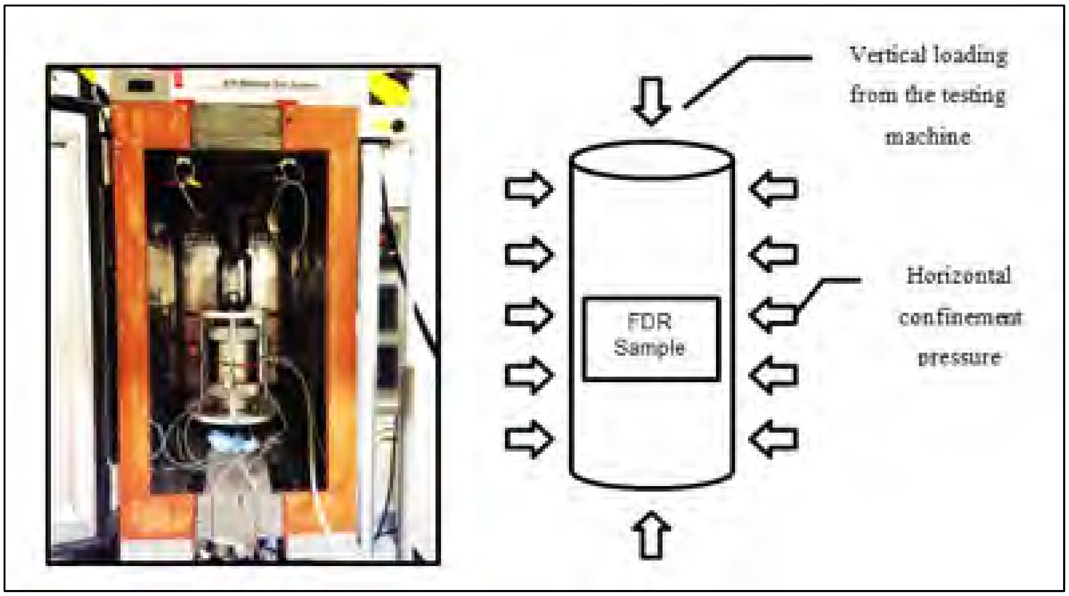

Figure 4-5 Complex Modulus Test with Confinement Pressure Cell

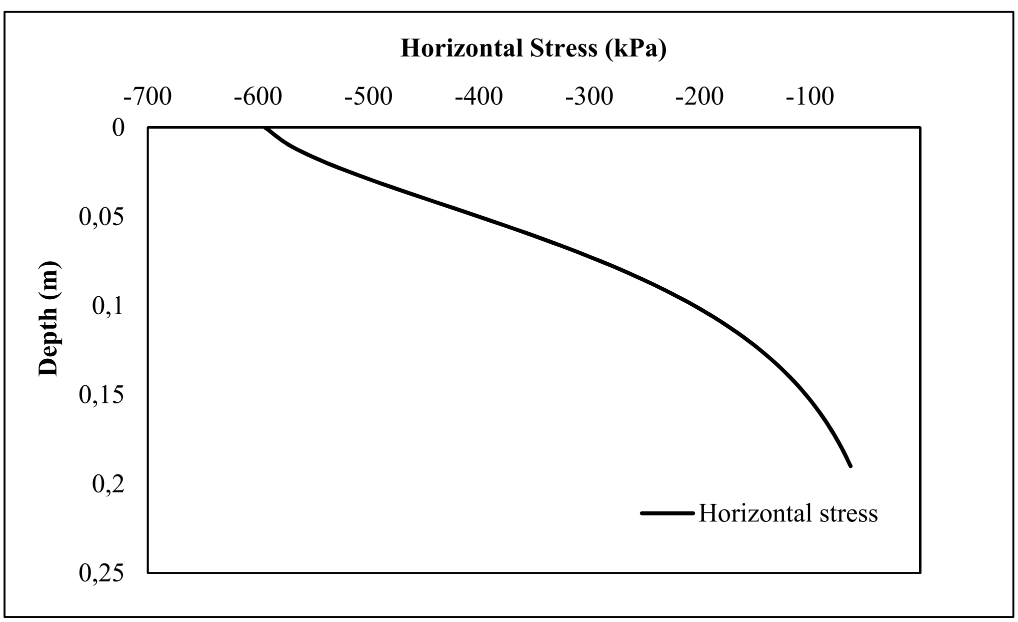

Figure 4-6 Horizontal stress distribution based on Boussinesq's theory

4.8 Results and discussions

4.8.1 Indirect tensile strength

All the full depth reclamation emulsified asphalt material with 50 percent RAP and 50 percent virgin aggregate (MG20) specimens for ITS test was tested at five different curing periods as shown in table 4.3. Figure 4.7 shows plot of curing protocols against ITS for dry and wet conditioned FDR emulsion mixtures, respectively. Results show that, ITS did not increase during the initial or younger stage of curing period, but dramatically increased during a later stage of curing and reached optimum values of ITS, generally when the moisture content of the materials reduced. Additionally, the ITS-dry increased significantly with time, at the same time the ITS-wet shows the unnoticeable changes, particularly in the last two curing protocols.

There are no universally accepted standard TSR values (Niazi & Jalili, 2009). TSR is used to predict the moisture susceptibility of the mixtures. According to previous investigations a TSR of 0.8 or above has typically been utilized as a minimum acceptable value for HMA. Mixtures with TSR less than 0.8 are moisture susceptible and mixtures with ratios greater than 0.8 are relatively resistant to moisture damage (Niazi & Jalili, 2009).

As shown in Figure 4.8, the first three curing protocols were initially cured for one day at 25°C to allow water to evaporate. The TSR testing protocol does not permit this initial curing time, so the last two curing protocols were developed for comparison to the TSR results.

4.9 Complex modulus

Complex modulus testing was carried out for six horizontal stress conditions and two testing frequencies (Tables 4.4 and Table 4.5). The influence of confinement pressure on dynamic modulus is shown in Figure 4.9.

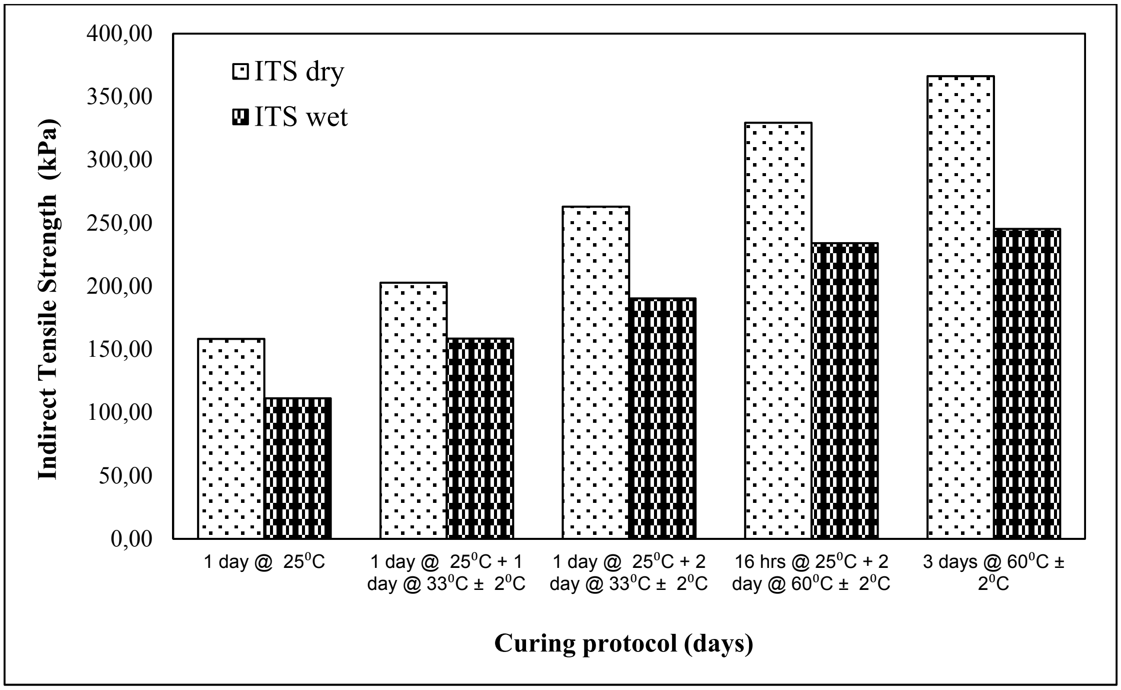

Figure 4-7 Influence of Curing Protocol on Indirect Tensile Strength (ITS)

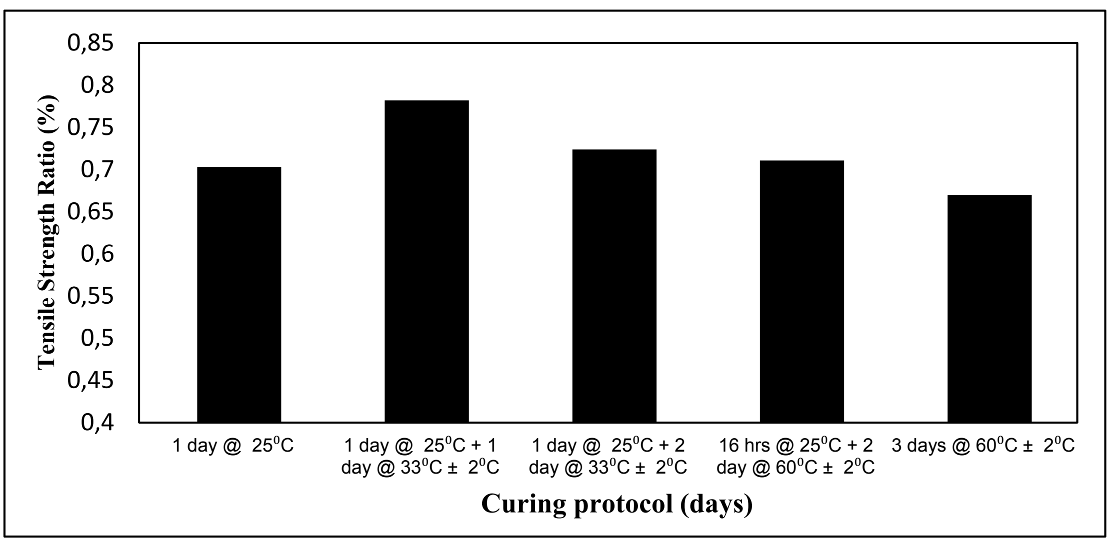

Figure 4-8 Tensile Strength Ratio values of the five curing periods

Initially, the complex modulus was evaluated considering 0 kPa confinement pressure and yielded dynamic modulus of 397.6 and 708.4 MPa at 0.03 and 1 Hz, respectively. In contrast, at the excessively high confining pressure of 115 kPa, the dynamic modulus values were 802.6 and 1229.2 MPa, respectively at 0.03 and 1 Hz testing frequencies. For all confining pressures evaluated, the FDR mixes showed increasing modulus with increasing confining pressure. The highest confining pressure (115 kPa) was selected because it was the maximum pressure that could be maintained at a constant level in the setup.

Also, as observed with HMA samples, frequency had an impact on the stiffness of the studied samples. In fact, dynamic modulus values tested at 0.03 Hz were lower than those tested at 1 Hz for each of the six horizontal stress conditions. Finally, observations on the curves plotted on Figure 4.9 indicated a distinctive rate of increase of the dynamic modulus before and after 50 kPa of confining pressure. The results show a low rate of increase until the confining pressure exceeds 50 kPa, after which the dynamic modulus increases significantly (around 40 percent). This may be due to the fact that confinement pressure below 50 kPa does not provide enhanced interlocking among the aggregates in the structure, but simply reduces the level of voids.

Table 4.4 Complex Modulus (E*) Results for Testing Frequency of 0.03 Hz

Confinement Pressure (kPa)	**|E*| MPa**	**φ Degree**	**E_1 MPa**	**E_2 MPa**
0	398	19.5	375	133
20	426	18.2	404	133
50	482	21.0	450	173
75	596	17.7	568	181
100	717	16.8	686	207
115	804	15.2	775	211

Table 4.5 Complex Modulus (E*) Results for Testing Frequency of 1 Hz

Confinement Pressure (kPa)	**|E*| MPa**	**φ Degree**	**E_1 MPa**	**E_2 MPa**
0	708	19.4	668	235
20	720	19.4	679	239
50	742	18.1	705	231
75	916	16.7	877	263
100	1067	15.3	1029	281
115	1229	13.8	1194	293

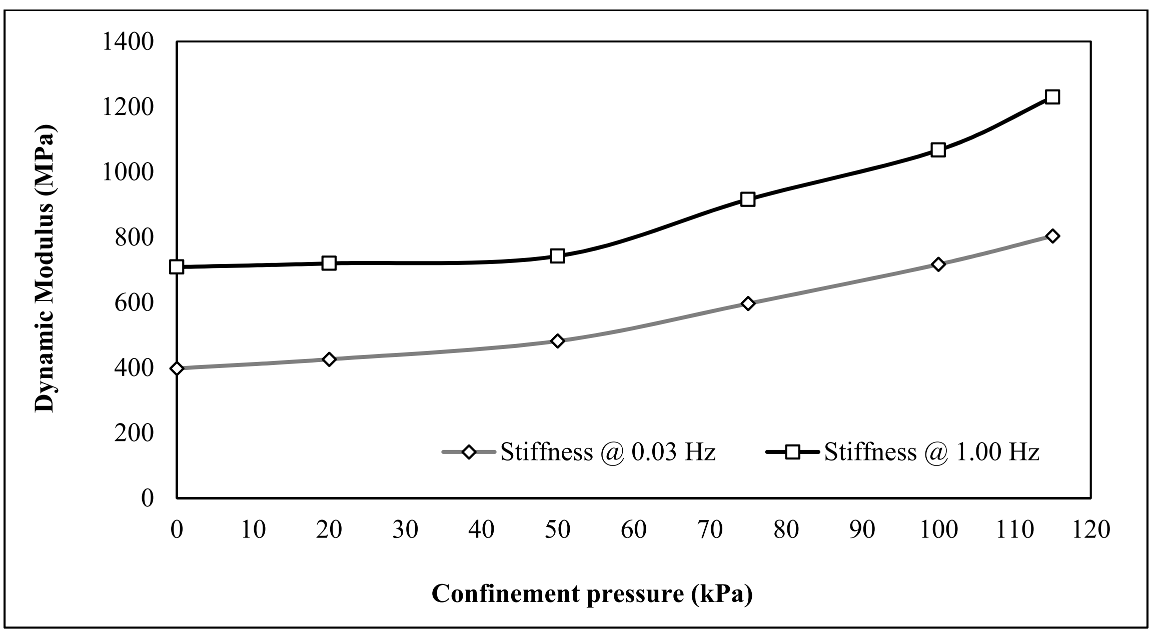

Figure 4-9 Influence of Confinement on the Dynamic Modulus at 23°C ± 1°C

With respect to better understanding the influence of confining pressure on the viscoelastic behaviour of the FDR material, the impact of confinement on the elastic component (E_1) and the viscous component (E_2) of the complex modulus was analysed separately.

Figure 4.10 indicates that confinement has a greater impact on the elastic component than on the viscous component. Therefore, better interlocking among the aggregates structure provided by confining pressure has more influence on the elastic behaviour of FDR mixtures. On the other hand, for complex modulus, Figure 4.10 represents that with an increase in confinement, the phase angle decreases except for 50 kPa at 0.03 Hz, which may have be due to a poor signal input. The observed trend, with respect to the phase angle, highlights that confinement mobilises to a greater extent the elastic behaviour of the sample.

Finally, based upon laboratory results from the LCMB and *Chaussées* 2, the maximum dynamic modulus at 115 kPa for FDR@115 kPa confinement pressure was compared with other dynamic modulus of typical mixtures used in pavement design by Québec Ministry of Transportation (Figure 4.11). The typical asphalt based mixtures are made of a mix of virgin aggregates and PG 64-28. They consist of ESG-10, ESG-14 and GB-20, which are surface layer, surface or base layer, and base layer respectively in pavement structures. The results shown in Figure 4.11 are very different to what is actually used in *Chaussée* 2 for pavement design in Quebec.

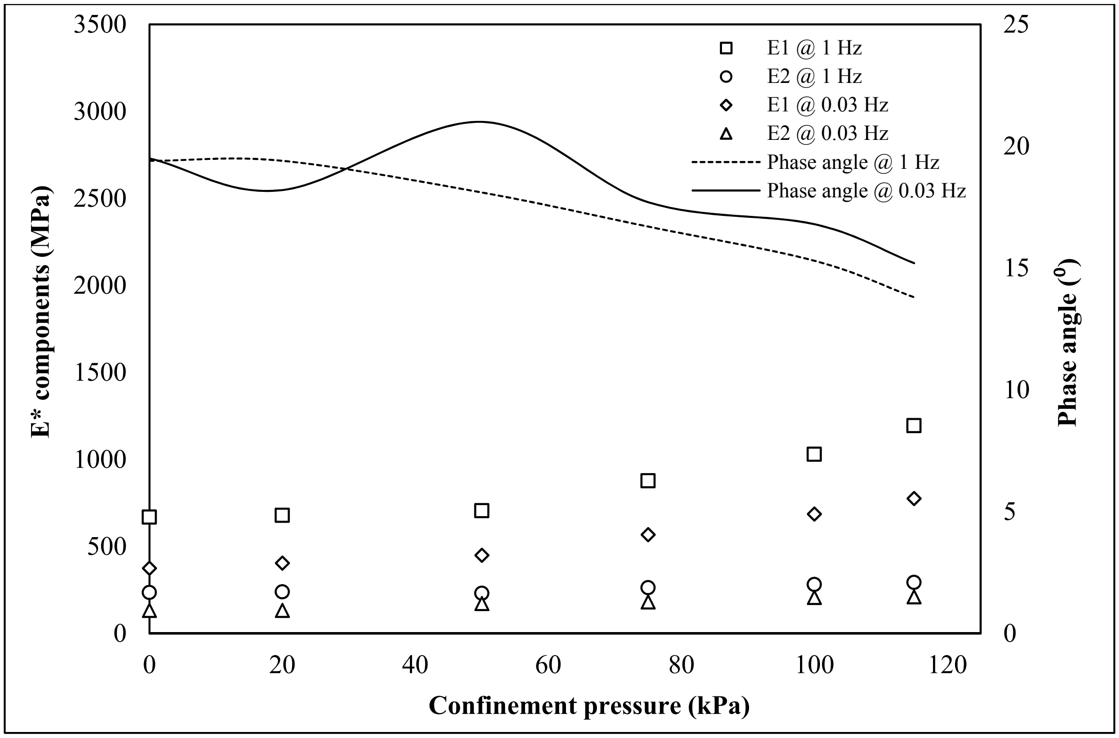

Figure 4-10 Influence of confinement on the E* components and the phase angle

In Table 4.6, you can see a pavement designed with *Chaussée* 2 that can withstand 30,000,000 Equivalent Single Axle Loads (ESALs). If instead of the default value for FDR modulus, using the value obtained from this study, the pavement can now survive 44 million ESALs if we consider the 0.03 Hz value, and 81 million ESALs with the 1 Hz value. This shows that it is particularly important to correctly estimate the modulus of any materials used in a pavement.

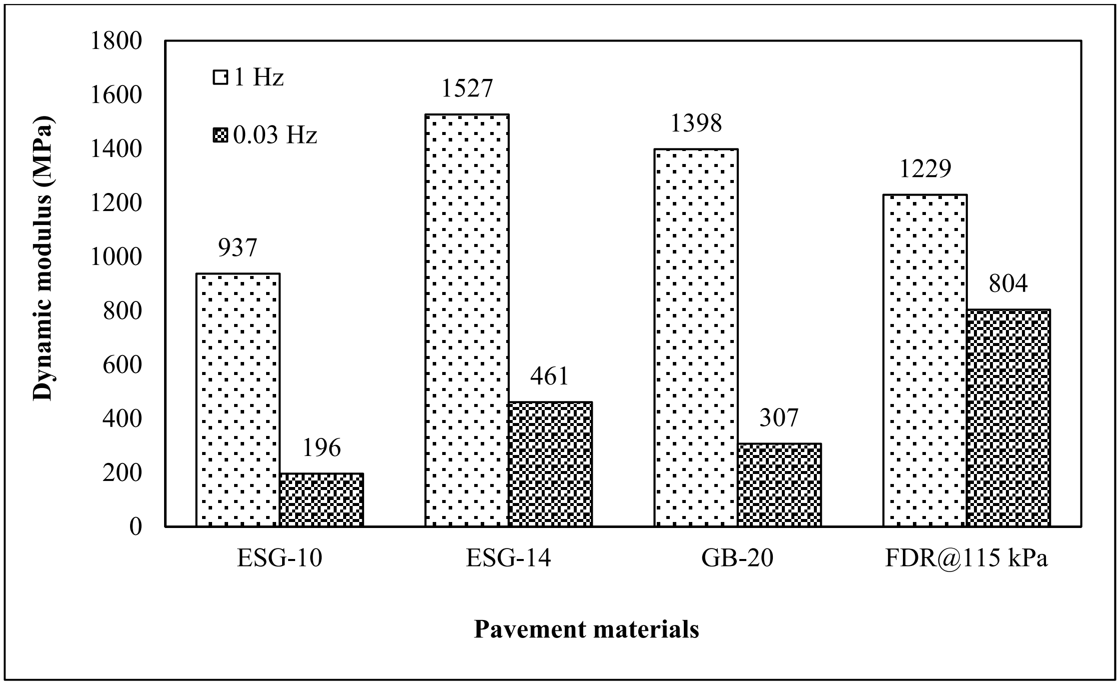

Figure 4-11 Comparison of dynamic modulus of pavement materials at 23°C ± 1°C

Table 4.6 Pavement Design with Full Depth Reclamation with default values for modulus

Material	Thickness (mm)	Modulus (MPa)
HMA	155	3101
FDR + Emulsion + 1,5% cement	250	635
FDR (not treated)	150	84
MG 20	150	110
MG 112	950	74
CH (subgrade)	-	20

4.10 Conclusions

This paper presents a comprehensive study on use of confining pressure when measuring the complex modulus and impact of curing period on ITS of FDR materials.

Based upon the test results, the following conclusions can be made:

- the performance of the FDR mixtures (50 percent of RAP and 50 percent of virgin aggregate) was evaluated based on the ITS test for five various curing periods. The curing protocol presented in this paper clearly indicated that curing period influences the ITS values and moisture damage;
- the ITS test results indicated that a longer curing period would produce the highest tensile strength and a significant increase in the resistance to moisture damage;
- however, ITS values are significantly affected by the bitumen emulsion content, which was around 2 percent of emulsified asphalt results in optimum ITS;
- the notable influence of confining pressure on the complex modulus of the FDR mixtures was mainly on the elastic component.

More work is needed to develop criteria for different confining pressures, frequencies with various temperatures, and different pavement conditions. With more results, it will be possible to use the true value of complex modulus for FDR materials in pavement design, which should result in more designs that are accurate.

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CHAPTER 5

STUDY OF THE IMPACT OF THE COMPACTION AND CURING TEMPERATURE ON THE BEHAVIOR OF COLD BITUMINOUS RECYLED MATERIALS

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5.1 Abstract

In most countries, there is a low temperature limit to lay down hot asphalt mixes because if it's too cold, it becomes impossible to get proper compaction. For cold recycled bituminous mixture (CRM), there is little information on the effect of the low temperature on their behavior. The goal of this study is to evaluate, in laboratory, the impact of the compaction and curing temperature on the behavior of CRM. To do so, CRM containing 50% Reclaimed Asphalt Pavement (RAP) and 50% natural aggregates treated with foamed asphalt or bituminous emulsion were mixed and cured at different temperature between 0°C and 23°C for up to 10 days before being tested in indirect tension. The results show that for all mixes, a cure at lower temperature means lower tensile strength, but the decrease is more noticeable for emulsion treated materials than for foamed treated. The analysis of the results also showed that the decrease in mechanical performance remains important even after a second cure at higher temperature.

5.2 Introduction

In most countries, there is a low-temperature limit to lay down hot mix asphalt (HMA) because if it is too cold, it becomes impossible to get proper compaction. All over the world, various studies have been done to understand the compaction behavior of hot mix asphalt materials. However, limited research is available on low-temperature compaction of cold recycled bituminous mixtures (CRM).

Cold recycling of bituminous materials is a sustainable technology to pavement rehabilitation. It is the reuse of different percentage of Reclaimed Asphalt Pavement (RAP) that is mixed with virgin or recycled aggregates, in different layers of pavement. In order to increase the strength of those mixes, different binders can be used. The most common one are bituminous emulsion and foamed asphalt.

The choice between foamed asphalt and bituminous emulsion is, in many regions, based on cost and availability. However, it has been shown that both binders can give similar results (Carter et al., 2013), even if the method in which the binder glue the particles together differs greatly.

The compaction of CRM with bituminous emulsion or foamed asphalt is a very important factor to get good mechanical characteristics. It helps to position the particles of the material and redistribute the binder from separate globules to continuous films (Needham, 1996). The compaction quality has an impact on air voids of the CRM (Kassem, 2008; Lauter, 1998; Pellinen & Witczak, 1998). Not only the quantity, but the level of uniformity of the air voids distribution considerably affects the behavior of the mixture (Xu et al., 2012; Castillo & Caro 2013). However, too much compaction can also be detrimental. Quick and Guthrie (2011) stated that the severity level of compaction impacts strength development in emulsified asphalt mixture. Compaction can contribute to the initial damage of the emulsified asphalt but also worsen the curing period within these mixtures.

5.3 CRM compaction

Compaction can lead water to disperse from the asphalt bitumen and impact the mix curing time and cohesion (Asphalt Institute, 1997). Barbod & Shalaby (2014) studied the emulsified asphalt mixtures in cold regions and concluded that laboratory prepared specimens at low (5°C) temperature resembles the lower dry density compare to the emulsified aggregate at 24°C which is due to the impact of compaction temperature. For foamed asphalt, it has been shown that a compaction temperature between 13°C and 23°C is optimal and mixing below that will lead to poor quality mixes (Bowering & Martin 1976).

5.4 Curing Process of CRM

Generally, the curing procedure of cold recycled asphalt materials has a significant impact on the final behavior of the mix. Due to that, curing has been considered as important parameter in the asphalt industry.

Various definitions for the curing procedure of cold recycled asphalt materials can be found in the literature (Wirtgen 2010; Tebaldi et al. 2014). Jenkins (2000) defines the cure of CRM as the process in which the water is discharged of the specimen. The Asphalt Institute (1997) mentioned that insufficient curing may increase the chance of asphalt stripping along with a reduction of the rate of strength development when a hot mix asphalt (HMA) overlay is constructed. The curing process can be fairly fast in convenient weather conditions, however, it can be significantly impacted with relatively high humidity, lower temperatures, or rainfall occurred after Cold In-place recycling placement (Y. Kim et al., 2011).

The World Road Association (PIARC 2002) mentioned that the residual moisture has to be significantly evaporated prior to the application of the hot mix asphalt overlay. This period should not only depend on the weather conditions resulting cold in-place recycling construction, but also on the level of traffic. It’s during this period that the material cures and forms some internal structure before being covered with a HMA layer as a wearing course (AASHTO Task Force No.38, 1998; ARRA, 2001; Bergeson & Barnes, 1998). Even though different curing protocols have been adopted in most countries, a universally accepted curing procedure is presently not available.

Marais & Tait (1989) recognized that the cold recycled emulsified asphalt mixture properties changed seasonally with considerable variation in the initial six months to two years. Another researcher, Leech (1994), concluded that full curing of cold recycled asphalt mixtures on construction site may happen between two months and 24 months purely depending on the climatic conditions. However, a time period of 14 days is usually identified as an acceptable cure duration (Croteau & Lee (1997); Kandhal and Mallick (1998)).

The lack of consensus in curing method (duration and temperature) can be seen in the different protocols that can be found in the literature. For example, SABITA (1999) used curing of 24 hours at room temperature, 48 hours at 40ºC with optimum moisture content (OMC) and 45 hours at 60ºC. Robroch S. (2002) worked with 24 hours at 40ºC (sealed) and 48 hours at 40ºC (Unsealed). Asphalt Academy (2003) conducted research at 24 hours at ambient temperatures in mould and 3 days at 40ºC with sealed specimens. Lee et al. (2003) used 6 hours @ 60°C to represent hot summer day and 24 hours at 25°C to represent cool summer night. Carter et al. (2007) used 24 hours at 60°C to accelerate curing, and the mix performances after that short period were satisfactory. Finally, Wirtgen (2004) performed curing protocol at 24 hours at ambient temperatures unsealed and 48 hours at 40ºC sealed, and Gandi et al. (2017) studied laboratory prepared specimens that were cured for 10 days at 38°C.

Some curing methodologies include periods in which the specimens are sealed. This is done to represent field conditions. Batista & Antunes (2003) sealed their specimens with a plastic film, except for the surface; in order to let the water evaporates. They mentioned that moisture content progression in the field would be in between the laboratory prepared CRM emulsified asphalt specimens with plastic film and without plastic films. The change in moisture content is greatly influenced by the temperature. Part of the water will evaporate, and the lower the temperature, the slower this process is. Bocci et al. (2011) studied the temperature influence on three curing protocols (28 days at 40°C, 63 days at 20°C and 56 days 5°C) on the indirect tensile stiffness modulus (ITSM) tests development of the mixture as illustrated in Figure 5.1. They concluded that curing at 40°C and 20°C resulted in higher modulus, whereas at lower temperatures, like 5°C, the curing process is slower.

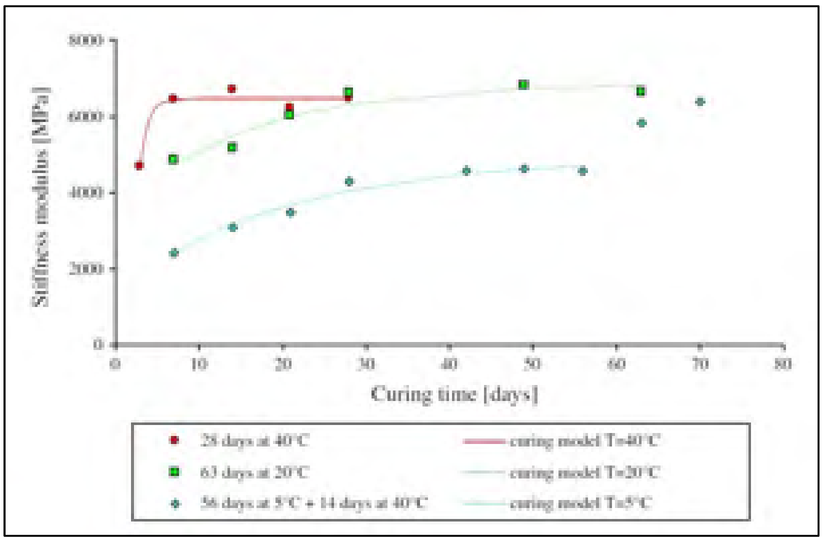


Figure 5-1 Development of Stiffness modulus with curing temperature and time
Taken from Bocci et al. (2011)

5.5 Objective

The main objective of this study is to evaluate the impact of the mixing, compaction and curing temperature on the mechanical properties of CRM. The specific objectives of this study are to evaluate the difference between the evolution of Marshall stability and indirect tensile strength (ITS) according to time and temperature, and also to compare the behavior of foamed asphalt treated CRM (CRM-foam) with bituminous emulsion treated CRM (CRM-emulsion).

5.6 Materials and Methods

In order to reach the objectives, a single mix design of CRM was chosen. A 0-10mm mix of 50% Reclaimed Asphalt Pavement (RAP) with 50% virgin aggregates was prepared in laboratory with the same gradation for each mixes. The mix design used is shown in Table 5.1.

Table 5.1 Mix Design of CRM with Foamed Asphalt or Bituminous Emulsion

	CRM Foam Mixes	**CRM Emulsion Mixes**
Aggregates	50% RAP (4,1% Bitumen)	
	50% Virgin Aggregates	
Cement	1.0%	
Total Water[1]	3.3%	5.0%
Added Residual Bitumen[1]	3.0%	2.0%

[1]% of the weight of the dried aggregates

As it can be seen in Table 5.1, 1.0% of cement was used in all the mixes. Depending on the binder used, foam or emulsion, different amount of water and total added residual bitumen was used. For the emulsion mix, a CSS1 bituminous emulsion containing 62% bitumen was used. The total water shown in Table 5.1 includes the water that comes from the emulsion. The mix design for both binders (CRM-foam and CRM-emulsion) was done according to Quebec’s method LC26-002 which is based on Marshall stability. Basically the amount of water and residual bitumen is a compromised between dry Marshall Stability, retained stability and air voids.

For the foam mixes, before making the mix design, the foamed asphalt design had to be done. Different water content and bitumen temperature (160, 170 and 180°C) were tested, and the optimum foam in regard to the expansion and the half-life was obtained at 160°C with 3.15% water.

The aggregates and the RAP was oven dried and pre-mixed beforehand. Water was added to the mixture 24 hours before mixing to ensure absorption by the solid particles. Then the aggregate-RAP mix was stored for 24 hours at different temperature, as shown in Table 5.2.

Table 5.2 Testing Methodology

	Duration	**Temperatures**
Storage of RAP-Aggregates and mixing accessories	24 hours	Variable (0, 5, 10, and 23°C)
Mixing	< 5 minutes	
Compaction (Marshall 75 blows on each sides)	15-30 minutes	
Demoulding	3 minutes	
Cure	Variable (0-15 min, 1 and 3 Hours, 1, 3, 7 and 14 days)	
Tests (Marshall stability and ITS)	5 minutes	23°C

The chosen temperatures are 0, 5, 10 and 23°C. 23°C was the lab temperature when those experiments were performed, and the other temperatures represent possible field temperatures found in Canada. As shown on Figure 5.2, for more than 50% of the year, the air average air temperatures in Montreal and Vancouver are below 10°C, which is why those temperatures were selected. The relative humidity was not controlled during the cure or the tests. The measured relative humidity in the laboratory during this experiment was between 57% and 63%.

The mixing of the materials was done with a mechanical mixer, and the mixer's bowl as well as the beater was stored at the same temperature than the RAP mixtures. Just like it would be done in the field, the emulsion and the foam were not at those low temperatures. As mentioned before, the foam was produced at 160°C, and the emulsion was stored at 40°C as recommended by the supplier.

Once mixed, the specimens were compacted with a Marshall hammer with 75 blows on each side. The same energy was used for all the specimens, which can results in different air voids, and like it was done for mixing, the Marshall mould were stored at the same temperature as the RAP-Aggregates mixture.

For the specimen to be tested during the first 15 minutes, the demoulding was done right after the compaction. For all the other specimens, the demoulding was done after 1 hour. The demoulding right after compaction resulted in the breakage of many specimens that did not have enough cohesion.

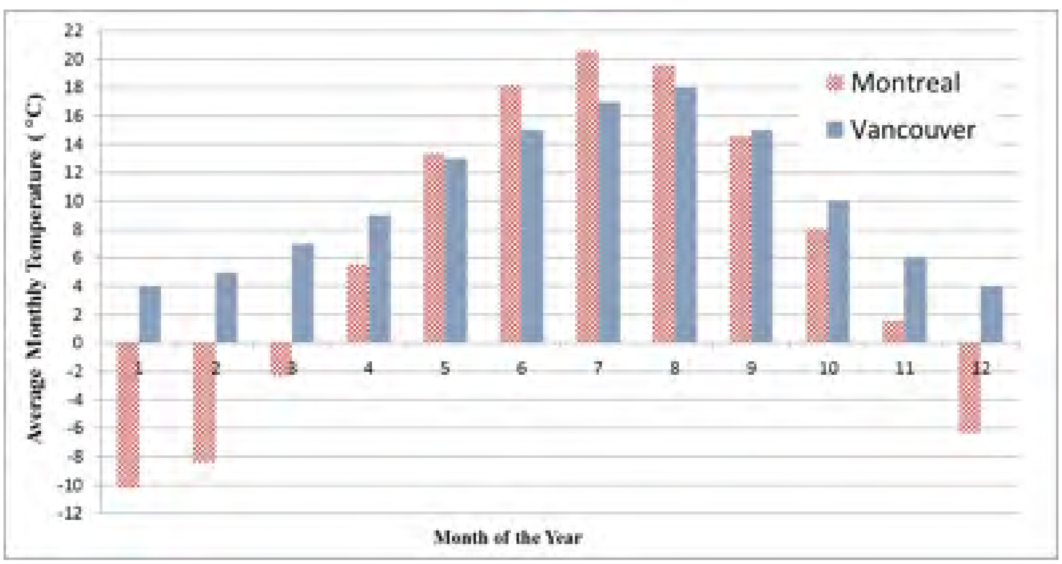


Figure 5-2 Average Monthly air Temperature in Montreal and Vancouver
Taken from WeatherNetwork (2017)

Once the different cure completed, the specimens were left 1 hour at room temperature to be tested in Marshall Stability or in indirect tensile strength (ITS), with the exception of the 0-

15 minutes specimens which were tested right away. It's important to note that during compaction, the temperature of the specimen increases. Therefore, for the 1 and 3 hours cure, the specimen did not have time to stabilize at the cure temperature before being tested.

Marshall Stability was used for this research project because it's the specified method for CRM characterization according to Quebec's standard. Since ITS is used by many agencies and research center to evaluate CRM mechanical behavior, it was decided to use this method also.

Both tests were performed at the same loading rate (51 mm/min) and on the same apparatus (Figure 5.3), but with a different loading setup. For ITS, a small loading strip curved for 100 mm samples was used at top and bottom, and for the Marshall stability, the usual Marshall breaking head was used.Those two tests give mechanical properties of CRM. The results of the tests can be associated with the cohesion of the materials, but not with their durability.

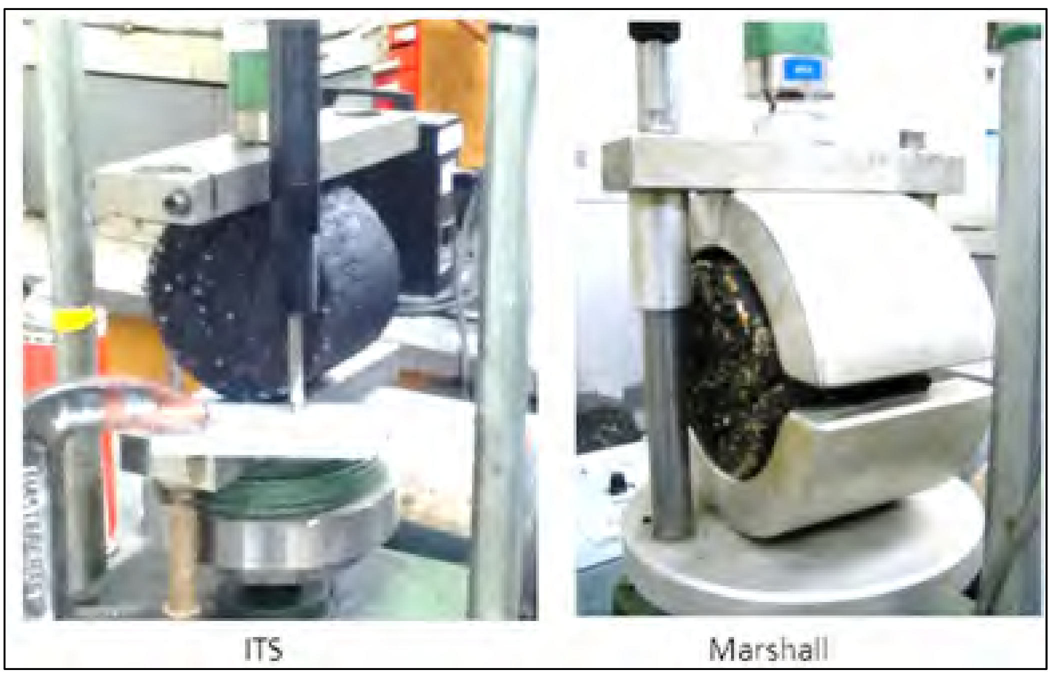


Figure 5-3 Indirect Tensile Strength (ITS) and Marshall Stability Test Apparatus

5.7 Results

For every mixes, a minimum of three specimens were prepared for ITS tests and three more for dry Marshall Stability. However, many specimens were broken during demoulding or handling, which resulted in having two specimens for most of the tests. The results shown are the averages of those two or three when a third specimen was available. If the difference between two mechanical results were above 10%, new specimens were mixed, cured and tested. According to Fu et al. (2010), higher variability in the results are expected at cure temperature below 15 to 25°C.

The Marshall dry stability results for different cure duration and conditioning temperature for foamed asphalt and bituminous emulsion treated CRM are presented in Figure 5.4. As it can be seen, the results at 23°C, for both types of CRM mixes (CRM-foam and CRM-emulsion), are similar. This shows, as mentioned in the literature, that both binders give similar properties in ideal conditions. However, it can be seen that the low mixing and cure temperature has a greater impact on CRM-emulsion mixes than on CRM-foam mixes. For example, at 14 days, the Marshall stability of the CRM-emulsion mix cured at 0°C is about 20% of the Marshall stability of the CRM-foam mix, which is about the same difference that we got between a cure at 0°C and 23°C for the emulsion mix. In addition, it should be noted that there are no Marshall stability in the first three hours for the emulsion mixes at 0°C. This is because the cohesion was too low to take the specimens out of the mould; all those specimens broke during demoulding.

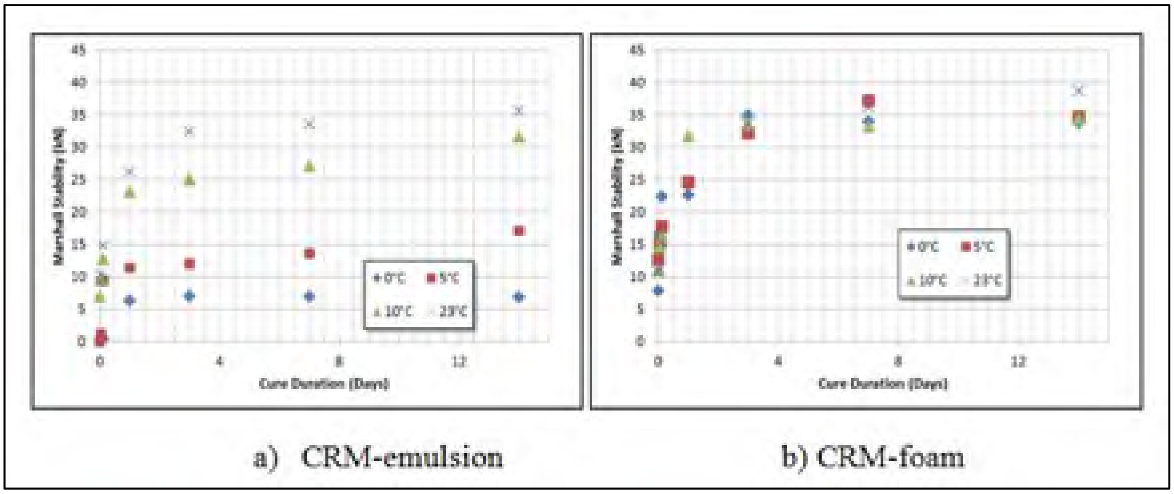


Figure 5-4 Average results of dry Marshall Stability after different curing time and temperature for all CRM mixes

Another aspect that is interesting is the rate of increase of the Marshall stability, which can be related to the rate of increase of the cohesion. The temperature has a very limited effect on the rate of cohesion increase for CRM-foam mixes, but it has a major impact for the CRM-emulsion mixes. At 23°C, the rate is about the same for both types of CRM mixes, but at lower temperature, the cohesion increases at a much slower rate with emulsion.

The results of the ITS is shown on Figure 5.5. On the left side (Figure 5.5a), we can see that, just like the results for the Marshall stability, there is a big difference between the results at low temperature than the results at room temperature for the emulsion mixes.

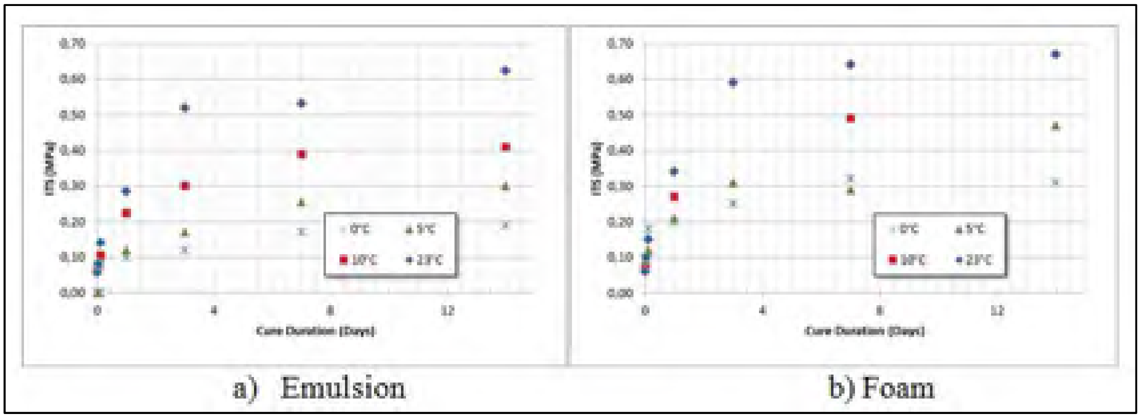
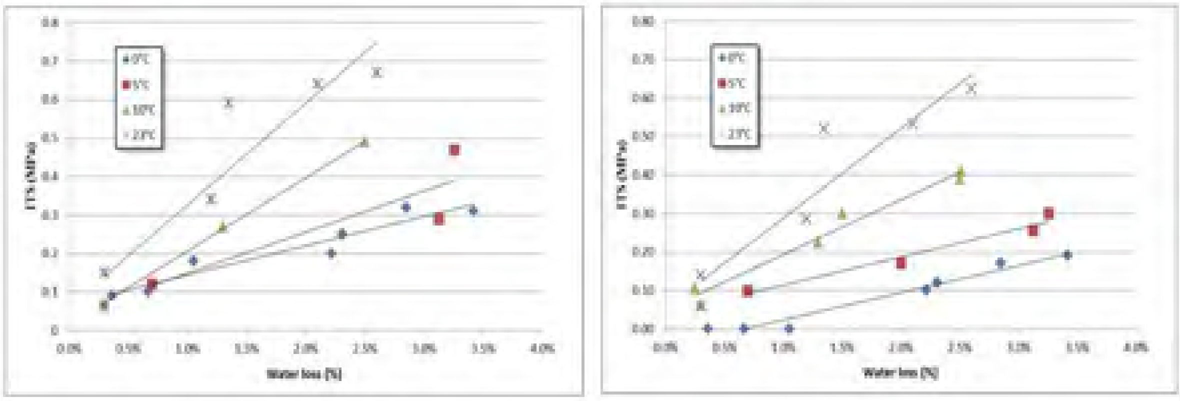


Figure 5-5 Average dry ITS results of after different curing time and temperature for all CRM mixes (a. Emulsion mixes, and b. foam mixes)

However, in the case of ITS results for the foam mixes (Figure 5.5b), the trend in the results is different. For the Marshall stability, the temperature seems to have little influence on the results. For ITS, a clear difference is seen according to temperature. For example, at 14 days, there is barely over 10% of difference between the Marshall stability at 0°C and 23°C, but there's a 50% difference for ITS. More tests are needed, but it seems that ITS is more sensitive to the difference in the behavior of those materials.

Contrary to the Marshall stability, the rate of increase in the cohesion, which is related to ITS this time, is affected by the temperature for both CRM-foam mixes and CRM-emulsion mixes. However, the effect is greater for CRM-emulsion mixes.

On Figure 5.6, we can see the relation between ITS and water loss. This presentation of the results shows that the cohesion increase follows a linear trend at every temperature tested, but that this rate diminishes with the temperature. The rate of the cohesion increase is represented by the slope of the best fit curves (all $R^2 > 0{,}8$).



(a) CRM-Emulsion (b) CRM-Foam

Figure 5-6 Average dry ITS results of after different curing time and temperature for all CRM mixes Vs water loss (a. Emulsion mixes, and b. foam mixes)

Another aspect of bituminous mixes, CRM or hot mix asphalt, that has a big impact on the mechanical performances, is the air voids. By mixing and compacting the specimens at different temperature but with the same energy, it was suspected that the air voids may be different according to the temperature. The air voids, calculated from the dry bulk density and the maximum density are shown in Figure 5.7.

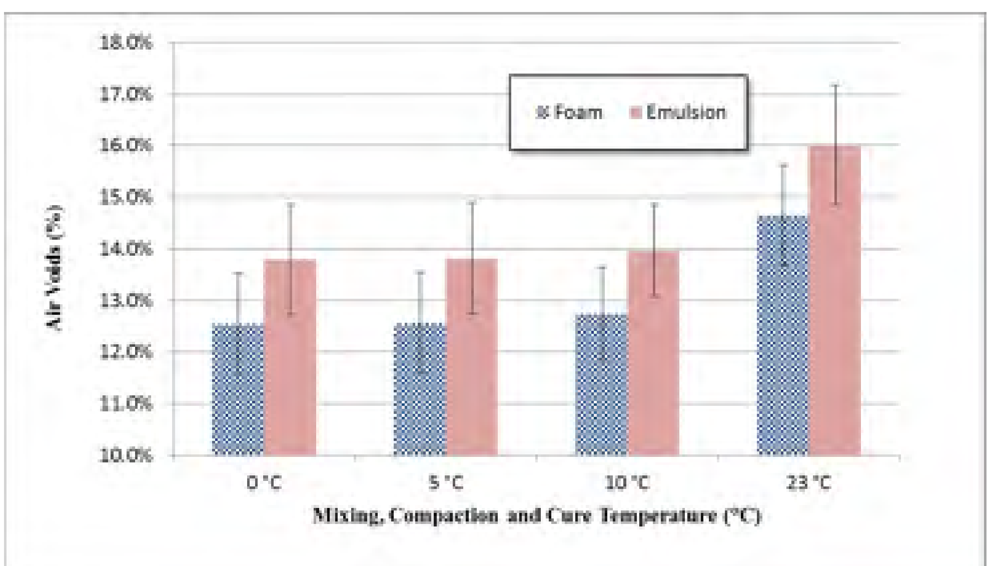


Figure 5-7 Air Voids according to the mixing and compaction temperature

According to Quebec's specifications, the maximum acceptable air voids content in CRM is 18%. Even if the results respect the limit, the variation of the air voids is high for a given temperature. The error bars on the figures represent one standard deviation. However, at least two trends are visible here. First, globally, the air voids of the emulsion mixes are higher than for the foam mixes. This is expected because of the higher water content in those mixes. During compaction, water serves as a lubricant, but it also creates voids. Lower water content can results in higher cohesion and mechanical properties, but it can also results in an incomplete coating, which can in turn results in poor retained stability, or high moisture sensitivity. Second, the air voids at low temperatures are lower than at room temperature. This can be explained by the black rock effect. At 23°C, the bitumen of the RAP does not seem sticky, but with the pressure applied during compaction, it restrains movement of the particles since it increases the friction in the mix. At lower temperature, the same bitumen is stiffer, less sticky, so it does not increase the friction. It could be hypothesized that in fact at 0°C for instance, the bitumen on the RAP particles makes them somewhat rounded, which results in easier compaction.

Lower air voids should mean higher ITS value and higher Marshall stability. This can lead us to think that if the specimens were compacted at equivalent air voids and not with equivalent energy, the ITS and the Marshall stability results would be even lower for the tested temperature below 23°C. The fact that the measured mechanical performances are lower at lower temperature must be due to the repartition of the bitumen in the mix. For foamed asphalt, when the hot bitumen droplets come into contact with the cold aggregates, the temperature of the bitumen reduces very rapidly, which increases it's viscosity, and reduces its capacity to adhere properly to the fines. So instead of having homogeneous mastic around the aggregates, we can postulate that there are clusters of bitumen separated by fines particles, which creates weaker plane in the mix.

For the bituminous emulsion, the reduction in temperature has a direct impact on the coalescence of the bitumen droplets. In fact, at temperatures below 8 to 10°C, the flocculation of the bitumen droplets happens really fast when the emulsion comes in contact with the cold aggregates, but the coalescence is limited, and even impossible in some cases (Audeon, 1993). This could explain why very little cohesion, if any, was measured at short time for low temperature. Low temperature destabilizes the emulsion, just like a quick change in pH does. James (2006) mentions that solvent can be added to emulsion to accelerate coalescence at low temperature. This means that the manufacturer could design an emulsion that breaks normally at low temperature, but that emulsion would break too rapidly at normal temperature.

5.7.1 Effect of an increase in the curing temperature

The obtained results clearly shows that a reduction in mixing and curing temperature has a negative impact on the mechanical properties measured with ITS and Marshall stability for CRM-foam and CRM-emulsion mixes. The results do not, however, show if this decrease in performance is permanent or not. In order to study that aspects, the new specimens were prepared with the same exact methodology, but with an added curing protocol after the initial 14 days. Once the 14 days at the various curing temperature was reached, the specimens were left for another 14 days at 23°C before being tested in ITS. Results are shown on Figure 8. On Figure 8, the ITS results obtained at 28 days is divided by the ITS results obtained at 14 days 23°C in order to better evaluate the change during this new curing period.

As it can be seen, for CRM-foam (Figure 5.8b), with an additional 14 days at 23°C, the ITS value increase significantly, but they still do not reach the values obtained after only 14 days at 23°C. However, a longer curing period at that temperature should results in higher ITS value.

For the CRM-emulsion at 0°C, the additional curing period did not have a significant effect (Figure 5.8a). In this case, it seems that the cohesion measured through ITS at 14 days is the maximum that this mix will reach.

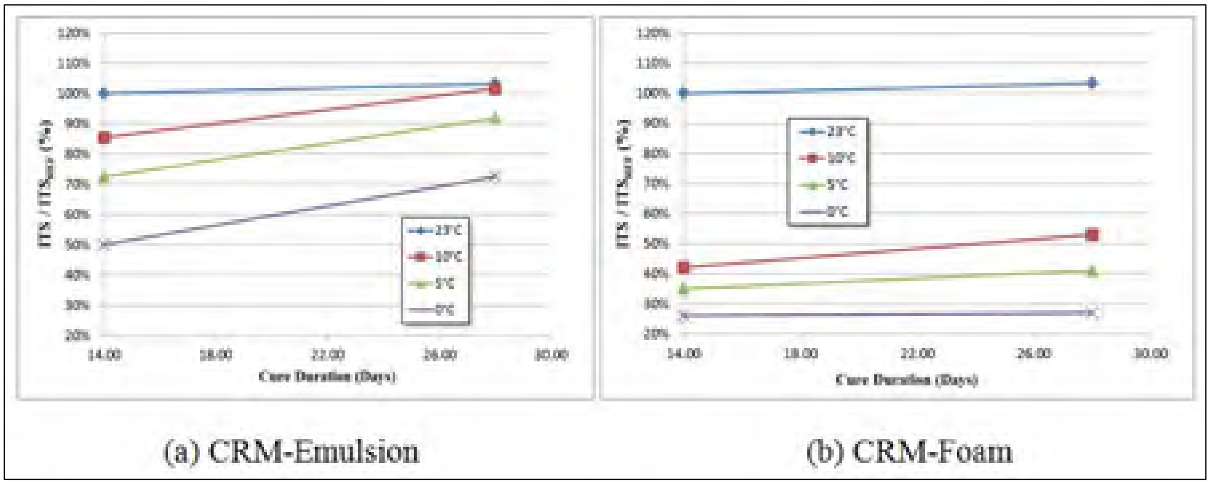


Figure 5-8 Average dry ITS results of after different curing time and temperature for all CRM mixes with an additional 14 days at 23°C cure (a. Emulsion mixes, and b. foam mixes)

5.8 Conclusion

The objective of this research was to evaluate the effect of cold temperature mixing and curing of cold recycled materials treated with foamed asphalt and bituminous emulsion. The CRM tested were made of 50% RAP and 50% virgin aggregates and they were mixed and cure at 0, 5, 10 and 23°C for up to 28 days before being tested in Marshall stability and ITS. Globally, it has been shown that low temperature for mixing and curing of CRM mixes do have a major impact on their mechanical performances.

More specifically, the main conclusions that are drawn from this study:

- Marshall and ITS results are related, but ITS results are less variable and more sensitive to mixing, compaction and curing temperature;
- CRM-foam and CRM-emulsion have similar Marshall stability and ITS when made and cure at room temperature;
- CRM-foam is less sensitive to low temperature cure than CRM-emulsion. This can be explained in part because of the lower water content of CRM-foam mixes compared with CRM-emulsion mixes, and also because of the curing mechanisms of both mixes type;
- mixing and compaction at 0, 5 and 10°C enables better compaction, so lower air voids,

than at 23°C. This shows that at low temperature, RAP has more a black rock behaviour than at room temperature;

- An additional curing period at 23°C does have a significant impact for low CRM-emulsion specimens that sustained initial low temperature cure. This additional cure does however help with CRM-foam mixes.

In order to better understand the results obtained here, new tests should be done. The measurement of the complex modulus on the CRM at different mixing, compaction and curing temperatures would help to better grasp the impact of the lower temperature. Also, it would be helpful to test at longer curing period, and to test the moisture sensitivity.

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CHAPTER 6

EFFECT OF BINDER TYPE ON FULL DEPTH RECLAMATION MATERIAL BEHAVIOUR

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6.1 Abstract

The present study aims to evaluate the potential use of both foamed asphalt and emulsified asphalt on Full Depth Reclamation (FDR) mixtures to have better performance with four different types of FDR mixtures. Different combinations of adding procedures were tested to find the optimum mix design procedure. The scope of work for this research consisted of determining the optimal mixing procedure according to moisture sensitivity tests, complex modulus (E*) at different loading frequencies and temperatures. It was concluded that the better performance can be achieved with double coating practices. In particular, mixing procedure shown that the first coating the coarse aggregate with foamed asphalt and second coating the fine aggregate with emulsified asphalt ensures the best results in terms of the performance based tests. The complex modulus showed that using both binders it was possible to produce a mixture with higher modulus than mixtures characterized by a single coating.

6.2 Introduction

Full Depth Reclamation (FDR) is a popular rehabilitation technique for flexible pavements, in which the old asphalt pavement and predetermined portion of granular base are recycled at the same time to lay down a new single layer (ARRA, 2001; Carter et al., 2010; Gandi et al., 2015). The FDR is a cost effective and environmentally sustainable approach for construction of pavements compared to conventional Hot Mix Asphalt (HMA) (ARRA, 2001; Epps & Allen, 1990). FDR can be done with two different techniques, which are FDR with emulsified asphalt (EA) and FDR with foamed asphalt (FA). Over the years, both the technologies are fully consolidated in practice and witness of numerous studies (Y. Kim & Lee, 2006), and developments. In the framework of this study, preliminary tests were carried out on combined usage of FDR-EA and FDR-FA techniques. It is believed that with emulsified asphalt, most particles are well coated, which is not the case with foamed asphalt. However, foamed asphalt does work as a binding agent in Cold recycled asphalt materials. As of now, there have been no precise mix design specifications to understand the double coating (combined) technology. Therefore, this can achieved through using the proper approach to develop the mix design and validate the probability of using EA and FA together on FDR mixtures to have superior mechanical characteristics.

Double coating is an innovative technology that consists in splitting the production process into two parts (coarse and fine aggregates) in order to obtain the optimal combination in term of aggregates coating and rupture time. The present study was done in two steps. The first step of the study focus on the determination of the laboratory optimum mix design procedure by varying each of the components involved: emulsified asphalt, foamed asphalt, an emulsion/emulsion double coating mixture, emulsion/foam double coating mixture and aggregates gradation curve (Gandi, Bensalem, Bressi, Carter, & Bueche, 2016). Moreover, in the second step, this is a part of the validation effort to assess the consistency of the developed optimum mix design procedure. The Complex modulus tests were conducted on FDR mixtures (50% of Reclaimed Asphalt Pavement (RAP) and 50% of Virgin Aggregates) of four different combinations of the binders like FDR emulsified asphalt mixture (Mix-A), FDR foamed asphalt mixture (Mix-B), FDR emulsified asphalt – emulsified asphalt double coating mixture (Mix-C), and FDR emulsified asphalt - foamed asphalt double coating mixture (Mix-D). The results obtained from the complex modulus test were analyzed to characterize the double-coated FDR materials.

6.3 Background

6.3.1 Bitumen stabilized Cold recycled asphalt mixtures

All over the world, bitumen stabilized material(s) (BSM) as emulsified asphalt and foamed asphalt mixtures usage is eventually increase in road construction and rehabilitation. However, it has created a need for sound guidelines to be established for the laboratory mix design procedures for FDR materials. Typical normally contains water (25% to 60%), bitumen (40% to 75%), and emulsifier (0,1% to 2,5%), depending on the specific type of emulsified asphalt and the necessary viscosity (James, 2006). The role of EA in cold in-place recycling method delivers a robust binding through the recycled asphalt pavement material. In the last two decades, various researchers have been studied on the emulsified asphalt technologies for road rehabilitation and construction, in addition to substantial developments were succeeded on that (James, 2006). For instance the performance of the final mix is increases by adding the polymers to the emulsion (Chavez-Valencia et al., 2007).

Foamed asphalt is a process in which water is injecting in to the expansion chamber containing hot bitumen at 170°C to 180°C, resulting in spontaneous foaming, produces the foamed or expanded asphalt (Muthen, 1998). From the early 1960s, Bowering (1970) (Bowering, 1970); Bowering (1976) (Bowering & Martin, 1976); Acott (1979) (Acott, 1979a); Lee (1981) (Lee, 1981); Ruckel et al. (1983) (Ruckel et al., 1983) have been studied the foamed asphalt mixtures using virgin materials. Subsequently, foamed asphalt has begun to be implementing in the FDR process of aged asphalt pavements. Further, Wood (1982) (v W. & Wood, 1982); AI (1983) (The Asphalt Institute, 1983); and Hicks (1988) (Hicks, 1988) have been researched the design procedure and the performance of FDR foamed asphalt. Maccarrone et al. (1994) (Maccarrone, S., Holleran G., Leonard, D. J. and Hey, 1994) was introduced a FDR-foamed asphalt process called FOAMSTAB with benefits such as a rapid curing, better fatigue performance and cost effective. In 2002, in full depth reclamation, the foamed asphalt has been used as a stabilizing agent for Route 8 in Belgrade (Marquis et al., 2003). However, there have been very few projects were constructed with foamed asphalt as a stabilizing agent.

According to literature, FDR-EA and FDR-FA techniques signify an effective solution for old asphalt pavements; however, detailed comparison between the two gives in depth advantages and disadvantages of each technique and helps in understanding one of objective of the present study. Primarily, it is very important to understand that these two technologies have the different form of distribution of binder. Essentially, the EA acts as a lubricant in the process of the compaction stage and, prior to it start breaking, the coarse aggregates and fine aggregates are totally being covered by the binder (K. J. Jenkins, Robroch, Henderson, Wilkinson, & Molenaar, 2004). On the other hand, the fine aggregates mainly covered by the foamed asphalt mixture. The breaking clearly visible in emulsified asphalt is uniformly distributed in the FDR mixtures. On the other hand, irregular black spots appeared in foamed asphalt mixtures.

As mentioned by recent studies (Y. Kim et al., 2011), Given the same compaction effort, cold recycled emulsified asphalt specimens showed lesser density than cold recycled foamed asphalt specimens. Both Indirect tensile strength and Marshall Stability of cold recycled emulsified asphalt specimens were about same as those of cold recycled foamed asphalt specimens (Kim & Lee, 2010). With respect to curing, EA mixtures have lower dynamic modulus than the FA mixtures; it could be due to the inferior moisture content. In addition, ITS results are affecting by the RAP percentage and type of bitumen grade (He & Wong, 2008).

6.3.2 State of the art on Full Depth Reclamation material mechanical properties

Until now, numerous studies concluded that, at early periods, the FDR materials behaviour is seems to be like granular material, nevertheless, when the curing is done the FDR materials behaviour is close to HMA. As a result, it is considered that FDR-EA and FDR-FA materials have a time-dependent behaviour (Pérez et al., 2013). Locander (2009), explained that granular and FDR materials have a distinctive behaviour due to the presence of the binder, and coating FDR's aggregates. Molenaar (K. J. Jenkins, Van de Ven, de Groot, & Molenaar, 2002) concluded that, in comparison to an equivalent granular material, the inclusion of binder (foamed asphalt) in cold recycled mixes resulted in greater cohesion. Jenkins stated that, Foamed bitumen mixtures with 2% binder content perform similarly to granular materials. Whereas, with less than 4% binder content foamed bitumen mixtures shows stress dependent behaviour(K. J. Jenkins & Van de Ven, 2001). Santagata, Chiappinelli, Riviera, and Baglieri (Santagata et al., 2010b) reported that when properly designed CRM, in the long-term, can achieve stiffness values comparable to those obtained for an HMA mixture. Therefore, Pérez (Pérez et al., 2013) explained that treating FDR materials, which are stabilised with a binder, as a granular material is unrealistic. Moreover, there is a persistent gap between the predicted life as a result of pavement design simulation and the observation in the field with respect to FDR layers in flexible pavement structures.

Cizkova and Suda (2017) studied the mechanical behaviour of cold recycled asphalt mixtures with foamed asphalt and emulsified asphalt. They concluded that the CRMs are sensitive to thermo-mechanical behaviour. However, these materials are less dependent on temperature and frequency than traditional HMA mixtures. Particularly, at lower temperatures and higher frequencies, these materials shows elastic behaviour. Carter, Bueche, and Perraton (2013) investigated complex modulus of cold recycled asphalt materials treated with EA and FA. Based on laboratory test results they concluded that, for full depth reclaimed asphalt materials, at higher temperature and lower frequency foamed asphalt treated mixtures are higher modulus values then emulsified asphalt treated mixtures. Godenzoni, Graziani, and Bocci (Godenzoni et al., 2015) studied the cold recycled emulsified asphalt materials with different percentages of RAP (0%, 50% and 80%) contents. They concluded based on the complex modulus test results reveled that the cold recycled emulsified asphalt materials with RAP showed as asphalt-like behaviour than without RAP mixtures. Godenzoni, Graziani, and Perraton (Carlotta Godenzoni et al., 2016) studied the Linear Viscoelastic region (LVE) response of cold recycled asphalt mixtures treated with foamed asphalt. They revealed based on results that the values of the phase angle and stiffness modulus are lower than the traditional hot mix asphalt mixtures. Gandi et al., (2017b) investigated the complex modulus of cold recycled materials treated with emulsified with different RAP contents. Figure 6.1 represents the cole - cole diagram of the 2S2P1D model of the respected mixtures. They concluded that at lower frequency and higher temperature of 100 percentage of RAP shows high stiffness values.

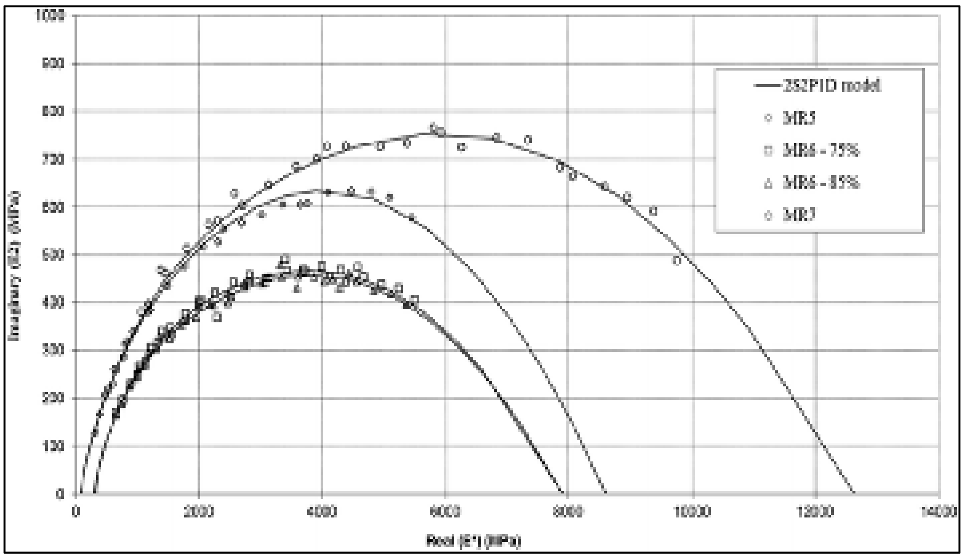

Figure 6-1 Complex modulus of Cole–Cole diagram with 2S2P1D model
Taken from Gandi et al. (2016)

Despite the recent efforts employed for the investigation of the FDR mechanical behavior, few studies have been conducted on combined usage of FDR-EA and FDR-FA techniques (Double Coating). The present study was undertaken to provide additional information on the rheological properties of FDR materials using both emulsified asphalt and foamed asphalt. That emulsified asphalt allows an appropriate coating of the aggregates while foamed asphalt does not reach the same efficiency as a binding agent, thus a mix of both techniques could result in higher level quality mixes (Gandi, et al., 2016; MTQ, 2001).

6.4 Objectives

The objectives of the present study were to:

a) Determine the mix design procedure for double coating full depth reclamation materials with the addition of four different combinations of the binders.

b) To evaluate the complex modulus of the double coated full depth reclamation materials with the addition of four different combinations of the binders.

6.5 Experimental Plan

6.5.1 Materials

In this study, the Full Depth Reclamation (FDR) samples like 50 percent of Reclaimed Asphalt Pavement (RAP) and 50 percent of Virgin Aggregate (MG20) were fabricated in the laboratory with VA, RAP, Emulsified Asphalt (EA), Foamed Asphalt (FA), water and Portland cement. The RAP used in this research was acquired from a stockpile in the Montreal city. The RAP was homogenized to confirm that all representative samples have likely similar gradation. The VA was the nominal maximum aggregate size (NMAS) of 20 mm (MG20), which is aggregate usually used in Quebec as a base material for highway construction. The FDR asphalt mix gradation is according to TG 2 (TG 2, 2009) as shown in Table 1. Intended for the mixes with emulsified asphalt, two different types of binders (CSS-1S and CSS-1P) were employed as mentioned as shown in Table 6.1. Foamed Asphalt was produced in the laboratory based foaming plant as showed in Figure 6.2. The Foamed asphalt was produced by combing small amount of water with hot bitumen under air varying pressure (Table 6.1). The FDR mix gradation and other properties of the mixtures used in the experiments are presented in (Table 6.1).

6.5.2 Mix design

The compaction ability of the double coating mixture emulsion/foam was studied performing four series of tests, which are directly related to four types of mixtures: FDR emulsified asphalt mixture (Mix-A), FDR foamed asphalt mixture (Mix-B), FDR emulsified asphalt - emulsified asphalt double coating mixture (Mix-C), and FDR emulsified asphalt - foamed asphalt double coating mixture (Mix-D). The latter has been compared to the first three reference mixes with Mix-D. Furthermore, the addition of coarse aggregates and fine aggregates to the Foamed Asphalt and/or Emulsified Asphalt in the mix design has been distributed in two parts. To test the Indirect Tensile Strength (ITS) and Marshall Stability 10 replicates were compacted for each mix.

Table 6.1 Full Depth Reclamation mixture gradation and its properties

	Percent of Passing sieve	
Sieve Size (mm)	**Requirements**	**FDR mix gradation**
28	80-100	100
20	-	95
14	50-90	89
10	52-75	74
5	25-55	48.5
2.5	-	29.1
1.25	-	23.9
0.630	-	12.2
0.315	5-20	6.4
0.16	-	3.7
0.080	3-10	2.3
% of residual binder in RAP (According to ASTM D6307-10(ASTM D6307-10, 2010))		6.38
AC of Emulsified Asphalt CSS-1S (%)		65.2
AC of Emulsified Asphalt CSS-1P (%)		61.6
Compaction		Marshall and Superpave gyratory Compaction
Curing Time (days)		10 days at 38 ± 2°C
PCC (%)		1.0
Water content (%)		6.5
Targeted Air Voids V_a (%)		13 ± 1
Foamed Asphalt Production		
Bitumen Grade		PG 58-28
Water Content (%)		3.25
Expansion ratio		15
Half-life		12 seconds at 170°C temperature.

Note: V_a= Air voids of the mixture; AC=Asphalt Content; PCC=Portland Cement Content; CSS-1S = Cationic Slow-Setting with soft bitumen emulsion; CSS1P= Cationic Slow Setting 1 with Polymer; PG= Penetration Grade.

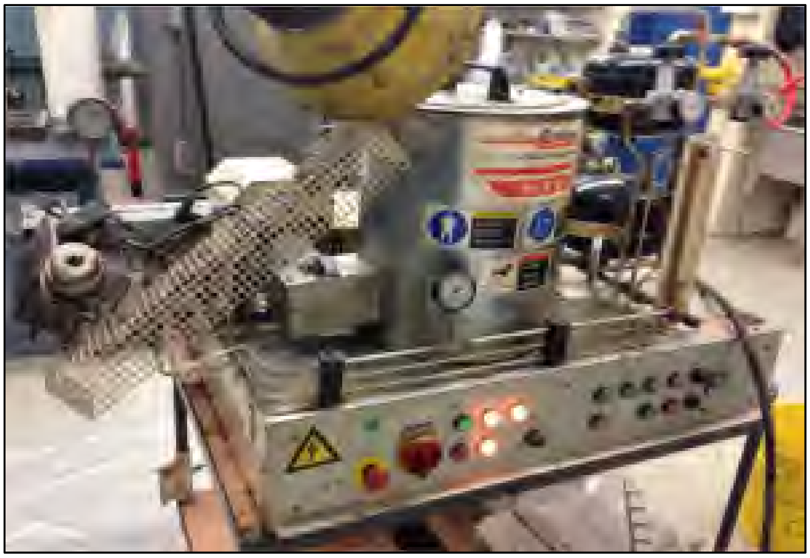

Figure 6-2 Writgen laboratory foaming Plant

6.5.2.1 Single coating and double coating of emulsified asphalt mixtures

Single coating of emulsified asphalt mix design has been done with respect to Quebec standard LC 26-002 (MTQ, 2001). The pre-mix optimum water content was fixed at 6.5% by weight on the dry aggregates, including cement. The rate of pre-mix water can lead to several advantages, such as higher RAP content, virgin aggregate coating, increased lubrication during compaction, and accelerating the cement hydration process (Yan et al., 2010). The exact dosages of water and cement are added to aggregates and thoroughly mixed for one minute. Then, CSS-1S emulsified asphalt is poured according to proportions, and the mix is blended for one more minute.

The same process is applied to double coating emulsified asphalt mixes, but the entire mixing process is split in to two phases. Initially, the 0/5 aggregate fraction is mixed with Portland cement and water and for one minute. Then, half portion of the first emulsified asphalt (CSS-1S) is added and mixed again for one minute. Before performing the second coating, the emulsified asphalt needs to break first. After that, the second aggregate fraction (5/20) and second emulsified asphalt (CSS-1P) is poured and mixed well for one minute duration.

6.5.2.2 Double coating of emulsified and foamed asphalt mixtures

Double coating of emulsified asphalt and foamed asphalt mixtures mix design is follows the similar process as mentioned in section 6.5.2.1, even though it has two separate stages in the mixing procedure. The initial aggregate fraction is mixed with 50% of water content and the necessary amount of emulsified asphalt (CSS-1P) for one minute. Then, immediately after the emulsified asphalt breaks, the mixture and the second fraction of aggregate are added directly into the laboratory foam mixer. Afterwards, the remaining 50% of water content and 1% of cement are poured, whereas foamed asphalt was added according to mix design proportions.

6.5.3 Sample Preparation

To determine the mix design of FDR materials in this study, a Marshall compactor was used to produce specimens at targeted percentage of air voids, applying 50 blows on each face. The following curing process was performed at one day at ambient temperature with mould and one day at 38± 2°C in demolded state. In particular, curing humidity was not controlled, even though laboratory relative humidity resulted being always around 50%. The air voids of all FDR mixtures were measured and presented in Table 6.2.

Table 6.2 Percentage of air voids of FDR Mixtures

S.No.	Mix Type	Percentage of Air voids (%)
1	Mix A	10,62
2	Mix B	11,36
3	Mix C	11,64
4	Mix D	12,24

In addition to that, to evaluate the rheological characteristics of the double coated full depth reclamation materials with the complex modulus test, cylindrical specimens were produced by means of a gyratory compactor fixing the targeted air voids content. Specimens were immediately demolded after compaction and cured for 10 days at 38 ± 2°C. At the end of the curing process, samples of 75 mm x 120 mm prepared with the help of coring and sawing.

6.6 Testing

6.6.1 Marshall Stability Test

Marshall Stability and flow test results along with density and other parameters are normally utilized to compare and evaluate the laboratory mix designs of asphalt mixtures. In addition, it evaluates the properties of conditioning such as with water (ASTM D6927-15, 2015). For Marshall Stability and flow, the cured specimens are tested with laboratory Marshall testing equipment at room temperature, and it reaches failure under a constant load (Figure 6.3). The maximum load linked to failure is named Marshall Stability, which needs to be corrected according to the sample height.

Figure 6-3 Compacted Marshall Specimens and Marshall Testing setup

6.6.2 Indirect Tensile Strength Test

The Indirect Tensile Strength (ITS) test values can be used to evaluate the moisture damage and quality of asphalt mixtures (ASTM D6931-17, 2017) (Figure 6.4). The ITS test was performed at room temperature. The following equation 6.1 is used to obtained the ITS value, which is calculated dividing the maximum compressive strength by the specimen's geometrical properties (ASTM D6931-17, 2017):

$$St = \frac{2000 \times P}{\pi \times t \times D} \tag{6.1}$$

Where,

S_t : Indirect Tensile Strength, kPa;

P : Maximum load, N;

D : Specimen diameter, mm, and

t : Specimen height immediately before test, mm.

Figure 6-4 Indirect Tensile Strength Test loading

6.6.3 Complex Modulus Test

The structural performance of flexible pavement is significantly influenced by the rheological properties of the asphalt mix layers. Therefore, the rheological analysis aims at defining the constitutive laws of road materials in order to associate their specific mechanical properties with the performance of road materials in exercise and the expectations of the pavement service life.

During the pavement design phase of road structure, it is necessary to consider that the temperatures are normally between 0°C and 60°C while the loads from vehicular traffic have a short time application but not sufficiently short to induce purely elastic behavior in the bituminous material. FDR materials, containing bitumen, should be studied referring to models and principles used in the rheological analysis of the viscoelastic material. This means that the application of a constant effort (σ) produces both an instantaneous deformation and a deferred deformation that grows during the entire period of the load application, i.e. elastic and viscous contributions coexist. An identical behavior is observed when the load is removed. The elastic deformation returns instantly, followed by a delayed recovery delayed while a rate of irreversible deformation due to viscous flows represent a plastic deformation. In the Linear Viscoelstic region (LVE) only the first two components are taken into account. Therefore, the complex modulus for asphalt concrete is defined by the following Equation 6.2:

$$|E^*| = \sqrt{E'^2 + E''^2} \tag{6.2}$$

Where:
E''= loss modulus, viscous contribution [Pa]
E'= storage modulus, elastic contribution [Pa]

The phase angle (δ) represents the distributions of the elastic and viscous contributions (Equation 6.3):

$$\delta = arctg\left(\frac{E''}{E'}\right) \tag{6.3}$$

The rheological properties become important input parameter to implement mechanistic-empirical pavement design models (Witczak & Fonseca, 1996). The complex modulus and phase angle depend on the mixture characteristics, the loading frequencies, and pavement temperature profile. Clearly, a lack or fragmentary information regarding the rheological behavior of FDR has become a source of reluctance to use this type of alternative as pavement base materials (Depatie, Bilodeau, & Gold, 2012). As of today, attempts to characterize the stiffness of FDR materials through a tri-axial test, by measuring the resilient modulus (MR), or through a complex modulus (E*) test have been undertaken.

The experimental results obtained from the complex modulus test are analysed through the 2S2P1D (2S: two Springs, 2P: two Parabolic elements, 1D: one Dashpot) model and graphical representation of the model is in Figure 6.5 (Hervé Di Benedetto et al., 2004).

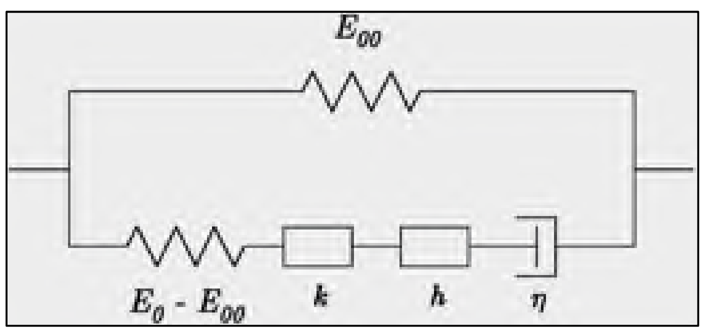

Figure 6-5 The graphical representation of the 2S2P1D model
Taken from Di Benedetto et al. (2004)

It is extensively used to model the LVE unidimensional or tridimensional behavior of bituminous materials which includes binders, mastics and mixes (Olard & Di Benedetto, 2003). The 2S2P1D analytical expression of the Complex Young's Modulus, at a specific temperature, as expressed by Equation (6.4):

$$E^*(i\omega\varpi\tau) = E_0 + \frac{E_\infty - E_0}{1 + \delta(i\varpi\omega\tau)^{-k} + (i\varpi\omega\tau)^{-h} + (i\varpi\omega\beta\tau)^{-1}} \qquad (6.4)$$

The temperature (τ) change is dependent by means of the shift factor at temperature (T) as presented in the equation (6.5):

$$\tau_E(T) = a_T(T) \times \tau_{0E} \qquad (6.5)$$

Where aTref (T) is the shift factor at temperature T and $\tau_E = \tau_{0E}$ at reference temperature Tref. Seven constants (E00, E0, δ, k, h, β and τ_{0E}) are required to completely characterise the linear viscoelastic properties of the tested material at a given temperature. The evolutions of τ_E were approximated by the William-Landel-Ferry (WLF) model (Ferry, 1980) (Equation 6.6). τ_{0E} was determined at the chosen reference temperature Tref. When the temperature effect is considered, the number of constants becomes nine, including the two WLF constants (C1 and C2 calculated at the reference temperature).

$$\log(a_T) = \frac{-C_1\left(T - T_{ref}\right)}{C_2 + T - T_{ref}} \qquad (6.6)$$

All the experimental and analytical results fit on a single curve in the Cole-Cole plan of the model, if the material has linear viscoelastic behaviour. Furthermore, for reference temperature, with considerations to the principle of time and temperature equivalency, master curves are deducted from the test results and highlight the evolution of the dynamic modulus with regard to a constant reference temperature and a changing frequency.

Coefficient of evolution C^*_{CE} is introduced, in order to compare objectively the experimental results of complex modulus of mixtures with different binders. The calculation of the RAP coefficient of evolution (C^*_{RCE}) was proposed by Di Benedetto (Delaporte, Di Benedetto, Chaverot, & Gauthier, 2007). It is defined as the ratio between the complex modulus of a specific mix at the equivalent frequency (f_e) and complex modulus of a reference mixture at the same frequency (f_e) as mentioned in equation (6.7).

$$C^*{}_{RCE}(f_e) = \frac{E^*{}_{mix}}{E^*{}_{ref-mix}} = |C^*{}_{RCE}|e^{i\Phi RCE} \tag{6.7}$$

$C^*{}_{RCE}$ is a complex number, as shown in equation (6.7). It is standard is the ratio of the norms of the complex modulus of the recycled mixture to the one of the reference as calculated by equation (6.8). Its phase angle is the difference between the phase angle of the recycled mixture and the one of the reference as determined by equation (6.9).

$$|C^*{}_{RCE}| = \left|\frac{E^*{}_{mix}}{E^*{}_{ref-mix}}\right| \tag{6.8}$$

$$\Phi_{REC} = \Phi_{E^*{}_{mix}} - \Phi_{E^*{}_{ref-mix}} \tag{6.9}$$

It is important to note that the $|C^*{}_{RCE}|$ value is calculated in the reference mixture. The complex modulus was measured with a servo-hydraulic testing system (MTS 810). The axial strain was measured on the center portion of the testing specimen with the help of three 50 mm extensometers, placed 120° apart as shown in figure 6.6. Each sample was subjected to haversine compression loading (stress controlled) along the axial direction. Experiments were performed under strain control with target amplitude of 50 µdef. The test was performed at eight temperature (-25^{o}C to 45^{o}C) and five frequencies (0.03 to 3.00 Hz). After each temperature change, 6 hours of conditioning period has been applied.

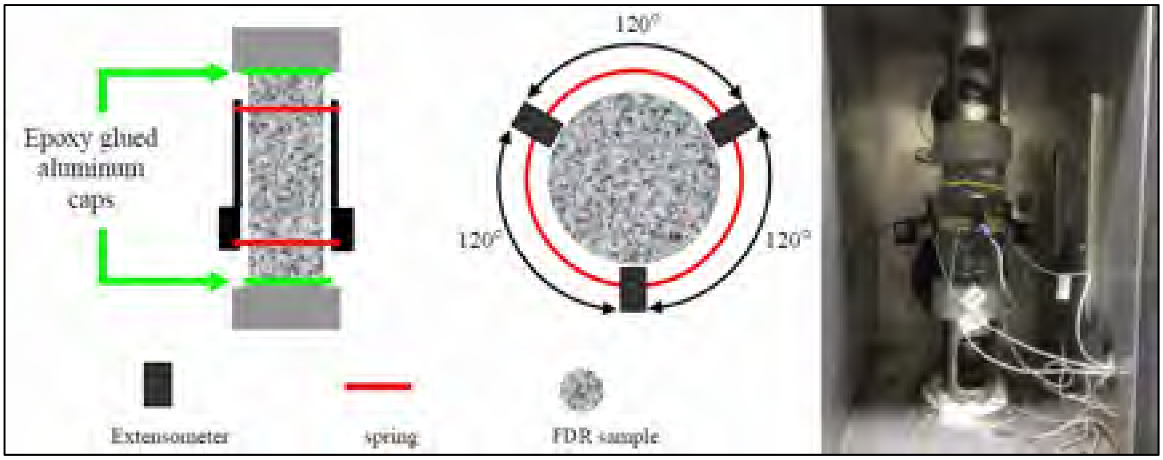

Figure 6-6 Complex Modulus Test setup and MTS Machine
Taken from Gandi et al. (2017b)

6.7 Results and discussion

6.7.1 Marshall Stability

Figure 6.7 is presented the test results of all mixtures starting form Mix-A to Mix-D in dry condition and wet condition. According to Ministry of transportation Quebec, minimum of 8 kN of Marshall Stability required (MTQ, 2001), which is satisfied by all the mixtures. As anticipated, the Marshall Stability of Mix - A and Mix - B are relatively lower than Mix - C and Mix – D. Furthermore, in saturated conditions, all formulations showed almost the same resistance value.

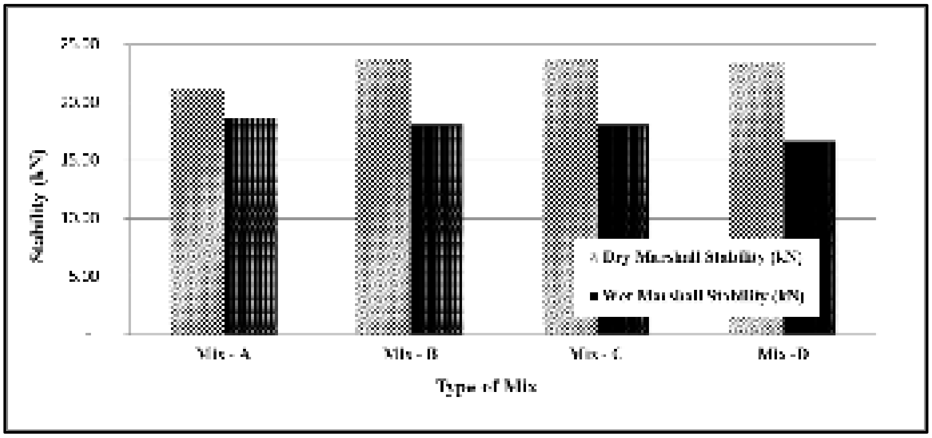

Figure 6-7 Marshall Stability of FDR materials

Overall, Marshall Moisture susceptibility results were not satisfactory enough. Figure 6.8 shows that double coating mixes are the most influenced by the presence of water. At contrary, Mix-A has the lowest stability loss (11.35%), due to the optimal coating action provided by the single film of emulsified asphalt. On the other hand, higher moisture sensitivity was obtained by foamed asphalt mixtures, probably due to the tendency of foamed asphalt to merge mainly with the fine fraction, leading to a lower coating. However, it was expected Mix-D to reach lower moisture sensitivity than Mix-B (single foamed asphalt).

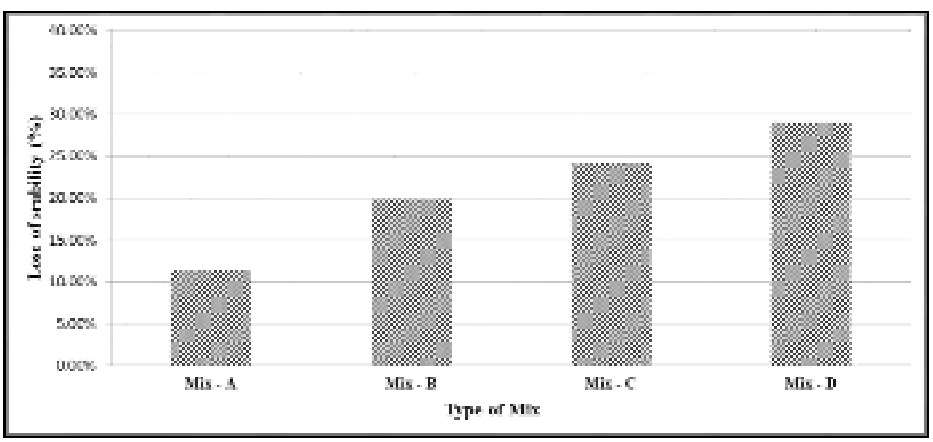

Figure 6-8 Loss of stability of FDR Materials

6.7.2 Indirect Tensile Strength

Figure 6.9 shows both ITS-Dry and ITS-wet experimental results. As for Marshall Stability values, no particular gain in strength is visible among the four formulations, especially between single and double coating. The Mix - B and the Mix - D shows good results. The Mix - B is very similar to the latter.

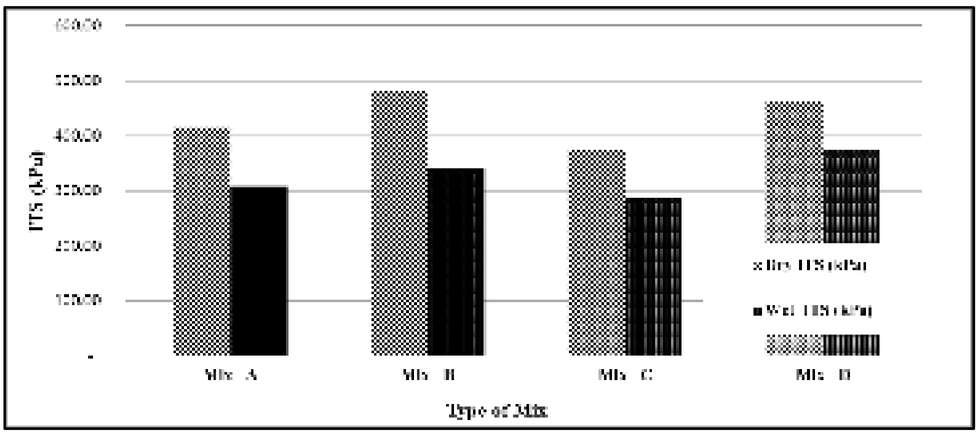

Figure 6-9 Indirect Tensile strength test results of FDR Materials

ITS moisture sensitivity results are more comprehensible, if compared to the Marshall Test ones. As expected, double coating FDR mixtures are the most performant, with Mix-D reaching the highest value (81%) (Figure 6.10). Since moisture susceptibility is considered one the important parameters in this study, it is fundamental that double coating mixtures respect the Wirtgen reference criterion. Such good results for Mix-C and Mix-D indicate a good and suitable coating of aggregates and demonstrate the effectiveness of the formulation used for the double coating. In particular, for detailed mixing procedure refer Gandi et al. (Apparao Gandi, Bensalem, et al., 2016). They concluded that the first coating the coarse aggregate with foamed asphalt and second coating the fine aggregate with emulsified asphalt ensures the best results in terms of the performance based tests. For an approach to optimize the double coating mix design procedure please refer to Appendix II.

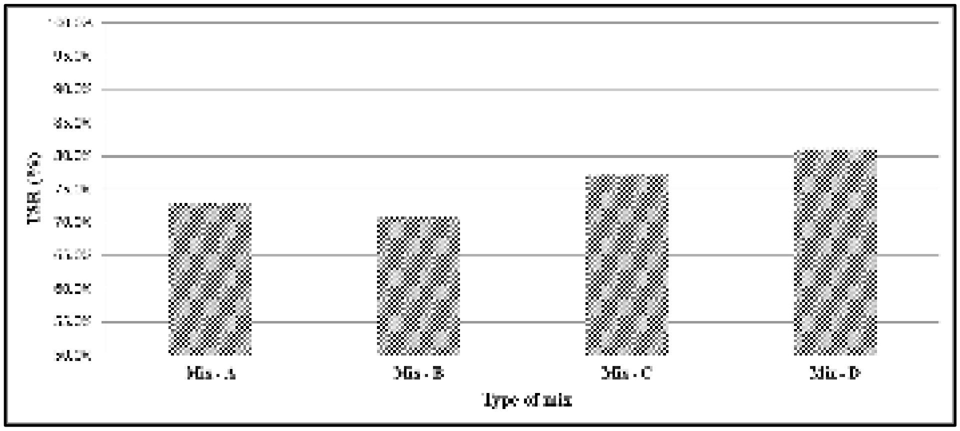

Figure 6-10 Various tensile strength ratios of FDR materials

6.7.3 Complex Modulus

As mentioned before, complex modulus test was performed on samples in a range of eight different temperatures and five different frequencies. The 2S2P1D rheological model was used as a tool to analyze results obtained from the laboratory investigation.

6.7.3.1 The Cole-Cole diagram and Black space diagram with 2S2P1D model

The 2S2P1D model is generally used to explain both behaviors of the asphalt mixtures and binder (Olard & Di Benedetto, 2003). The complex modulus tests were carried out on four different asphalt mixtures (Mix-A through Mix-D) at eight temperatures and five frequencies; this allows to determine accurately the modeling parameters (E_0, E_∞, k, h, β, δ, C1, and C2) to be employed in the 2S2P1D model to characterize the linear viscoelastic response of the asphalt mixture. The modeling parameters are listed in Table 6.3 at a reference temperature. Such parameters are determined by the best-fitting curve for all the measured complex modulus data plotted in the Cole – Cole and Black space diagrams of the 2S2P1D models. Figure 6.11 and Figure 6.12 represent the Cole – Cole and Black space diagrams respectively. The binder rheology is represented by the k, h, δ and β parameters (C Godenzoni et al., 2015). These parameters are nearly same for single and double-coated mixtures separately, which means a double coating of the asphalt mixtures could lead to a change in the binder rheology. For what concerns the other parameters, E_0 is the static modulus (E when $\omega \rightarrow 0$), and E_∞ is the glassy modulus (E when $\omega \rightarrow \infty$), which is normally related to the air void content and aggregate skeleton (Nguyen et al., 2009). However, it should be noted that the targeted percentage of air voids and the aggregate gradation is the same for all mixtures. Our results show that the binder type affected the glassy modulus, which is moderately higher for mixtures treated with foamed asphalt (Mix-B and Mix-D).

Figure 6.12 illustrates the black space diagram of 2S2P1D model, in which the complex modulus norm is linked to the phase angle (φ). As the experimental data suggest, the phase angle, which is the loss coefficient of the material, varies between 3.85° (low temperature/high frequency) and 33.18° (high temperature/low frequency). In general, if the material has high φ values, it is supposed to be highly viscoelastic and to absorb more cyclic loading energy as a consequence; on contrary, with less φ value it absorbs less energy. However, values of both E_0 and φ for all tested cold recycled asphalt mixtures are below those normally measured on HMA (C Godenzoni et al., 2015). Figure 6.12 shows that phase angle is significantly attenuated at higher temperatures with respect to Mix-B. It can be seen that the Mix-B has relatively high φ values than the other mixtures. This can suggest that the Mix-B is more viscoelastic and in addition to this, double coating does have higher impact on elastic response rather than on viscous behaviour.

Table 6.3 Parameters of the 2S2P1D model for the FDR Mixtures (Tref = 5^{0}C)

Mixture	**E0 (MPa)**	**E∞ (MPa)**	**k**	**h**	**δ**	**β**	**C1**	**C2**
Mix - A	80	8750	0.18	0.5	4.8	1000	16.24	108.38
Mix - B	41	9600	0.17	0.5	3.6	375	21.56	150.15
Mix - C	26	3800	0.16	0.5	2.5	1200	20	136.88
Mix - D	75	7500	0.16	0.4	3.0	1200	19.08	137.01

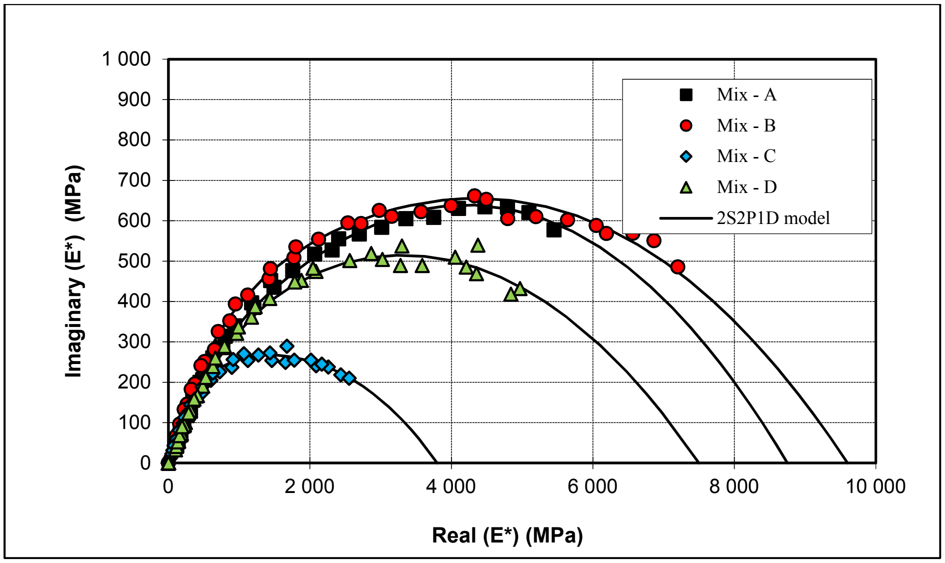

Figure 6-11 Complex modulus master curve in Cole & Cole axes

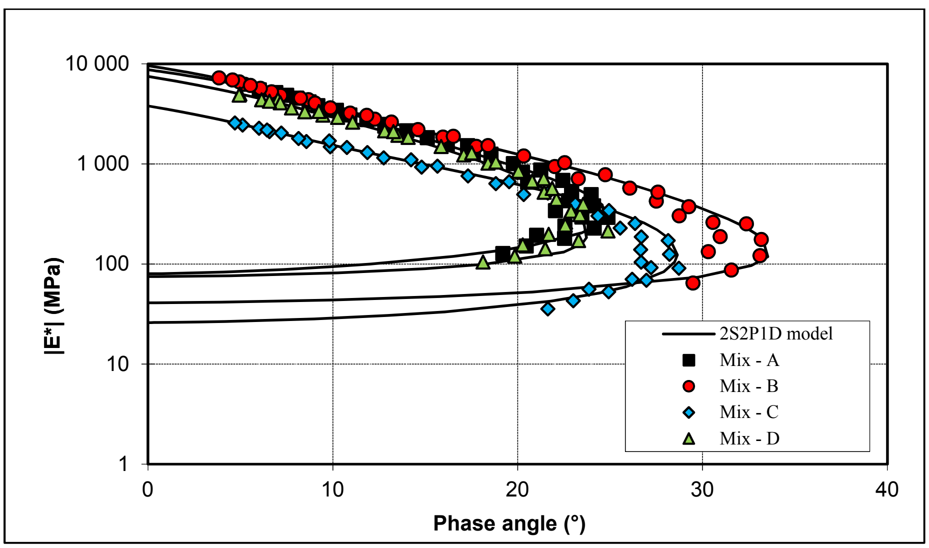

Figure 6-12 Complex modulus master curve in Black space diagram

6.7.3.2 Master curves of the tested FDR asphalt mixtures

An optimal tool to understand the complex modulus test results is to plot them as a master curve. If the hypothesis that the asphalt mixture satisfies the Time-Temperature

Superposition Principle (TTSP) is assumed, the master curve can be plotted as a function of an equivalent frequency. Initially, the reference temperature is selected (T_{ref} = 5^{0}C), and then, all data at different temperatures need to be shifted with respect to time in order to obtain a single smooth master curve. The build-up of the master curve requires the determination of the shift factors for each testing temperature *T,* named $a_T(T)$*,* that can be done by means of Equation 6.6. However, to achieve a complete understanding of the rate and temperature effects, both the master curve and the shift factor $a_T(T)$ are needed (Singh, 2011). Figure 6.13 illustrates the master curves (complex modulus norm as a function of a frequency of the material) of the four mixtures at the reference temperature T_{ref} = 5^{0}C.

In Figure 6.13, the top right portion of the |E*| master curves at a higher frequency or low temperatures approach asymptotically to a maximum value which describes a maximum stiffness value of the corresponding asphalt mixtures (Mix-A and Mix-B). At the bottom left quarter of the graph, which means at lower frequencies or high temperatures, |E*| master curves approach a minimum value which describes the minimum stiffness value of the corresponding asphalt mixture (Mix-C). In addition to this, at the lower frequency and higher temperature, the other two mixtures (Mix-A and Mix-D) represent the maximum stiffness value. In particular, FDR asphalt mixtures treated with foamed asphalt are representing high stiffness values at lower frequencies which are characterized by an improved cohesion with respect to unbound granular materials. It should be noted that Mix-C showed relatively lower stiffness at both lower frequency - higher temperature and high frequency - low temperature. A hypothesis could be the less cohesion presents in between double coated emulsified asphalt materials (Mix-C).

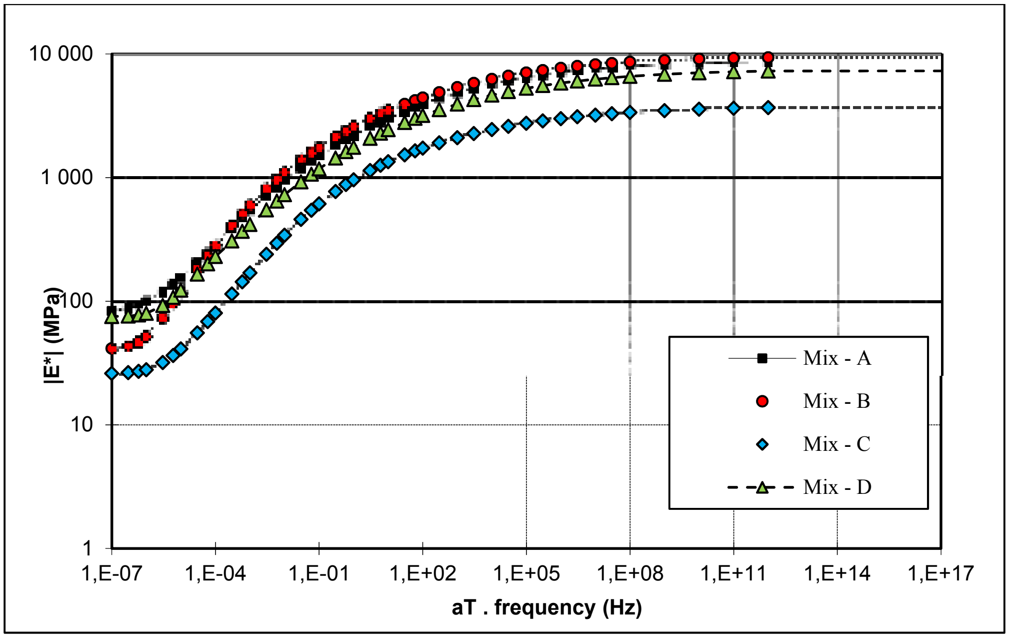

Figure 6-13 Master curves of the norm of complex modulus

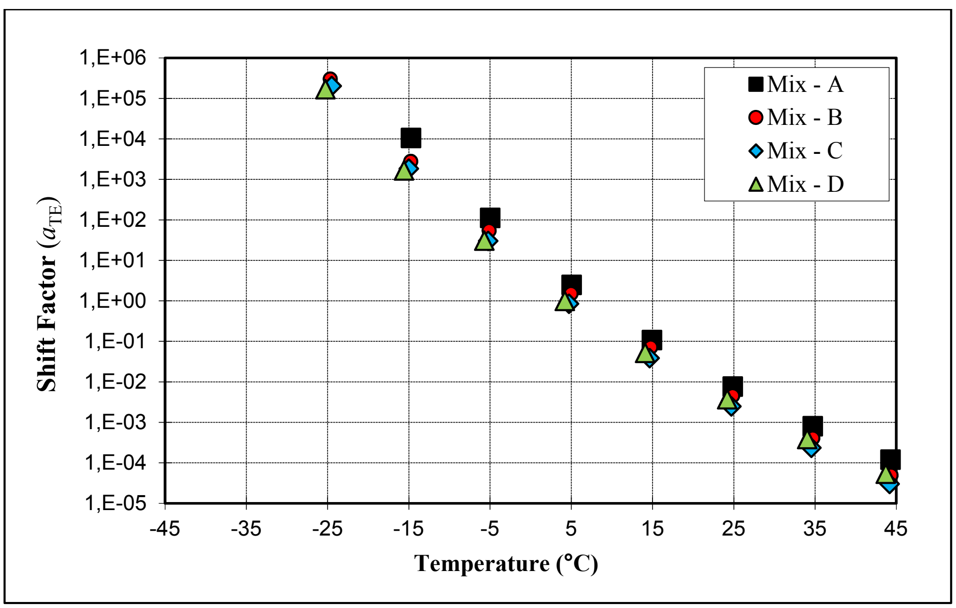

Figure 6-14 Master Curves and Shift factors of
complex modulus norm at (Tref = 5^0C)

Figure 6.14 represents the shift factors for the norm of the complex modulus at 5^{0}C. Both the master curve and the shift factor $a_T(T)$ are needed for a complete depiction of the rate and temperature effects (Singh, 2011). From Figure 6.14, Mix-A has higher thermal susceptibility in the entire temperature domain. In addition to this, the double-coated mixtures (Mix-C and Mix-D) have lower thermal susceptibility. In other words, double-coated asphalt mixtures could be less sensitive to temperature. This aspect needs further research and other laboratory tests.

6.8 Conclusions

The present study was carried out to determine the mix design procedure for double coating FDR materials and evaluate its rheological characteristics of four different combinations of the binders like FDR EA mixture (Mix-A), FDR FA mixture (Mix-B), FDR EA-EA double coating mixture (Mix-C), and FDR EA-FA double coating mixture (Mix-D).

The following conclusions can be drawn based on the results.

- Prior to mechanical testing, two-stage mixing procedure was done to produce specimens with uniform binder percent and same volumetric properties. This new two-stage mixing technique is primarily based on pre-coating a portion of the aggregate with the suitable quantity of optimum binder content. Based on the ITS and Marshall stability test the Mix-D results indicated better performance. Nonetheless, with respect to moisture content remain to be made some developments;
- this indicates that mixes containing high content of bitumen in the form of EA or separating the fine aggregates and coarse aggregates are appears to be better solutions to deal with the inadequate coating. Enhancing the aggregates coating, should effect in a lower water sensitive asphalt mixture. Furthermore, these developments in the mix design and including production process could increase resistance, which is already enhanced with respect to the conventional formulations. If the mixing procedure is optimized taking into account results for Tensile Strength Ratio (Figure 6.10), the first coating

should be performed on coarse aggregates with foamed asphalt, whereas fine aggregates should be coated by emulsified asphalt afterwards. An adequate time gap between the two coatings is one minute. However, to reach better performance further investigations are needed;

- the laboratory based complex modulus experimental results are considered satisfactory since they respect the 2S2P1D rheological model. As well as a master curves are plotted using a shifting procedure at a reference temperature of 5°C. FDR single coated (Mix-A and Mix-B) and FDR double-coated (Mix-C and Mix-D) asphalt mixes are satisfies the Time - Temperature Superposition Principle (TTSP);
- the results confirm that for the study of FDR it is necessary to refer to models and principles used in the rheological analysis of viscoelastic material. It should be noted that, the binder type had an influence on the glassy modulus that is moderately higher for mixtures treated with foamed asphalt (Mix-B and Mix-D). In addition to this, Mix-C showed relatively lower values of complex modulus over all the range of temperatures and frequencies tested. This may be due to the less cohesion that characterizes the FDR emulsion-emulsion double coating mixture (Mix-C);
- the hypothesis is that the residual water trapped in the mixture, after the first emulsion coating step, reduces the adhesion of the bitumen once the second coating step takes place. This may result in a non-homogeneous distribution of the bitumen film thickness, and consequently the formation of weaker points that decrease the mechanical performance of the mixture. Indeed, it should be considered that the bitumen film thickness has an important influence on the rheology. In this case, it means that the time-span between the first and second coating should be handled in a way to remove or evacuate the residual moisture. Therefore, additional work is needed to study and apply a possible solution during the mixing phase;
- the results revealed also that, FDR double coated foam – emulsion asphalt mixture (Mix-D) increases stiffness approximately 49.32 % when comparing with the FDR double coated emulsion – emulsion asphalt mixture (Mix-C). In this case, the problem of the residual moisture is overpassed because foam technology application (first step of coating) reduces significantly the amount of total water in the whole process. From the

shift factors point of view, double coated mixtures could be reflected in a lower thermal susceptibility;

- after having solved the limits developed during this research, it will be required to validate the results obtained in the laboratory when the mixtures are produced in the Central mixing plant as well.

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CONCLUSION

This doctoral research program is based on the laboratory experimental and analytical investigations of mechanical behaviour of cold recycled asphalt materials (CRM). Due to its complexity, there are still some critical problems existing in this technology that require research and investigation. During this doctoral program, a research including a large experimental campaign on the characterization of bitumen treated full depth reclamation materials (FDR) was performed to solve relevant research problems. Various aspects involved in CRM characterization was investigated analytically and experimentally. The materials used, the scope and all the detailed experimental program and analysis are presented in four different papers, which are presented in this manuscript-based Ph.D. thesis.

The results of this research program have significantly contributed to the increase of knowledge in the field of cold recycled asphalt materials. For instance, we now have a better understanding of the role of each component of the mixtures on its mechanical behaviour. We also have a better knowledge on the effect of the binder on the FDR materials and also a better understanding on how to optimize the mix design with emulsified asphalt and/or foamed asphalt.

More specifically, the principal aspects studied in this research program are the rheological behavior of cold recycled asphalt materials with different contents of recycled asphalt pavements, and the effect of confining pressure on the complex modulus. The impact of the compaction and curing temperature on the behavior of cold bituminous recycled materials, and the optimization of the binder type by the use of double coating was also studied. Here is a summary of the principal findings of this project.

For CRM, the environmental conditions during mixing and during curing are very important. Because of this, we studied the effect of low temperature. It is possible to state that low mixing, compaction and curing temperatures have highly influenced the final mechanical properties of the mixes. Nevertheless, other conclusions can be deducted:

- Results from Marshall and ITS tests show the same general trend, even if ITS results are more sensitive to low temperatures with lower variability;
- Mixes produced with foamed asphalt or bituminous emulsion showed similar results (of both ITS and Marshall) if produced at room temperature;
- In general, foamed asphalt mixes are less problematic at low temperatures. This can be caused by the lower water content in the mix and the respective different cohesion development kinetic;
- Compactability is globally improved by low temperatures, probably because of RAP, which is closer to a black rock behaviour when used at “cold” temperatures;
- An additional curing period at 23°C for all mixes did not have a strong impact on bituminous emulsion mixes; on the other hand, foamed asphalt mixes were able to recover part of the residual strength;

Then, the linear viscoelastic behavior of cold recycled emulsified asphalt mixtures with various percentages of RAP has been analyzed. Complex modulus testing was done on MR5, MR6 - 75%, MR6 - 85%, and MR7 asphalt mixtures with six temperatures and six frequencies respectively. It was observed that MR7 asphalt mixture exhibits high stiffness value at high frequency and low temperature. This can be explained in part by its high total binder content. On the other hand, at lower frequencies and higher temperatures, the stiffness value approaches a limiting value which possibly depends on the aggregate skeleton. Also, the combination of MR7 and MR5 have higher phase angle value than MR6 mixtures. From this consideration, it can be said that the MR7 and MR5 mixtures have higher viscous components than the MR6. This could lead to the conclusion that, contrary to what is found in the literature, the amounts of RAP do not have a strong influence on the phase angle, but more work is needed to support this statement.

From a pavement design standpoint, the moduli measured in this study do have a big impact. However, since different pavement structure are achieved with those different materials, the stiffest material, the CIR, ended up the least performant structure.

In order to see if CRM behaves more like granular materials or cohesive materials at young age, a comprehensive study on use of confining pressure was performed. Based upon the test results, it was found that confining pressure has an influence on the complex modulus of the FDR mixtures, mainly on the elastic component. Different cure length were also tested, and it was clearly shown that the curing period influences the ITS values and moisture damage. Basically, a longer cure means more durable materials.

In order to increase the moisture damage resistance of CRM, a study was carried out to optimize the binder type, which was done with the use of a combination of emulsion and foam (double coating). Prior to mechanical testing, two-stage mixing procedure was done to produce specimens with uniform binder percent and same volumetric properties. This new two-stage mixing technique is primarily based on pre-coating a portion of the aggregate with the suitable quantity of optimum binder content. Based on the ITS and Marshall stability test the, the CRM mix with foam and emulsion results in better performance.

RECOMMENDATIONS

Based on the observations from this study, the following recommendations are made for future studies.

- More work is needed to develop criteria for different confining pressures, frequencies with various temperatures, and different pavement conditions;
- With more results, it will be possible to use the true value of complex modulus for FDR materials in pavement design, which should result in more accurate designs;
- The study on the effect conferred by low temperatures to CRM could be improved by performing complex modulus tests. In addition to this, results should cover a wider gap of curing time, and consider water susceptibility together with dry conditions;
- The laboratory experimental outcomes characterize a first promising step concerning a new type of production process for the cold mix asphalt materials. This is the case for the two-phase mixing process with coarse aggregates separated which is currently not adequate enough for cold in place recycling but can be used for central plant recycling unit;
- After having solved the limits developed during this research, it will be necessary to validate the results obtained in the laboratory when the mixtures are produced in the central mixing plant;
- It is recommended that future studies include a life cycle cost analysis to help choose the FDR optimum structure and material.

APPENDIX I

REVIEW OF MIX DESIGN PROCEDURES FOR COLD RECYCLED ASPHALT MATERIALS

Recently, many mix design methods have emerged in an effort to improve the CIR process as a viable method for pavement rehabilitation. Methods proposed by different agencies and groups that appear to have the most developed mix design procedures for CIR (Epps & Allen, 1990):

- California Test 378.
- Chevron USA, INC. Mix Design Method
- Corps of Engineers
- Nevada
- Oregon Mix Design
- New Mexico
- Pennsylvania Mix Design Method
- Purdue
- Texas
- Indiana (Tia et al., 1983)
- The United Kingdom (Stock, A. F., 1987)
- Ontario (Emery, 1993)
- Israel (Cohen et al., 1989),

Table - A I-1 summaries the mix design procedures, and the sampling and testing techniques used by some of these organizations(Oqueli, 1997). The methods are generally very similar. All but one uses the Marshall and Hveem tests. Kneading or gyratory compaction is used by some, while others use the Marshall method. The main differences are in the addition of new aggregate, and in curing time and temperatures.

Table - A I-1 Summary of mix design and testing procedures, performed or studied by different organizations
Taken from Oqueli (1997)

ORGANISATION	SAMPLING	DETERMINATION OF RAP PROPERTIES	ADDITION OF NEW AGGREGATE	DETERMINATION OF EMULSION REQUIREMENT	MIXING	COMPACTION	CURING	TESTING
STATE OF CALIFORNIA	• Pavement cores. • Large pieces crushed in the laboratory	• Bitumen content by Abson recovery test. • Aggregate grading. • Viscosity from Abson recovered bitumen	• None	• Done by means of the surface area of aggregate.	• Mixing is done by adding 2% water and different emulsion contents	• By kneading compactor at 60°C.	• Mixed specimens are cured loose at 60°C for 16 hours	• Hveem stability at 60°C. • Air void determinatio n. • Specific gravity
STATE OF PENNSYLVANIA	• Pavement cores. • Bags from stockpile	• Bitumen content by Abson recovery test. • Aggregate grading. • Viscosity at 60°C. • Penetration at 25°C.	• Up to 50%	• Determined by total bitumen needed by the RAP aggregate after extraction, and calculated by means of the aggregate surface area.	• Mixing is done by hand. • Emulsion content kept at 2.5% by weight and heated to 60°C. • 3% of MC is used with increments of 1%.	• Compactions at 23°C using a Marshall hammer applying 75 blows per face.	• Loose curing at 41°C for 45 minutes. • Mould curing at 23°C for 15 to 24hrs. • Extruded specimens cured at 40°C for 3 days.	• Resilient modulus at 25°C. • Marshall Stability and flow. • Bilk specific gravity. • Vacuum saturation and soaking at 25°C to determine modulus.

STATE OF OREGON	• Field samples reduced to 100% passing the 25mm sieve.	• Bitumen content by Abson recovery test. • Aggregate grading. • Absolute viscosity at 60°C. • Penetration at 25°C • RAP grading.	• Not Mentioned	• Estimated usinf absolute viscosity and penetration charts from previous charts from previous projects or by established formula	• Hand mixed with preheated emulsion at 60°C for 1hr. • Water content of 0.5%,1.0% and 1.5% are used	• kneading compaction at 60°C applying 50 blows. • Re-compaction using kneading compaction at 60°C applying 50 blows	• Loose curing at 60°C for 1 hr. • Mould curing at 60°C for overnight. • Further Mould curing at 60°C for 24hrs. • Extruded specimens cured at room temperatur e for 72 hrs.	• Hveem stability. • Resilient modulus.
PURDUE UNIVERSITY INDIANA	• Not Stated	• Bitumen content. • Clean aggregate grading.	• New aggregate added depending on Clean aggregate grading.	• Optimum emulsion content determined by a gyratory compactor machine	• Mechanically mixed for 2 minutes and half a minute hand mixing	• By gyratory compactor using of 20 revolutions at 1400kPa and 60 revolutions at 1400kPa.	• Mould curing for 24 hrs. at room temperatur e or 60°C.	• Unit weight • Resilient modulus • Hveem R-value • Marshall stability and flow.

CHEVRON USA INC.	• Crushed material to produce laboratory specimens.	• Bitumen content. • Aggregate grading. • Viscosity at 60°C. or Penetration at 25°C. • Viscosity at 135°C.	• New aggregate added if required.	Determined by either: • Centrifuge kerosene equivalent or, • Aggregate surface area formula	• Mixed at different recycling agent contents.	• By kneading compactor at applying 10 to 50 blows per face and 175 kN plunger method at 73°C.	• Mould curing at 23°C for 24hrs. • Final Mould curing at 23°C for 72 hrs • Water soaked curing under vacuum saturation at 100mm mercury.	After final curing: • Hveem resistance and cohesiometer . • Resilient modulus. • 4 days vacuum desiccation at 10-20mm mercury
ONTARIO	• Representative sample from millings or from • coring.	• Moisture content. • Bitumen content. • Aggregate grading. • Penetration.	• New aggregate added if required.	• Emulsion content is determined from plots of density, air voids, stability at 22° and 60°C against the percentages of emulsion used.	• Mixing is done with five different emulsion contents heated at 60°C at estimated field • Moisture content.	• Compaction using Marshall hammer applying 50 blows per face after loose curing. • Re-compaction applying 25 • blows per face	• Loose curing at 60°C for one hour. • Mould curing overnight at 60 °C. • Mould curing on their side for 24 hrs. at 60°C. •	• Marshall stability and flow at 22°C and at 60°C. Bulk relative density.
ASPHALT INSTITUTE	• Obtained randomly from the field.	• Bitumen content. • Aggregate grading.	• New aggregate only added to correct RAP grading.	• Emulsion content determined by means of an aggregate surface area formula presented in the asphalt Institute Manual No. 21.	• Determined by the agency using the method.			

APPENDIX II

AN APPROACH TO OPTIMIZE THE DOUBLE COATING MIX DESIGN PROCEDURE

As showed in Chapter 6, the FDR emulsion-foam double coating mixture (Mix - D) shows a satisfactory value of ITS moisture sensitivity. Aiming to increase this value and to determine the optimal mixing procedure for the double coating, three parameters were tested as follows:

- The order of the aggregate sizes addition during the mixing phase. The FDR material was divided in: coarse parts, from 5 mm to 14 mm, and fines parts, from 0 mm until 2.5 mm. Two combinations were tested using: 1) the coarse aggregates in the first part of the mix and the fines in the second, 2) the fine aggregates in the first part of the mix and the coarse grains in the second;
- The type of binder for the first and second coatings. The following binders were used: emulsion and foamed bitumen;
- Rupture time: it was used a time span of 1 and 2 minutes.

Considering the above mentioned parameters, eight different combinations representing eight different procedures for the double coating were carried out. For each combination, 48 samples were compacted with the Marshall hammer (8 Combinations x 2 curing conditions (wet and dry) x 3 ITS Tests). The specimens were compacted to test the dry and wet ITS. Table - A II-1 shows each combination and their results.

Table – A II-1 The double coated FDR materials with eight different Combinations.

Combination	Double Coating procedure	Avg. ITS_{Dry} (kPa)	Avg. ITS_{Wet} (kPa)	TSR (%)
1	1 coating: CAF_a 2 coating: FAE_a Rt: 1 min.	334.66	324.57	97
2	1coating: CAF_a 2 coating: FAE_a Rt: 2 min.	283.18	276.29	98
3	1 coating: FAF_a 2 coating: CAE_a Rt: 1 min.	389.19	329.11	85
4	1 coating: FAF_a 2 coating: CAE_a Rt: 2 min.	372.72	278.57	75
5	1 coating: CAE_a 2 coating: FAF_a Rt: 1 min.	388.03	336.94	87
6	1 coating: CAE_a 2 coating: FAF_a Rt: 2 min.	377.08	348.67	92
7	1 coating: FAE_a 2 coating: CAF_a Rt: 1 min.	378.74	305.41	81
8	1 coating: FAE_a 2 coating: CAF_a Rt: 2 min.	427.10	271.92	64

Note: CAF_a= Coarse Aggregate with Foamed Asphalt; FAE_a= Fine Aggregate with Emulsified Asphalt; FAF_a= Fine Aggregate with Foamed Asphalt; CAE_a = Coarse Aggregate with Emulsified Asphalt; Rt= Rupture time.

In order to achieve the optimal mixing recipe for the double coating, it is necessary to understand which factors are significant and which are negligible. A variable or factor is any parameter that has, in reality or all likelihood, an influence on the studied phenomenon. The factors are considered as the possible causes of the response. To determine which parameters have a stronger influence on the results a sensitivity analysis was conducted using a multiple regression analysis, i.e. considering all the factors at the same time. When several variables are treated at the same time, they are expressed in different units, and it is thus convenient to convert them to a common scale to proceed with the model implementation. For this reason, the variables were standardized so that the coefficients are not dependent on the measurement unit(Ayyub & McCuen, 2011). The following parameters were investigated:

- x_1= order of the aggregates addition in the procedure (fines or coarse)
- x_2= order of addition of the type of binder in the procedure (emulsion or foam)
- x_3= time span between the first and the second phase of the mix

A linear model with interactions was developed. In Table – A II-2 is summarized all the coefficients of the model considered for the sensitivity analysis.

Figure AII-1 represents the distribution of the measurements in the experimental domain investigated. The points highlighted in Figure - A II-1 correspond to the combinations tested and reported in Table AII-1.

Table - A II-2 Summary of coefficients used in model.

Coefficient notation	**Type of coefficient**	**Coefficient meaning**
x_0	Constant factor term	Constant
x_1	Main effects	Order of aggregates
x_2		Order of binders
x_3		Time span
x_{12}	Two-way interaction effects	Order of aggregated/order of binders
x_{13}		Order of aggregates/Time span
x_{23}		Order of aggregates/Time span
x_{123}	Three-way interaction effect	Order of aggregated/order of binders/Time span

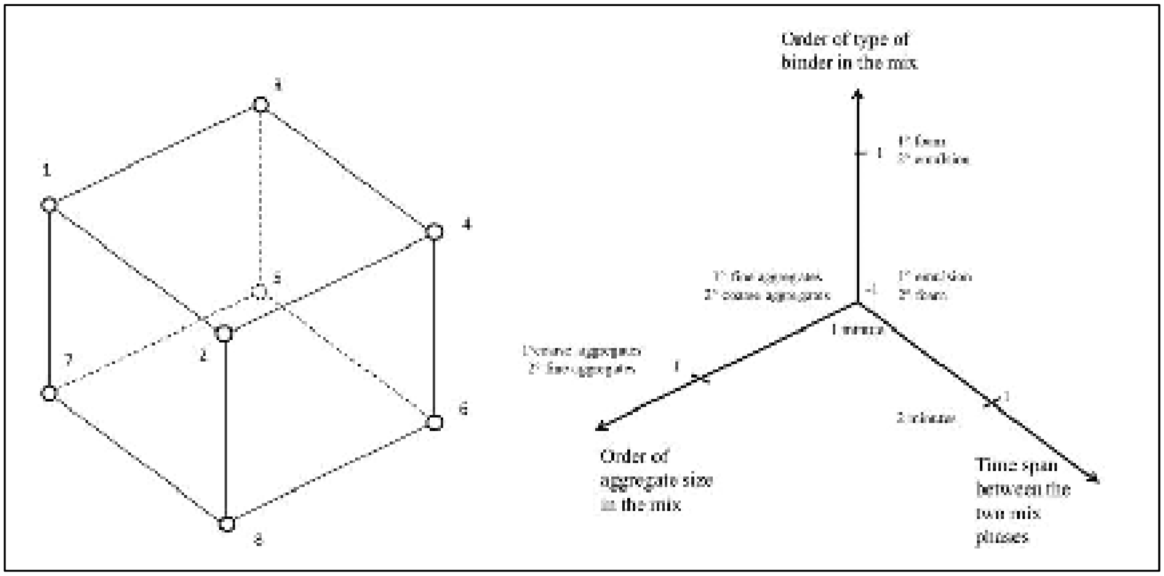

Figure - A II-1 Distribution of measurement points in experimental domain

To calculate the coefficients of the model that describes the phenomenon in the specified validity range, it is possible to use the least square fit algorithm (Equation AII-1). Dividing the coefficients for the constant term (x_0) allows the relative effects to be obtained (Equation A II-2):

$$x = (Z'Z)^{-1} \cdot Z'y \qquad \text{(A II-1)}$$

$$x_i^* = \frac{x_i}{x_0} \qquad \text{(A II-2)}$$

Where:

y = vector (n x 1) of the observation of the dependent variable (measurements of ITS or TRS)

Z = model matrix (n x (k+1))

x = vector ((k+1) x 1) of unknown coefficients of the model

ε = vector (n x 1) of stochastic errors

Figure A II-2 to A II-4 compares the relative effects obtained from Equation A II-2. For the dry results the most important parameters are the order of the addition of the aggregates and the order of the binder type. The time span between the two coatings (x_3) is not relevant. The sign of the effect x_1 (aggregates order) is negative, that means that if small aggregates are used in the first mixing phase (first coating) the ITS will increase and vice versa. The sign of the effect x_2 (type of binder) is negative, that means that the best results are obtained using the emulsion during the first coating. The two-way interaction terms are not negligible (x_{12}; x_{13}, x_{23}), that means that the factors have a mutual influence.

Analyzing the results for the dry specimens, it is possible to conclude that the optimal procedure to first coat the fine aggregates with emulsion and afterwards coat the coarse aggregates with foam bitumen.

On the other hand, as can be seen in Figure - A II-3, for wet results x_1 is positive which means that using coarse aggregates in the first mixing phase the ITS is higher. The time span between the two coatings (x_3) becomes important for the water resistance and its effect is negative, which means that increasing the rupture time between the coatings the ITS decreases. The order of the type of coating (x_2) is less relevant in this case. The interaction terms are not negligible (x_{12}; x_{13}, x_{23}), that means that the factors have a mutual influence and that the factors are not independent from each other.

Analyzing the results for the wet specimens it is possible to conclude that the optimal procedure is to first coat the coarse grains with the emulsion and afterwards coat the fines with the foam bitumen. Contrary to what happens with the dry specimens, for the wet ones the rupture time is important. Finally, as it can be seen from Figures - A II-3 and A II-4, treating separately the results, the definition of the optimal recipe is not unique. Indeed, it depends of the type of criterion used (wet or dry ITS).

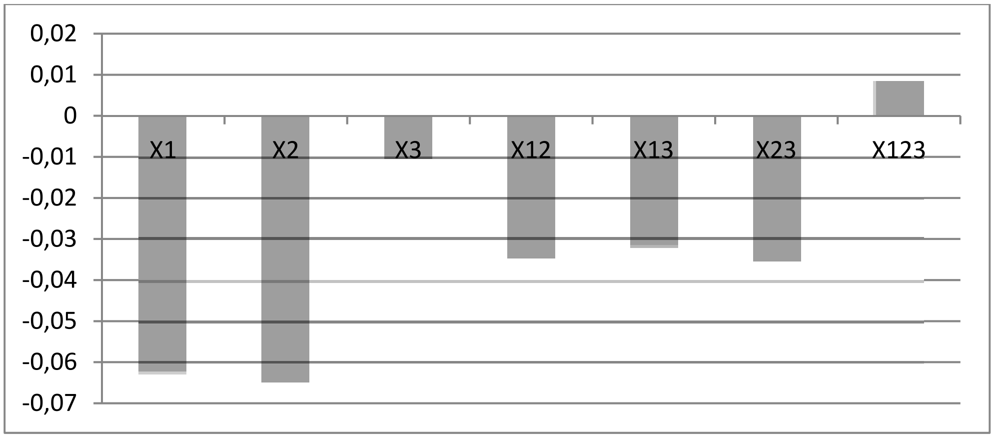

Figure - A II-2 Relative effects in case of ITS dry

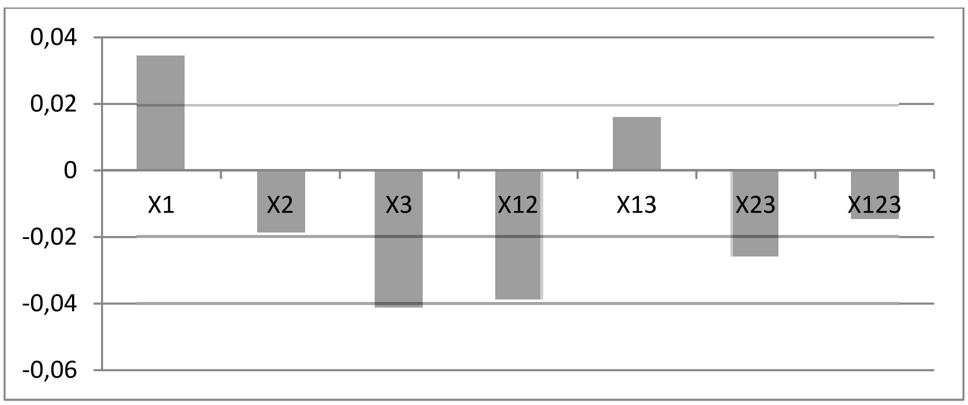

Figure - A II-3 Relative effects in case of ITS wet

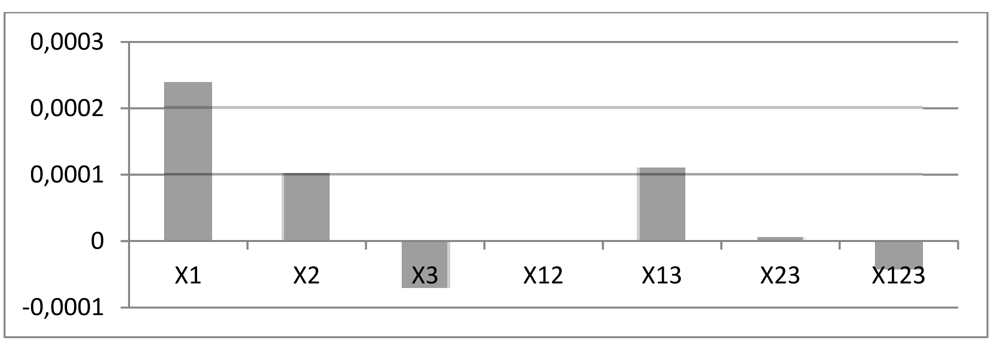

Figure - A II-4 Relative effects for the ratio between the ITS dry and ITS wet

As shown in the Figure - A II-4, analyzing the results for the ratio between the results characterizing dry and wet specimens, it is possible to conclude that the optimal procedure is to first coat the coarse part with foam and afterwards coat the fines with the emulsion. The appropriate time span between the coatings is one minute. In this case the interactions results less significant and could be ignored (with the exception of x_{13}).

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