Investigating the effect of temperature and aging on the failure mechanism of cold recycled mixes with bituminous emulsion

by

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Étudier l’effet de la température et du vieillissement sur le mécanisme de fissuration des enrobés recyclé à froid avec émulsion bitumineuse

Mohammad BEHESHTIGHANAD

RÉSUMÉ

L’application des enrobés recyclé à froid est utilisée depuis des décennies. Aussi, plusieurs projets de recherche ont été réalisés sur des matériaux recyclés à froid. Compte tenu de la variété des applications de mélange et du comportement complexe des enrobés recyclé à froid, les chercheurs ont manifesté un intérêt à étudier la performance de ces mélanges dans différentes conditions. Par conséquent, il y a des aspects découvrables du comportement de ces matériaux à étudier. Par exemple, la fissuration de la chaussée en asphalte est un problème controversé qui a été étudié par des chercheurs du monde entier. Toutefois, il n’existe pas de protocole communément accepté pour tester les mélanges d’asphalte froid pour la caractérisation de la résistance à la fissuration. Compte tenu du comportement des enrobés recyclé à froid qui se situe en quelque sorte entre les matériaux granulaires et l’asphalte mélangé à chaud (HMA), il est très important de mieux comprendre ses performances et le mécanisme de défaillance. La résistance à la fissuration en tant que paramètre influent dans la performance des enrobés recyclé à froid est soumise dans cette étude. Ce paramètre est étudié dans cette recherche en simulant diverses conditions de terrain, y compris différentes températures et différents vieillissements, afin de construire des fondations pour la prédiction des performances couramment utilisées dans les procédures de conception de chaussées en asphalte. Le test de *Semi-Circular Bending* (SCB) est choisi comme test de laboratoire couramment utilisé pour caractériser la résistance à la fissuration des mélanges d’asphalte. En outre, le test de traction indirecte (*Indirect Tensile Strength* (IDT)) est utilisé pour évaluer la sensibilité à l’humidité des mélanges. Les mélanges proposés dans cette étude comprennent trois modèles de mélange avec une teneur en bitume différente. Les tests sont effectués en trois étapes de vieillissement qui ont été simulées à l’aide d’un four en laboratoire et de trois températures différentes. La résolution de ces problèmes contribuera à améliorer les procédures de conception des mélanges et à optimiser le contrôle de la qualité et l’assurance de la qualité des mélanges d’asphalte, en particulier lorsque des composants complexes tels que des matériaux recyclés sont couramment incorporés dans le béton bitumineux de nos jours.

Mots-clés: Enrobé recyclé à froid, *Semi-Circular Bending* (SCB), *Indirect Tensile Strength* (IDT)

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ABSTRACT

The application of cold recycling mixtures has been used for decades, and several research projects have been done on cold recycled materials. Given the variety of mixture applications and the complex behaviour of cold recycled mixtures (CRM), there has been an interest among researchers to investigate the performance of these mixtures in different conditions. Therefore, there has been discoverable aspects of CRM behaviour to be investigated. For instance, cracking in asphalt pavement is a controversial problem that has been studied by researchers around the world. However, there is not a commonly accepted protocol for testing asphalt mixtures for cracking resistance characterization because of variability of test results, non-uniformity in test specimens, and overall complexities of the tests that prevent them from being adopted for daily uses. Considering the behaviour of CRM which is somehow between granular materials and hot mix asphalt (HMA), it is highly significant to better understand its performance and the failure mechanism of these materials. Cracking resistance as an influential parameter to evaluate the performance of CRM studied here. This parameter was investigated through the research by simulating various field conditions including different temperatures and different aging in order to build foundations for performance prediction commonly used in asphalt pavement design procedures. Semi-Circular Bending (SCB) test was selected as a commonly used laboratory test to characterize crack resistance of asphalt mixtures. Besides, Indirect Tensile Strength (IDT) was employed to evaluate moisture sensitivity of mixes. The proposed mixes in this study include three mix designs with different binder content. The tests were conducted in three stages of aging which simulated using a draft oven in the laboratory and three different temperatures. Addressing these problems will help to improve mixture design procedures and optimize quality control and quality assurance of asphalt mixes, especially when complicated components, such as recycled materials are commonly incorporated into asphalt concrete nowadays.

Keywords: Cold recycled mixture (CRM), Semi-Circular Bending (SCB), Indirect Tensile Strength (IDT)

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LIST OF ABBREVIATIONS

AC	Asphalt Concrete
AV	Air Voids
BSM	Bituminous Stabilized Material
CBTM	Cement Bituminous Treated Materials
CIR	Cold In-place Recycling
COV	Coefficient of Variation
CR	Cold Recycled
CRM	Cold Recycled Materials
CRM	Crumb Rubber Materials
CSS-1	Cationic Slow Setting
EPFM	Elastic Plastic Fracture Mechanics
FDR	Full Depth Reclamation
FPZ	Fracture Process Zone
HFMS-2P	High Flow Medium Setting with Polymer
HFMS-2S	High Flow Medium Setting with Solvent
HIR	Hot In-place Recycling
HMA	Hot Mix Asphalt
IDT	Indirect Tensile Strength
ITR	Indirect Tensile Strength Ratio
ITS	Indirect Tensile Strength
LEFM	Linear Elastic Fracture Mechanics

LTOA	Long-term Oven Aging
NMAS	Nominal Maximum Aggregate Size
PMS	Pavement Management System
RAP	Reclaimed Asphalt Pavement
SCB	Semi-Circular Bending
SGC	Superpave Gyratory Compaction
SIF	Stress Intensity Factor
STOA	Short-term Oven Aging
VFL	Voids Filled with Liquid
XLTOA	Extra Long-term Oven Aging

INTRODUCTION

Road networks have been playing a significant role in human lives. In the past decades, population growth and economic development have resulted in an expansion in road networks. Aside from the existing roads, thousands of kilometres were added to the networks to meet the demand and provide safe traffic for the increasing road transportation rate. Canada has over one million kilometres of roads which connect different locations all around the country. Among these roads, around 40% are paved and 95% of them are flexible pavement and the rest are paved with concrete, so rigid or composite pavement. Since safety and efficiency are vital to support consistent traffic on the road, poor pavement condition leads to poor ride quality and potential safety concerns. Therefore, construction for expansion purposes and rehabilitation of existing roads are the main concerns for transportation agencies.

The process of road construction and maintenance needs a considerable amount of resources. Asphalt concrete is the most common type of material for asphalt-paved roads which mostly consists of limited resources including aggregate and bitumen. This reveals a large amount of natural resources for the construction and maintenance of road infrastructure. Besides, a large amount of fuel is consumed for the process of asphalt production and transportation related to construction. Considering the exposure of roads to different climate conditions, deterioration happens after a certain time. Thus, maintenance and rehabilitation are investable needs for road networks. During this process, a considerable amount of waste is generated which reusing them has double sided benefits. Most importantly, it reduces the burden on landfills. In addition, natural resources will dramatically be preserved by recycling asphalt materials. Finally, In 1970's the economic concerns and development in asphalt recycling equipment resulted an increasing interest in using asphalt recycling methods in the world (Ferjani et al., 2019; Miliutenko et al., 2013).

Regarding the structure of flexible pavement, the surface layer plays a role to transfer traffic loads to other pavement layers in order to distribute the load based on layers properties to withstand the tensions (Singh & Sahoo, 2020). The surface layer is exposed to harsh weather conditions, touches of wheels, and temperature fluctuations. This layer has a crucial role in pavement durability and performance. However, most common distresses start from the surface and propagate to other layers, there is a combined interaction in performance of each layer to result in an acceptable pavement condition. Previously, hot mix asphalt (HMA) was used as the only material in flexible pavement either in construction or rehabilitation. However, asphalt recycling made it possible to reuse Reclaimed Asphalt Pavement (RAP) coming from milling the existing pavement as a new layer. Additional binder like bituminous agents or cementitious agents can be added to the RAP to recover their properties as a new asphalt layer (*Asphalt Recycling & Reclaiming Association ARRA*, 2001; Xiao et al., 2018).

Although it has been decades since asphalt recycling methods were introduced and applied to the pavement industry, due to its complex behaviour in different conditions there is not a commonly accepted standard for testing and characterization of Cold Recycled materials (CRM). Thus, the reactions of cold mixtures to different temperatures, humidity, traffic load, and other parameters require a better understanding. In addition, the sensitivity and validity of common tests for research purposes have not been fully proven given the lack of data quantity. By addressing these problems, production of cold mixtures along with quality control and quality assurance will be improved (Chen & Solaimanian, 2019).

The goal of this research is to better understand the crack resistance of cold recycled mixtures with the presence of bitumen emulsion and cement. Besides, the effect of different aging stages and various temperatures is investigated in this research. Two popular methods for CRM characterization are either acquiring core samples from fields or manufacturing specimens and simulating field conditions in the laboratory. Considering the availability to apply field conditions and prepare mixtures with a wide variety of parameters in the laboratory, this procedure was chosen over specimens cored out of the fields. Additionally, to investigate the effect of aging, it is rarely possible to collect cored samples with short-term aging conditions from fields.

To characterize failure mechanism of CRM, two main tests were selected. Semi-circular bending (SCB) test and indirect tensile strength (IDT) are used to investigate the fracture and cracking resistance of CRM. To better understand CRM behaviour, the above-mentioned tests were performed in various temperatures with samples in three aging levels. Specimens were manufactured with different bitumen content to study the influence of this additive on the results of the tests.

CHAPTER 1

RESEARCH PROBLEM AND OBJECTIVES

This chapter aims to present an overview about the general aspects of recycling asphalt pavements as well as the advantages and disadvantages of using CRM. In addition to the basic information, the main problem which is addressed in this study for the procedure of recycling asphalt pavement is presented in this chapter.

1.1 Background

A noticeable amount of funding is spent each year on construction maintenance and rehabilitation of road pavement to improve their serviceability and functionality. The road network plays an undeniable role in human life. The majority of these road networks are paved with asphalt pavement materials. These materials are made of asphalt binder, graded aggregates and in some cases, there are other additives and recycled materials. Since these materials are viscoelastic, their behaviour is highly dependent on the loading rate and temperature. Recently due to the advantages of using existing asphalt pavement, recycling technology in the rehabilitation process has gained great attention (Chen & Solaimanian, 2019; Y. Wang et al., 2018). Additionally, Xiao et al., 2018 showed the potential saving for cold in-place recycling (CIR) technology in comparison with conventional Hot Mix Asphalt (HMA) methods. Based on (Xiao et al., 2018), saving 62% aggregates consumption, 52% in greenhouse gas emission, 61% in sulfur dioxide, and 54% in nitrogen dioxide are the benefits of using CIR technology. Over 50 years ago when the initial use of asphalt recycling was started to get popular, roads with low traffic were the candidates for recycling but over the development of recycling methods, heavy-traffic roads are considered and the ratio of using RAP has been increased close to 100% (Karlsson & Isacsson, 2006). At the same time, the expansion of roadways resulted in the production of road waste materials. Managing these amounts of waste materials would be a challenging task for governments because

RAP consists of aggregates and asphalt cement. Therefore, it had the potential to remain on the sites and was used for rehabilitation purposes (Behnood et al., 2015a; Graziani et al., 2016).

There are various methods for recycling asphalt materials. Choosing the proper strategy depends on pavement conditions and existing distresses along with the period of pavement life. In General, Hot In-place Recycling (HIR) is used when a limited volume of distresses exists in the pavement and mostly related to the upper part of the pavement. HIR is employed when the pavement structure can withstand the traffic load. In case of existing higher volume of non-load-related distresses, Cold In-place Recycling (CIR) can be selected. However, load-related distresses can be treated with CIR with an overlay. Full Depth Reclamation (FDR) is another type of asphalt recycling for the purpose of reconstruction, lane widening, and increasing structural capacity. FDR has the capability to improve pavement profile while it addresses a wide range of distresses (Gandi et al., 2015). Cold In-place Recycling (CIR) uses existing bituminous materials with high percentages of RAP, up to 100%. In CIR method, bitumen as foam or emulsion is added to the material to partially restore the strength properties of pavement. Therefore, Bitumen-Stabilized Materials (BSM) behave differently from HMA. Contrary to HMA that bitumen covers all sizes of aggregates, added bitumen to CIR whether it is foam or emulsion, only dispersed among finer particles (*Asphalt Recycling & Reclaiming Association ARRA*, 2001; Dal Ben & Jenkins, 2014). Although these technologies have proven huge potential for saving resources, there are still some problems in terms of their performance that are not addressed by researchers so far.

To reach the most desirable mixture, the process of mix design is undergone to optimize ingredients in CRM and improve performance. However, a national accepted method to produce CRM is not available, agencies use different mix design procedures. In addition, to manufacture laboratory specimens for research purposes, there is not a universally accepted procedure for cold-recycled mixtures. Therefore, research centers are developing their procedures based on laboratory tests and field experiences (Tebaldi et al., 2014). Another way to investigate the performance of CRM and improve mix design is to collect field performance data. Then, apply the simulation of traffic loads with accelerated loading of full or scaled test tracks. These methods take a long time to be processed and require noticeable funding. On the other hand, using bench scale laboratory tests is much faster and cheaper approach to characterize asphalt mixtures (Chen & Solaimanian, 2019). Moreover, it is applicable to impose various conditions while characterizing asphalt mixtures.

Among all distresses for asphalt pavements, there are three critical ones including fatigue cracking, permanent deformation (rutting), and thermal cracking. These major distresses can be controlled with proper mix design, structural design, and an accurate construction process. Cracking which is the focus of this research, is critical potential distress in CIR due to the presence of aged bitumen. When cracks propagate, ravelling (loss of aggregates) happens in the latest stage of crack propagation. Afterward, ravelling leads to penetrating moisture into the pavement structure and deteriorates pavement (Chen & Solaimanian, 2019; Ozer et al., 2016). Permanent deformation (rutting) is a major distress that affects the pavement structure. Rutting is divided into two categories: structural rutting and material rutting. Both types of rutting refer to plastic flow of asphalt layers (Xiao et al., 2007b). After conducting an intensive literature review to select the proper laboratory tests for evaluating crack resistance in CRM, Semi-circular bending (SCB) and indirect tensile strength (IDT) are employed for this study.

1.1.1 Problem Statement

Given the unique behaviour of CRM in comparison with granular materials and HMA, it is critical to better define the performance of this material. However, researchers have been struggling to characterize the fracture behaviour of complex mixtures like CRM. There are no specific mechanical tests for routine mix designs and quality control and quality assurance.

There are various parameters that affect the performance of cold recycled mixtures. First, the production of CRM requires the use of various materials. For instance, RAP, as the main component, have different properties considering their sources and the condition of pavement before milling. Besides, it is common to use other materials like Crumb Rubber Materials

(CRM), cement, and different types of bitumen in CRM. Therefore, each mentioned component makes a specific mixture with exclusive properties. This fact proves the necessity to investigate the performance of CRM with different variables. Understanding the fracture behaviour of CRM could lead to better utilization of CRM technology and have significant benefits in cost savings and environmental conservation. Due to the presence of aged bitumen within the RAP, CRM with high percentages of RAP mostly have an issue in fracture resistance. On the other hand, added bitumen in the form of emulsion or foam could affect fracture properties of mixtures. Investigating the effect of aging and temperature on failure mechanism of CRM will help to better understand CRM behaviour and optimize more durable mix designs.

Considering the strength development of asphalt concrete over time, the effect of aging is a significant parameter to be investigated in asphalt mixtures. Considering the high volume of RAP in the mixtures that consist of old binder along with unaged bitumen in the form of bitumen emulsion, the influence of aging on the properties of CRM is complex and requires better understanding to optimize the performance. Besides, temperature is another variable that affects performance of asphalt mixes due to the viscoelastic properties of these materials.

1.2 Research Objective

The main goal of this research is to evaluate cracking resistance of cold recycled mixtures with RAP at different temperatures and aging conditions. To characterize the fracture properties of the proposed mixtures, Indirect Tensile Strength (IDT) test and Semi-circular bending (SCB) test are employed.

1.3 Scope of work and research approach

This research has well-defined tasks to cover all components of the study, including background information, material explanation, specimen preparation, testing, results, and analysis. Reclaimed asphalt pavement (RAP) that is selected for the research was obtained from DJL construction site in Québec. Throughout the research, the selected fracture tests, semi-circular bending (SCB) test and indirect tensile strength (IDT) are used to characterize the crack resistance of the mixtures in various conditions. The mixtures are tested in three different temperatures of -20, 0, and 19ºC to cover a wide range of temperatures. Three different mix designs with 1.0%, 2.2% residual bitumen, and a mix with no additives are tested in this study. The effect of aging is investigated by applying short-term and long-term aging in the laboratory. Due to the possibility to control height and targeted air voids, Superpave Gyratory Compacter (SGC) is used to compact specimens for both tests.

To address the objectives of the research, three phases of research with seven tasks are provided and shown in Figure 1.1. Details of each task are presented afterward.

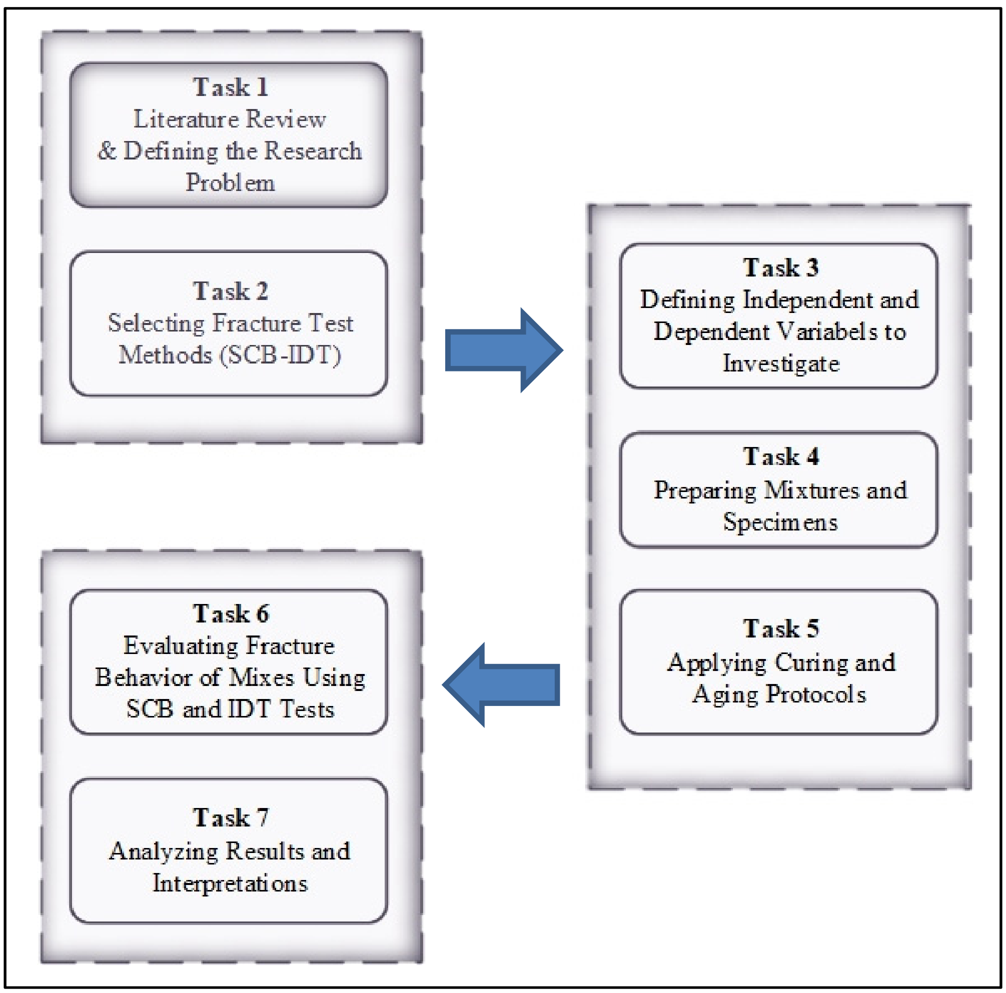

Figure 1. 1 Flow chart of overall research framework

Task 1. Literature Review and Problem Statement

This task consists of a comprehensive literature review and theoretical background study on the subject of Cold Recycled Materials (CRM) and more specifically the literature related to the characterization of CRM. The literature review was conducted in order to get familiar with all research and studies on the subject up to this point. As a consequence, by conducting the literature review the potential gap in the literature is defined and presented in this task.

Task 2. Selecting Fracture Test Methods

Semi-Circular Bending (SCB) test is selected as the proper laboratory test for routine mix design and quality control of asphalt paving materials. SCB is not a common test for CRM, but it can be used to evaluate the cracking propagation properties of that type of material. In addition, Indirect Tensile Strength test is chosen to evaluate the IDT strength and the water sensitivity of mixes. This task includes the details for conducting each test considering the specific conditions.

Task 3. Defining independent and dependent variables

To investigate the behaviour of CRM mixes, different variables were defined and considered in this research. Various temperatures, aging stages, and binder contents are the independent variables in the laboratory tests. In addition, water sensitivity, IDT strength, peak load, critical strain energy release rate, and Fracture toughness are the dependent variables.

Task 4. Preparing Mixtures and Specimens

This task includes the first step in the laboratory to prepare specimens. To do so, material properties are determined before and mixtures are manufactured according to the proposed mix designs and characteristics of ingredients.

Task 5. Applying Curing and Aging Protocols

In order to simulate field conditions, curing process is essential after compaction of specimens. Thus, the curing process is determined regarding common protocols in the literature and the previous studies at ETS. After curing, specimens were placed in an oven to simulate the aging process. Aging was assumed in three different stages including short-term oven aging (STOA), long-term oven aging (LTOA), and extra long-term oven aging (XLTOA).

Task 6. Evaluating Fracture Behaviour of Mixes

Fracture behaviour of cold mixtures is evaluated using SCB and IDT tests according to ASTM D8044 and ASTM D6931, respectively. To investigate the effect of independent variables (aging, temperature, and binder content), certain conditions are applied to the test procedure.

Task 7. Analyzing Results and Interpretation

At the end of the laboratory phase and specimen testing, the results were summarized and presented to clearly understand the performance of the materials. A variety of figures and tables are employed to better illustrate and interpret the results.

CHAPTER 2

LITERATURE REVIEW

This chapter of the research provides details on different types of cold recycling mixtures, their preparation fundamentals, possible approaches to characterize CRM, and their failure mechanism. However, a more detailed literature review will be presented in each chapter addressing its specific subject.

2.1 Asphalt pavement maintenance and rehabilitation

The process of rapid expansion of road networks has peaked many years ago and existing pavements are facing with deterioration and ending their service life. Due to the limited funding for reconstruction of roads, the preservation and extending service life of existing pavement has gained popularity in past decades. In general Pavement Management Systems (PMS) have been controlling and planning construction, preventative maintenance, and rehabilitation of roads network. Pavement maintenance is considered as a procedure for correction and preventative tasks to maintain durability and flexibility of the pavement. Since it does not increase the structural capacity, it is applied to pavements with good structural conditions. The effectiveness of these techniques varies based on the method that is applied to the pavement. It can vary from 1 to 2 years for fog seals and up to 10 years for HMA overlay (*Asphalt Recycling & Reclaiming Association ARRA*, 2001).

Contrary to maintenance activities, pavement rehabilitation comes when pavement condition has deteriorated and maintenance is no longer cost-effective. Pavement rehabilitation techniques are more expensive than maintenance activities. In both cases, using recycling methods will help to preserve resources. However, selecting the proper cold recycling technic in maintenance and rehabilitation depends on various conditions.

2.2 Cold recycling methods

Due to the recent developments in the pavement industry, the process of construction, maintenance and rehabilitation of asphalt pavement is performed using modern techniques, machinery, and equipment. This improvement led to implying various methods of recycling in road construction. However, the concept of using recycling material in asphalt pavement is not a recent task, using RAP and recycling techniques in developed countries goes back to 1900s (*Asphalt Recycling & Reclaiming Association ARRA*, 2001). Later on in 1970s, the increasing price of asphalt due to the energy crisis accelerated the demand for using recycling methods in asphalt industry (Bocci et al., 2011). During the development of cold recycling methods, mostly low traffic roads were candidates for cold recycling projects due to the lack of guidelines and frameworks for cold recycling applications. Besides, not having a clear understanding of the failure behaviour of CRM was another reason to limit this method into low traffic roads (Ameri & Behnood, 2012; Xiao et al., 2018).

Cold recycling is performed in two ways; in-place and in-plant. Cold In-place Recycling (CIR) minimizes hauling and every task is done by the recycling unit in the field. On the contrary, for in-plant method, recycling materials are transferred to a depot in plants. Then, recycling process happens in the plant. Cold in-place recycling is more environmentally friendly and cost effective than in-plant technic. However, controlling the variables is more precise in in-plant applications. In both methods, Reclaimed Asphalt Pavement (RAP) coming from milling of the existing pavement is the main material for cold recycling mixture (*Asphalt Recycling & Reclaiming Association ARRA*, 2001; Y. Kim & Lee, 2012; Xiao et al., 2018). Cold In-place Recycling (CIR) uses up to 100% RAP in the process of recycling asphalt. It usually addresses upper layers correction, 50mm to 100mm by adding binder to the RAP. While, using other additives like cement, lime, or fly ash is required to go deeper in the pavement structure. These additives are employed to improve moisture resistance and accelerate gaining strength in the early stages after compaction. CIR mixtures require more compaction energy in comparison with HMA due to higher viscosity, more inner friction angle between aggregates, and lower compaction temperature. After compaction, curing time varies up to two weeks depending on the environmental conditions (Godenzoni et al., 2016; Graziani et al., 2018).

Generally, selecting the proper cold recycling method depends on different factors including the type of pavement distress, quality of material (RAP), and expected service life of the project (Xiao et al., 2018). The advantages and limitations of cold recycling techniques are presented in Table 2.1.

Table 2.1 Characteristics comparison of cold recycling technology

Cold Recycling Methods	**Mixing Technology**	**Advantages**	**Limitations**
Cold In-place Recycling	In situ	Save the transport cost	Difficult construction quality control
		Solve moderate rutting and cracking	Curing is necessary
Cold In-plant Recycling	Central plant	Easy construction quality control	Curing is necessary
		Ideal grading can be achieved	Transportation cos is increased
		Repair non-structural distresses	

2.3 Cold recycled mixtures (CRM)

By the expansion of using cold recycled mixtures in road constructions, the majority of transportation agencies started to revise the guidelines and modify the regulations related to using RAP in asphalt mixtures (Kandhal & Mallick, 1998). However, more understanding of the behaviour of recycled mixtures was required to widely spread the application of cold recycled mixtures. Water plays a role as heat in HMA and provides workability for laying down and compaction of CR mixtures (Grilli et al., 2016). After compaction, stiffness is developed through time during the curing process. Therefore, the percentage of water in CR mixtures affects curing process and final performance (Graziani et al., 2018; Saadoon et al., 2018). Besides the presence of RAP in the body of CR mixtures, a proportion of virgin aggregates can be added to the mixture to replace RAP. Bitumen is used in CRM as binder to form cohesive and strength properties in mixtures. Since the production of CRM happens in ambient temperature, high viscosity of bitumen in this temperature decreases workability of mixtures. Therefore, bitumen is added to CRM either in form of emulsified bitumen or foamed bitumen to provide workability for the mixtures (Tebaldi et al., 2014). Considering a wide range of variables, CR mixtures can be produced with various parameters and their corresponding properties. Given the different proportion of components, CRM can be categorized in four groups as follow (Grilli et al., 2012):

- Cement-treated materials (CTM)
- Bitumen stabilized materials (BSM)
- Cement-bitumen-treated materials (CBTMs)
- Cold asphalt mixtures (CAMs)

Due to the complex composition of CRM that mostly consists of RAP, virgin aggregates, bituminous and cementitious stabilization agents, the performance of final mixtures is highly dependent on these components. Thus, it is vital to evaluate the properties of raw materials for the CR mixtures (Xiao et al., 2018).

2.3.1 RAP Material

Because CR mixtures mainly consist of RAP, the characteristics of RAP highly affect the mixture performance and service life of the projects. It is significant to evaluate the properties of RAP prior to the production of CRM. Reclaimed Asphalt Pavement (RAP) comes from milling and crushing existing pavements. Then, additives like rejuvenators are added to the RAP while mixing to make a homogeneous material. The most important parameters of RAP that highly affect the final mixture are moisture content, condition and content of bitumen, and gradation.

Since water content has a considerable influence on the final mixture, the optimum moisture content of RAP should be determined while calculating the amount of water to be added. The asphalt binder which exists in RAP might be active or not. This condition needs to be evaluated by rheological indicators like penetration value, viscosity, and softening point (Olsson et al., 1999). In case of having an active binder in RAP, the binder content will affect the adhesive properties. Based on the statistical results, binder content in RAP is expected to be 4% to 7.5% (Khay et al., 2014) . (Khay et al., 2014) worked on the effect of binder content on mechanical properties of CRM. RAP source is another important parameter which affects the CR mixtures. This parameter can be identified by PG grading of recovered asphalt binder. Moreover, the gradation of RAP can be varied based on the primary mix design of the existing pavement. However, gradation of RAP depends on in-place techniques like milling process. (Lee et al.) studied the effect of the RAP source on dynamic modulus of CR mixtures. They found that mixtures made from coarse RAP with low percentages of binder content are resilient to fatigue in low temperatures, while they are vulnerable to rutting at high temperatures. For example, (Cross, 2003) showed that CR mixtures containing finer RAP material and higher binder content have better rutting performance.

Apart from RAP, various materials are used to replace virgin aggregate in production of CRM. For instance (Behnood et al., 2015b) worked on copper slag and recycled concrete. (Niazi & Jalili, 2009) investigated the effect of cement and lime additives on CRM properties to find the optimum content and most effective additive. (Ameri & Behnood, 2012) used steel slag as a substitution for virgin aggregates. In addition to the mentioned research, there are more studies in this category. In most cases, the proposed additives improve the performance of mixtures, however, they impose some limitations for producing the mixtures in existence of new recycled materials.

2.3.2 Stabilization agents

The presence of stabilization agents aims to restore asphalt concrete properties. Using stabilization agents will provide flexibility, stability, and workability for the CR mixtures. With the development of CRM technology, various stabilization agents are being used. To choose proper stabilization agents, their cost and effectiveness as well as their environmental effects are being considered. (Bocci et al., 2010) studied the environmental influence of bitumen emulsion, cement, and soil with lime, then proved that CRM with the proposed additives have good environmental advantages. There are many variables involved in selecting proper stabilization agents. For example, price, availability, durability, and policy of transportation agencies are some of the criteria to choose suitable stabilization agents (Xiao et al., 2018). To this point, bituminous and cementitious stabilization agents are the most common material stabilization agents in the pavement industry.

2.3.2.1 Bituminous stabilization agents

Bituminous stabilization agents have been used along with the development of CR technology in past years. They are mostly categorized in two types: emulsified bitumen and foamed bitumen. Bitumen stabilized materials (BSM) are relatively flexible and more cohesive due to the presence of bitumen. In addition, shrinkage cracking is not an issue in these materials in comparison with cementitious stabilized materials. Both emulsified and foamed asphalt were subjects of many research in the last decades. (Cox & Howard, 2015) conducted laboratory tests to compare CIR process with foamed and emulsified bitumen. Based on (Cox & Howard, 2015) CIR-foam has less moisture content and higher tensile strength at the same curing time in comparison with CIR with emulsified bitumen.

The surface of bitumen droplets are charged in emulsified type. Therefore, bitumen particles tend to attach aggregate with opposite charges. Generally, materials which stabilized by emulsified bitumen are called BSM-emulsion (Xiao et al., 2018). Typically, the common bitumen emulsion type that is being used in CR stabilization are HFMS-2s (high-flow medium-setting with solvent), HFMS-2p (high-flow medium-setting with a polymer), CSS-1 (cationic slow setting), and CMS (cationic medium setting). HFMS-s2 and HFMS-2p are mainly similar, but the only difference is the modification of HFMS-2p with polymer. In case of having fine RAP, CSS-1 is usually selected to improve stability. CSS-1 requires to be mixed with wet RAP because the water within the bitumen emulsion tends to separate from asphalt particles in evaporation process. (Pouliot et al., 2003) worked on improvement of mortars with CSS-1 and cement, the results showed that CSS-1 had higher elastic modulus and strength in comparison with anionic emulsions. (Y. Kim & Lee, 2012) showed that the dynamic modulus, flow number, and flow time depends on the type of emulsion. (Yan et al., 2009) have done a series of laboratory tests with different emulsion content and determined the optimum content based on the results of Marshall stability, indirect tensile strength, and rutting tests. Due to the properties of CSS-1, this type of emulsion is used in this research.

2.3.2.2 Cementitious stabilization agents

Cementitious stabilization agents are mainly categories as Portland cement, lime (Hydrated or quicklime), fly ash, ground blast furnace slag, clay soil, etc. Cementitious agents are employed to increase bearing strength and compressive strength of final mixtures. The stabilized materials behave more brittle and less flexible when the amount of cementitious agent increases. However, it will raise the potential of shrinkage cracking in case of having excessive cementitious agents. Therefore, it is significant to optimize the amount of cementitious agent to meet the mix design requirement (H. F. Wang et al., 2011).

Cement is the most commonly used additive among other cementitious agents. it was the first stabilization agent used in the USA in 1917. That is the reason why cement has been a subject of much research so far. (Khay et al., 2014) evaluated mechanical characteristics of cement-treated RAP with up to 6% Portland cement. Laboratory experiments showed a noticeable improvement in high-temperature performance of CR mixtures with an increase in cement content. (Niazi & Jalili, 2009) investigated the effect of Portland cement, fly ash, and lime on the performance of CR mixtures. Their results proved that Portland cement is the best additive to reduce permanent deformation.

Portland cement is commonly used to improve strength in BSM with minor effects on the flexibility of mixtures. Moreover, to speed up the breaking process of bitumen emulsion and reduce construction time, cement can play an effective role in combining with bitumen emulsion. (Berthelot et al., 2007) studied the influence of cement and bitumen emulsion on the material characterization. They found that the combination of cement and bitumen emulsion can reduce the breaking time and help to shorten the construction process. Besides, (Ferjani et al., 2019) applied SCB results to evaluate the effect of different percentages of cement on cracking resistance of FDR. They concluded that cracking resistance of FDR increases by having more percentages of cement in the mix, however, it causes brittleness behaviour for FDR. Using moisture sensitivity results, they proved that brittleness of FDR materials increases in wet condition.

2.4 Mix design

Although using CR mixtures is expanding all over the world, there is not a specific guideline for the laboratory mix design procedure of CRM. Therefore, researchers and transportation agencies have tried to establish the most common applicable procedure for CR mixtures (*Asphalt Recycling & Reclaiming Association ARRA*, 2001). Since there are various parameters in CR mixtures application including aggregates, binder, additives, compaction, curing, and so forth, there are a large number of possibilities to change each variable and come up with specific performance. Nevertheless, this is out of the scope of this research to optimize the ingredients of CRM. World Road Association (PIARC) (2002) presented a report on Cold in-place recycling with bituminous emulsion and foamed bitumen. However, this report is not a specification of CIR application. Instead, it provided information about CIR applications in different countries.

Due to the performance of bitumen stabilized materials, BSM has the main role in CRM mix design. Based on the literature, researchers recommended various contents for emulsion and cement in CIR applications. (*Cold Recycling - Oregon 1985. Draft of Technical Report for Oregon State Highway Division*, 1986) suggested a range of 0.5% to 3% emulsion on the mass of RAP for CRM mix design, while this range can be varied for different places. For instance, in Pennsylvania it is ranged from 1.2% to 1.5%.

Besides using asphalt binder, the addition of virgin aggregate is another common application in production of CRM. There are various reasons for adding virgin aggregates including gradation correction, increasing thickness of the layer, and increasing stability of the pavement (Epps & Allen, 1990). Although the amount of added virgin aggregates depends on mix design criteria, for instance, (Wood et al., 1988) suggested adding 15 to 50 percent. (Murphy & Emery, 1996) showed that for CIR production, 20 to 25 % virgin aggregates decrease porosity and increase stability of mixtures. To identify the necessity of adding virgin aggregate, RAP properties need to be evaluated. In many cases, using new aggregates may not be required (Jahren et al., 2007; Yao et al., 2011).

Since there is no nationally standard method available for CRM mixture design, the process of establishing mix design is mostly based on Hveem stabilometer, Marshall stability test, air voids, resilient modulus, and other experimental results. In some cases, field trials and visual condition of samples are used to find the optimum content of ingredients for CRM (Apparao Gandi, 2018). Normally, the procedure for establishing a mixture design breaks down into the following steps:

- Obtaining RAP representative from either milling of old pavement or stockpiles,
- Evaluating RAP properties, including gradation, binder content, and binder condition,
- Selecting the amount of virgin aggregates to correct the gradation if necessary,
- Selecting the amount of stabilization agents to recover mixture strength,
- Establishing a proper procedure for mixture, compaction, curing, and water sensitivity,
- Finalizing adjustments based on field conditions.

Different agencies have established their own procedures for mixture design of CIR. However, the methods are very similar, the main differences are mostly in adding virgin aggregates, and curing process (Gandi et al., 2015). Generally, there are three basic theories have been considered in designing CIR mixtures. The first theory considers RAP as black rock, which means the process of CIR mixture design consists of determining recycling agents to coat the aggregates and make cohesive mixtures. The second assumption considers physical and chemical properties of old bitumen and uses recycling agents to restore existing asphalt binder to its original consistency. The third theory is a combination of the two methods. The assumption is that recycling agents and aged bitumen produce an effective asphalt layer. The third assumption is used for this research. The properties of existing bitumen, additives, and field conditions define the degree of softening. Since it is not easy to evaluate the degree of softening, mechanical tests are usually part of CIR mixture design process (Xiao et al., 2018).

2.5 Compaction

The main purpose of laboratory compaction is to simulate the same compaction that undergoes in the field. Compatibility and workability in CRM are different from HMA because of water and type of binder that exist in CRM. It is worth mentioning that there is no specific compaction method for CRM that is accepted all around the world (Flores et al., 2020). In previous research, various laboratory compaction methods were applied to manufacture CRM specimens. These methods include Marshall compaction (Behnood et al., 2015a; Brennen et al., 1983), gyratory compaction (Chen & Solaimanian, 2019; Ozer et al., 2016), Vibratory compaction (Hicks et al., 1979), and proctor compaction (Bleakley & Cosentino, 2013; Kukielka & Bakowski, 2019). Each compaction method has its specific features and guidelines. Selecting suitable compaction methods depends on the role of CRM in the pavement structure. There are many guidelines available for Marshall, vibratory, and proctor compaction. While the gyratory compaction has been subjected to investigate more. CRM with bitumen is mainly compacted using Marshall, gyratory, and vibratory compaction.

There are two important factors that are influential in compaction. The specimen size which differs from the actual sample in the field and air voids distribution in the sample are the parameters that matter in laboratory compaction. Compaction energy directly affects the density of the specimen (air voids) that relates to mechanical properties. Optimizing CRM components including water and bitumen can lead to better compaction results. (Grilli et al., 2012) studied the effect of liquid content in Cement-bitumen-treated materials (CBTM) on compaction. They used Shear Gyratory compactor and Proctor compaction to evaluate the volumetric properties of specimens with different liquid content. The results indicate linear and bilinear curves for samples with different water contents, but when the water content increases the bilinear part will expand. Figure 2.1 shows SGC results for a sample with 2% cement and 3% residual bitumen. VFL (Voids Filled with Liquids) will be explained in section 2.7

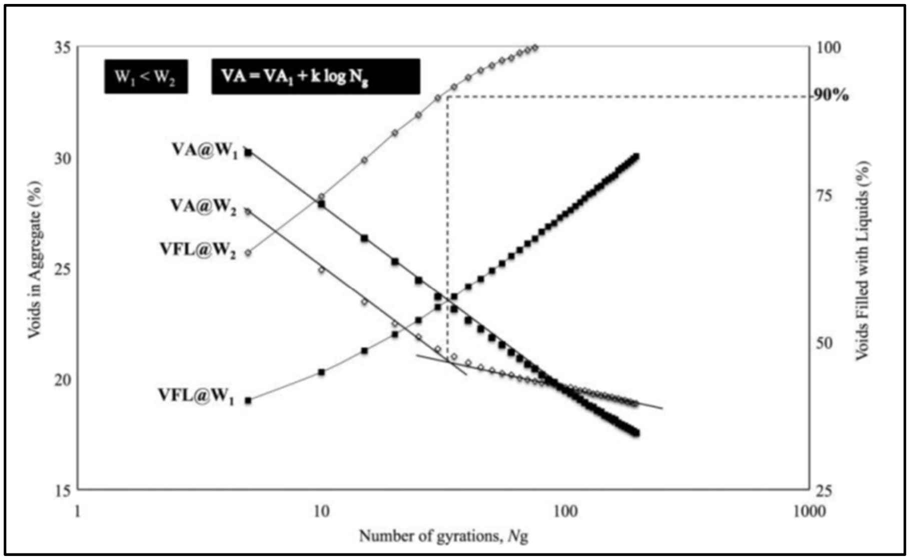

Figure 2. 1 SGC Test results from Grilli et al., (2012)

The bilinear part of the curves indicates the saturated condition in samples. Since then, compaction energy is dedicated to increase pore liquid pressure. In this situation, water starts to flow out from the bottom and top of the specimen.

2.6 Curing

Curing for the process of Cold in-place recycling (CIR) involves in propagation of strength which occurs through time. Inadequate curing can retain excessive moisture content which leads to increasing the possibility of stripping and decreasing the rate of gaining strength. Curing is mostly a function of environmental conditions. Thus, weather conditions, humidity, temperature, and the thickness of the layer can highly affect the curing process (Serfass et al., 2004). Due to the importance of curing, it is recommended that the application of HMA layover should be done after a large proportion of residual water is evaporated. In other words, initial curing of CIR is highly important during construction because it controls the proper time to overlay the upper layers and reopen the road to traffic. Although cementitious agents can accelerate curing process which is very important for reducing traffic interruption, water evaporation and building strength only happen over time for CRM. On one hand, cement hydration entails moisture and high rate of water evaporation can increase the risk of shrinkage cracking. On the other hand, the process of gaining strength happens by the evaporation of water over time. Therefore, it is necessary to consider moisture content for cement hydration during the process of curing. In case of simulating curing protocols for Cement-bitumen-treated materials (CBTM) in the laboratory, emulsion breaking, moisture loss, and cement hydration are the parameters that need to be considered. Moreover, the specimen shape and dimensions should be controlled (Cardone et al., 2015). Figure 2.2 shows the evolution of CRM over the curing process.

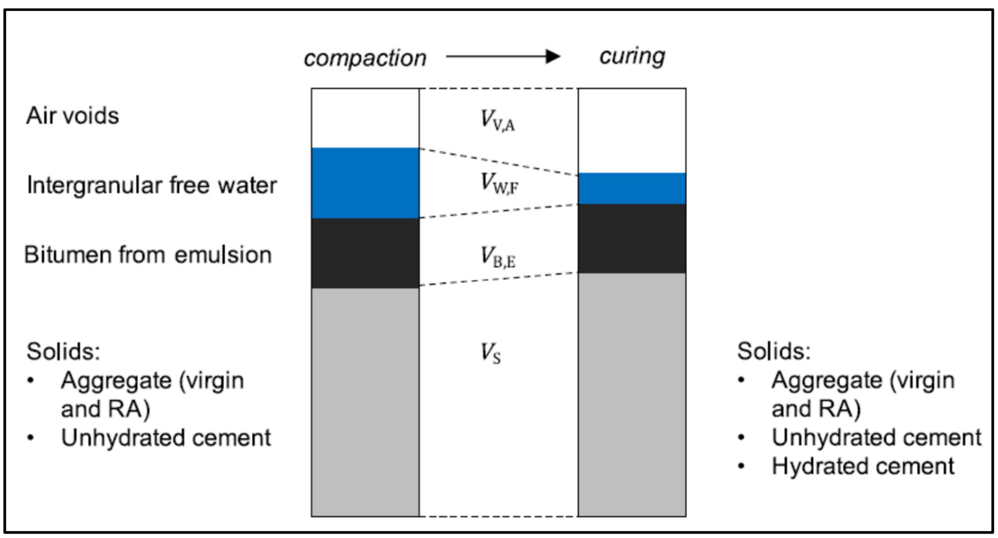

Figure 2. 2 Volumetric composition of CRM and evolution over curing from Graziani et al., (2018)

2.6.1 Field Curing

In many European countries, moisture content is used to indicate the proper time for HMA overlay. The residual moisture content of cold mix should be between 1% to 1.5%. While this percentage can be varied for the US from 1% to 2%. Generally, there is no difference in curing procedure for CIR mixture produced using emulsion or foamed bitumen. However, it is recommended for CIR-foamed to have less than 2% residual moisture content before a new layer is placed. For CIR-emulsion, the residual moisture is suggested to be 1% to 1.5% which is less than the corresponding amount in case of using foamed bitumen (Y. Kim et al., 2011). To simulate relatively the same condition as field curing, the temperature of 40°C was commonly used in literature. Also, curing duration can be varied from 12 to 36 months to represent different field curing conditions (Serfass et al., 2004).

2.6.2 Laboratory Curing Procedures of CRM

Temperature is the principal parameter of the curing process. Researchers have proposed different temperatures and durations for CRM curing. Based on the literature, the curing temperature varies up to 60°C. (Wegman & Sabouri, 2016) suggested curing at 60°C for 72 hours. While in other literature, curing at 40°C for 72 hours is proposed (Fu et al., 2009; Thompson et al., 2009). Curing temperature of 40°C is more commonly used in previous studies. (Sebaaly et al., 2004) studied the early stages stability of CIR-emulsion by using Hveem stability and resilient modulus. To achieve the early stages of curing, they applied three steps in the curing process. First, specimens were cured in molds for 15 hours at 25°C to represent initial curing. Then, they extruded specimens from the mold and cured at 60°C for three days for short-term curing. Finally, the specimens were kept at 60°C for 30 days to represent long-term curing. Since the purpose of this research is to study the mechanical properties of CRM after a process of short-term and long-term aging, the early stage of curing is not aimed to be achieved. Therefore, a curing temperature of 40°C as a common temperature is selected along with the duration of 14 days to make sure that the moisture content will reach the targeted amount of 1% to 1.5%. In addition to the common procedures in the literature for curing, there are some studies at ETS with the curing process of 40°C and 14 days. Thus, this is another reason to choose the same procedure in order to provide the possibility to compare the results (Baklouti, 2022; Raschia et al., 2019).

2.7 Aging

Oxidative aging is a main driving force that leads to spreading distress in AC mixtures. Aging will change rheological properties of asphalt binder and causes a significant reduction in serviceability of pavement. Short-term aging normally occurs in the construction phase, while the temperature of the AC mixture is high, and aggregates are coated with a thin layer of binder. Contrary, long-term aging takes place slower during the service life of the pavement. In this process, properties of asphalt binder change due to oxidation and radiation over time. Generally, aging causes more brittle materials with a high potential for cracking (Chen & Solaimanian, 2019; Y. R. Kim et al., 2018).

To simulate the process of aging in the laboratory, it is needed to accelerate this process by applying high temperatures to the specimens or materials. For short-term aging, it is common to place the loose mix in an oven with a high temperature for a period of two to four hours. On the other hand, long-term aging could be simulated in the laboratory by placing compacted specimens or loose mixtures in an oven for a longer period but at a lower temperature (Chen, 2019). Many studies have been done to investigate long-term or short-term aging and the effects associated with each condition. For instance, asphalt mixes containing crumb rubber modified are reported to have higher indirect tensile strength after long-term aging (Liang & Lee, 1996). Moreover, higher initial stiffness is expected for asphalt mixes due to long-term oven aging (LTOA) (Harvey & Tsai, 1997). (Raad et al.) investigate the effect of aging on fracture energy by comparing 10-year-old pavement and unaged specimens. They came up with the result that aging reduces fatigue properties of asphalt mixtures. However, it increases initial stiffness at intermediate temperatures (Raad et al., 2001). (Bonaquist et al.) employed SCB test and found a reduction in flexibility index (FI) of asphalt mixture after LTOA (Bonaquist et al., 2017). The majority of studies showed increased stiffness and reduced fracture resistance after laboratory or field aging.

A recent study by Newcomb shows that aging loose mix for HMA better represents the mixture stiffness between lab- and plant-produced specimens (Transportation Research Board &

National Academies of Sciences, Engineering, and Medicine, 2015). Following this statement, (Chen & Solaimanian, 2019) conducted the process of aging by placing loose mixtures at 150 °C for 2 hours before compaction to simulate short-term aging. For long-term aging, the process starts with conditioning the loose mixture at 150 °C for 2 hours then followed by 120 hours at 85°C. Finally, long-term aging specimens were placed at 150 °C for 2 hours before compaction. However, 60 °C as the temperature for aging is not common in previous studies, this temperature is selected along with longer duration of oven aging to offset the lower temperature.

2.8 Volumetric Properties

The main difference between CRM and HMA in terms of volumetric properties is the presence of water in CRM. Water in cold mixes plays the same role as heat in HMA. It provides workability and compatibility for the mixture. In addition, the process of curing makes another difference from HMA. These two parameters make a specific methodology to define volumetric properties of CRM. Since volumetric properties of CR mixtures have a direct impact on performance of the final pavement, different projects have specific requirements in terms of performance and volumetric features of final mixtures. These outlines are considered in the construction process and quality control and quality assurance phase (Grilli et al., 2012, 2016).

Given that numerous combinations of components could be considered to produce CBTM with variable characteristics, the precise definition of CBTM volumetric properties is acquired by analyzing mixtures in the fresh state and right after curing (Grilli et al., 2012).

Water as a principal component of CBTM could exist either as absorbed water by aggregates or as free water. The existing liquid in the mixture includes free water content (W_F), and residual bitumen content (B_R), which adds up to the free liquid content (L_F). These parameters can be obtained using equations 2.1 to 2.3.

$$W_F = \frac{M_{W,F}}{M_{B,R} + M_S} \times 100 \tag{2.1}$$

$$B_R = \frac{M_{B,R}}{M_{B,R} + M_S} \times 100 \tag{2.2}$$

$$L_F = \frac{M_{B,R} + M_{W,F}}{M_{B,R} + M_S} \times 100 \tag{2.3}$$

Where;

$M_{W,F}$ is the free water consisting of added water and water in emulsion,

$M_{B,R}$ is the mass of residual bitumen,

M_S is the total mass of solid phase including RAP, cement, and filler.

Due to the presence of water in CRM the calculation of air voids (AV) is done according to the following equation (2.4).

$$VA = \frac{V_V + V_{W,F} + V_{B,R}}{V_T} \times 100 \tag{2.4}$$

Where V_V is the volume of air voids, $V_{W,F}$ is the volume of free water that is not absorbed by the aggregates. $V_{B,R}$ is the volume of residual bitumen and V_T is the total volume of mixture. Besides the definition of AV, there is another significant parameter to characterize volumetric properties of mixture before the emulsion breaking. This parameter defines as the volume of voids filled with liquid (VFL). This parameter can be calculated by equation 2.5.

$$VFL = \frac{V_{W,F} + V_{B,R}}{V_V + V_{W,F} + V_{B,R}} \times 100 \tag{2.5}$$

Figure 2.3 illustrates the structure of CRM and presents a graphical perspective from the volumetric details of CBTM.

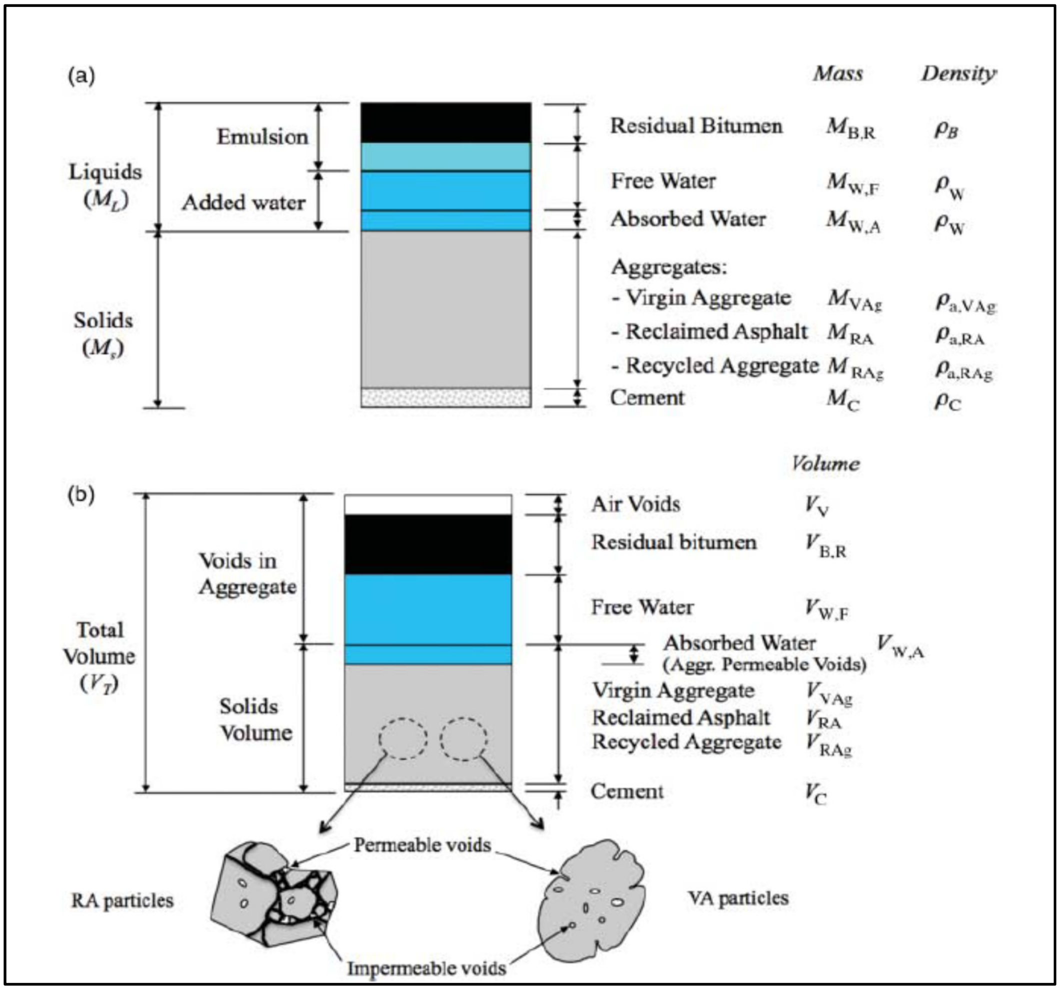

Figure 2. 3 Constituent materials and volumetric characteristics of CBTMs:
(a) constituents by mass and (b) constituents by volume from Grilli et al., (2012)

2.9 Fracture Characterization of Asphalt Pavement

Cracks in asphalt concrete (AC) are one of the main pavement deteriorations. This failure can be categorized in two groups. Cracks that occur due to traffic load applications, including reflective cracking, bottom-up, longitudinal wheel-path top-down fatigue cracking, and near-surface cracking. These cracks can be propagated regarding material properties and the pavement structure. Another group of cracks is non-load related cracks in which the initiation and propagation of cracks are mainly associated with environmental effects. Block cracking and thermal cracking are considered to be in this category. In non-load associated cracks, imitation and development of cracks occur regardless of the type of applied loads and traffic. Contrary, they happen because of changing stress in the materials (Ozer et al., 2018). Material characteristics, pavement structure, and traffic loading all influence crack development. Besides, modulus and fracture or damage resistance properties are two key material properties that mostly affect crack propagation. To estimate the progress of crack initiation and propagation, linear and non-linear fracture mechanics are presented in section 2.9.1. Due to the characteristics of asphalt mixes, non-linear fracture mechanics has a better capability to estimate crack development in comparison with linear fracture mechanics. Energy release rate and fracture energy are the two parameters involved in this approach (Anderson, 2017; Z. P. Bažant & Planas, 2019). The energy release rate indicates stored energy at the crack fronts that can be assumed as a crack driving force. The available energy for crack to be grown is considered as modulus properties, along with the loading and structure. On the contrary, fracture energy is a function of material characteristics which can be affected by the size of a structure (Z. Bažant & Planas, 1997; Soltani & Anderson, 2005).

Fracture phenomenon is the occurrence and propagation of cracks in asphalt pavement. Fracture mechanics is an approach to investigate crack resistance in asphalt mixtures. By using this method, the development of cracks is being studied. Fracture mechanics is categorized as an approach to characterize fracture behaviour of asphalt mixtures. Beside fracture mechanics, there are more approaches to address crack resistance in the laboratory, including the traditional phenomenological approach, viscoelastic continuum damage approach, and dissipated energy-based approach. In traditional phenomenological approach, the number of load repetitions until failure correlates to the stress and strain responses in the mixture. The viscoelastic continuum damage provides the principles of fatigue performance with a noticeable potential to characterize asphalt pavement (Youngsoo Richard, 1988). The dissipated energy-based approach considers the fatigue life of asphalt mixtures as a function of the dissipated energy of each loading cycle. The fracture mechanic concentrates on initiation and propagation of cracks. Many parameters are derived from this concept to characterize crack resistance of asphalt mixtures. These parameters have been developed over time. Stress intensity, fracture energy, flexibility index, crack resistance index, and so forth are the mentioned parameters in fracture mechanics approach (Chen, 2019).

2.9.1 Fracture test methods

By using fracture mechanics approach, failure is investigated by monitoring crack initiation and propagation. Many laboratory tests have been developed to study fracture resistance and susceptibility of asphalt mixtures to cracking, including thermal and intermediate-temperature fatigue cracking (Al-Qadi et al., 2012; Ozer et al., 2016), disc-shaped compacted tension, semi-circular bending (SCB) tests, beam fatigue test, push-pull test, indirect tension test (IDT), and Texas overlay are among the mentioned fracture tests (Li et al., 2010; Li & Marasteanu, 2004; Underwood et al., 2012).

2.9.1.1 Semi-Circular Bending (SCB) Test

Use of SCB test goes back to 1984 when it was employed to test fracture resistance of rock (Chong & Kuruppu, 1984). A semi-circular shaped specimen with a notch is loaded under a Three-point cyclic or static force as shown in Figure 2.4. This setup produces tension at the bottom of specimen that leads to crack initiation. SCB test has been proposed in many studies due to its simplicity of specimen production and consistency of results.

To address fracture properties of AC mixtures using SCB test, the combination of linear elastic fracture mechanics (LEFM) and elastic plastic fracture mechanics (EPFM) should be used. There are different parameters in definition of each method. Critical energy release rate J_c and integral are used in EPFM analysis. While, stress intensity factor (SIF) K, fracture toughness K_{IC}, and fracture energy G_{IC} are defined in LEFM method (Saha & Biligiri, 2016). Using the acquired data from SCB test and based on LEFM approach, the fracture energy represents the energy required for a crack to propagate. This parameter is derived by considering the area under the load-displacement curve. Contrary, critical energy rate J_c and J integral are considered as critical fracture energy which is calculated as the area under the load-displacement curve until peak load then plotted against the notch depth (Chen, 2019; Saha & Biligiri, 2016).

In general, LEFM remains valid for brittle materials. However, for quasi-brittle materials, EPFM is applicable by considering a larger fracture process zone (FPZ) at the crack tip. In previous studies, one or both methods were used to evaluate fracture properties of asphalt mixtures based on SCB test results (Saha & Biligiri, 2016). (Hofman et al., 2003) claimed that for asphalt mixtures with high binder content, fracture toughness K_{IC} is not applicable due to the viscoelastic properties of mixtures. Instead, J integral which is defined as critical fracture energy can be utilized as a part of the EPFM approach. According to the research studies, J_c showed higher sensitivity in comparison with peak load, strain energy, and nominal maximum aggregate size (NMAS). Therefore, J_c is a reliable index to describe the field cracking behaviour (Saha & Biligiri, 2016).

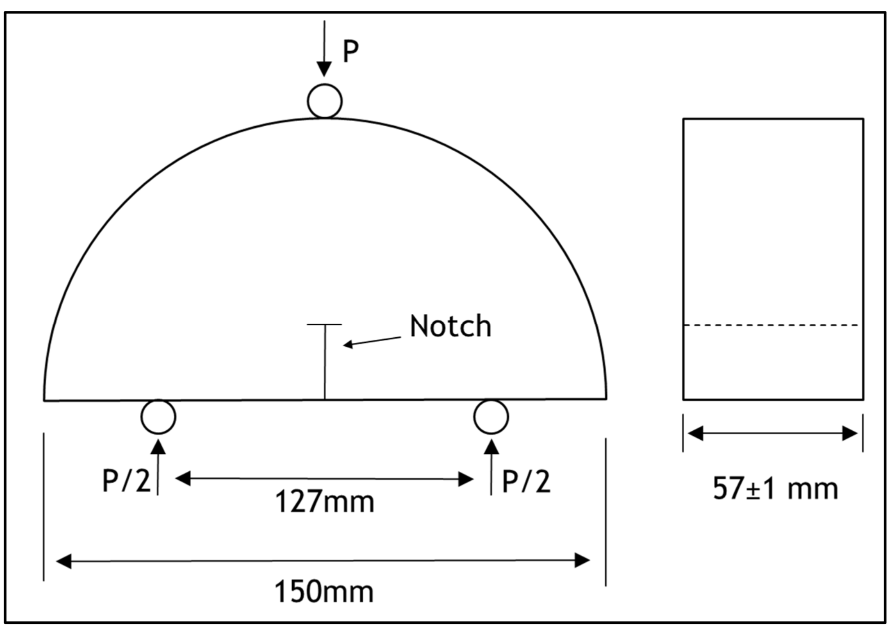

Figure 2. 4 Schematic of SCB test configuration

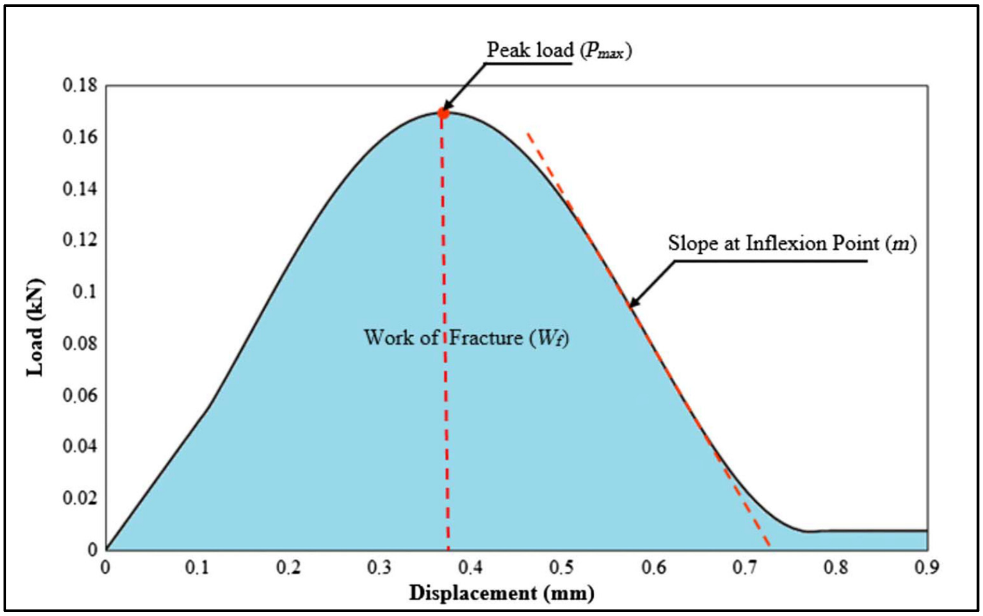

Figure 2. 5 Load displacement curve derived from SCB test
taken from Ferjani et al., 2019

2.9.1.2 Indirect tensile strength test (IDT)

The indirect tensile test is developed by Carneiro and Barcello in 1953. Later, this test method was applied to asphalt pavement materials (Anagnos & Kennedy, 1972; Hadley et al., 1969; Hudson & Kennedy, 1968). The test requires cylindrical specimens that are placed along two opposite generators to apply compressive loads. this application makes uniform tensile stress perpendicular to the applied load. When the specimen overpasses the strength, the failure occurs by splitting along the dimension of the applied load (Hudson & Kennedy, 1968).

The test is conducted with a loading rate of 50mm/min at 25 °C. Also, the specimen diameter is considered 100mm in most cases and 150mm when field cores are being tested (Xiao et al., 2007a). IDT test can be employed to evaluate tensile properties along with cracking potential of AC mixtures. Moreover, IDT test (also called ITS) can be used to characterize moisture susceptibility of asphalt mixtures (Khay et al., 2014; Niazi & Jalili, 2009). Figure 2.6 illustrates a diagram of IDT setup.

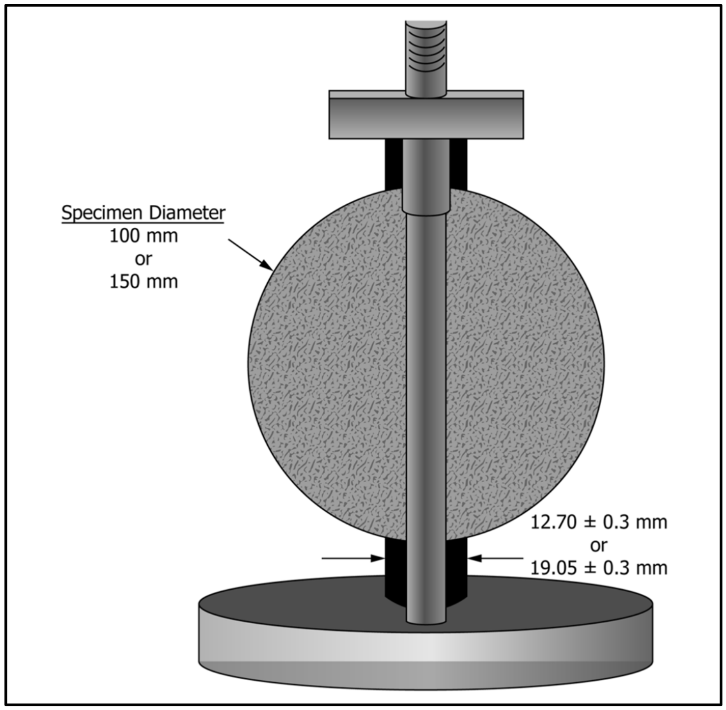

Figure 2. 6 Diagram of IDT Strength-Loading Fixture
from ASTM D6931 – 17

2.10 Summary

Cracking in asphalt pavement is a widespread issue that is important to be investigated and controlled to avoid the deterioration of asphalt materials and ensure a safe ride for the traffic. There are four approaches to address this distress; dissipated energy approach, fracture mechanics, and viscoelastic continuum damage approach are complicated methods in comparison with phenomenological approach. However, they better represent the fundamentals of asphalt mixtures. To investigate cracking resistance of asphalt mixtures, numerous test methods have been used in the literature. Considering different procedures for each test, some of which have more complicated approaches in specimen preparation and interpretation of the results. Moreover, some of the tests represent specimen performance instead of material behaviour. Overall, fracture mechanics was chosen over the other approaches.

Given the fact that using recycled materials in asphalt mixture will result in a stiffer and more brittle mixture, crack resistance in CRM is critical and needs to be controlled in different conditions. Therefore, there is an interest in employing test methods with a potential to be implemented in routine mix design with the advantages of having simple and low-cost procedures along with reliable results. Such methodology leads to developing quality of construction materials, durability, and service life of the pavement.

To investigate crack resistance of CRM, the effect of different binder content, temperature and aging are considered to be investigated using Semi-Circular Bending (SCB) test and Indirect Tensile Strength (IDT) test. Indirect Tensile Strength (IDT) and Semi-Circular Bending (SCB) are chosen to evaluate the crack initiation and the crack propagation, respectively. It is worth mentioning that having a performance database of CRM will help to better develop using recycled materials and improve the performance of asphalt mixtures for lower costs.

CHAPTER 3

MATERIALS AND METHODOLOGY

This chapter provides details about the materials and all components of the experimental phase involved in this research. The details about the materials and characteristics are presented along with the standards and procurers to manufacture the specimens. Given the large number of experiments and samples within the research, this chapter aims to clarify the procedures and draw a framework for this study.

3.1 Experimental Plan

The materials used in this study consist of a single source RAP, cationic slow setting bitumen emulsion (CSS-1), filler, cement, and water. The specimens are manufactured using Superpave Gyratory Compactor (SGC) in different dimensions. Then Semi-Circular Bending (SCB) tests and Indirect Tensile Strength (ITS) tests are employed to characterize the specimens.

Three different mix designs are considered to be investigated. Mix A contains 1% residual bitumen and Mix B has 2.2% residual bitumen, are the main two mix designs. Beside them, Mix C containing of 100% RAP without any additives (no added bitumen) is manufactured to focus on RAP performance over time. Considering the capacity of the mixer and the size of specimens, the weight of mixtures was selected in order to manufacture homogeneous specimens while trying to minimize water evaporation during the production phase. To better illustrate the framework of this research, Figure 3.1 presents the structure of experiments.

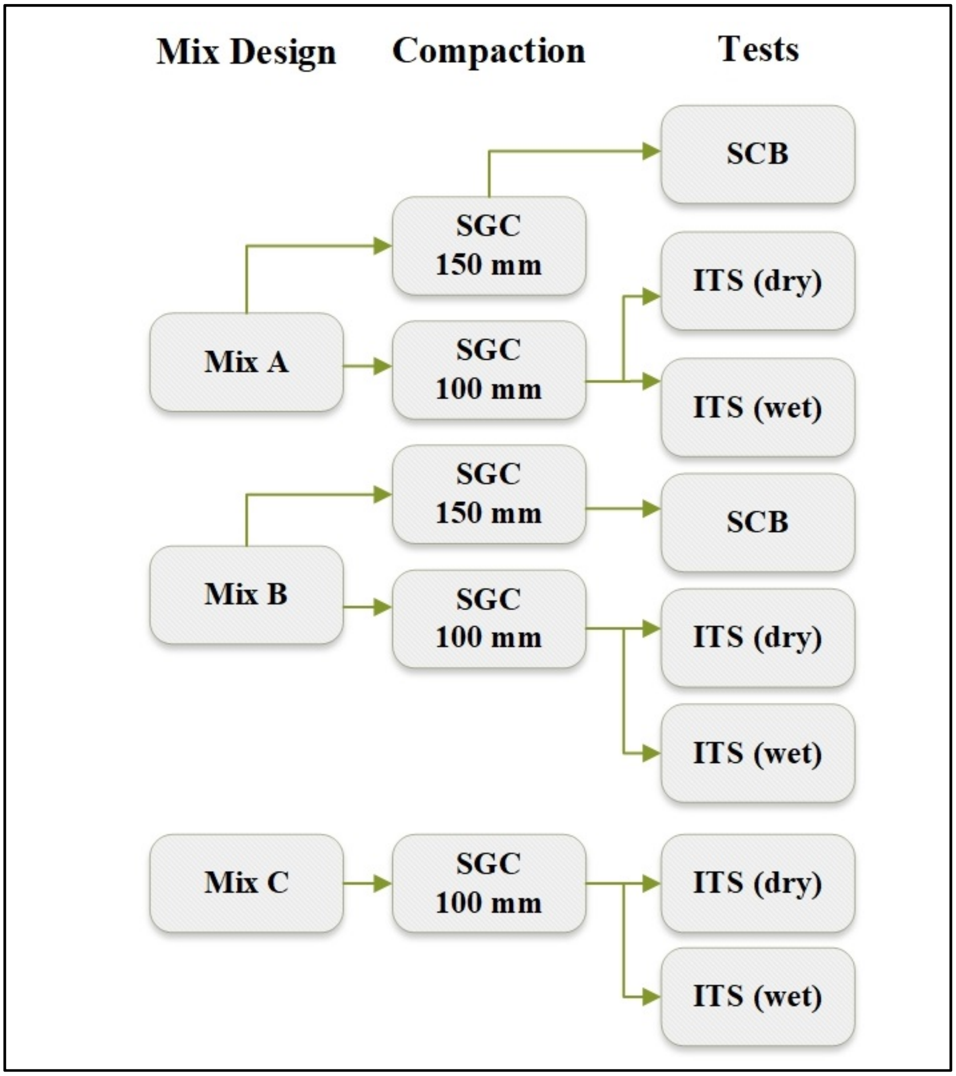

Figure 3. 1 Structure of experiments

Table 3.1 presents the laboratory experimental plan to conduct the tests. It contains the details for different mix designs, diameters of specimens, number of representatives for each test, and specific conditions applied to the tests. This research consists of 216 SCB tests for two mix designs, three aging conditions and three different temperatures. Moreover, there are 54 ITS tests have been conducted through this study for three different mixes in three aging conditions.

Table 3. 1 Laboratory experimental plan

Mix Design	Compaction Method	Tests	Aging[1]	Temperature (°C)	Total Number of specimens
Mix A (1% residual Bitumen)	SGC 150 mm	SCB (3 notches, 4 rep.)	STOA, LTOA, XLTOA	-20, 0, 19	4×3×3×3=108
	SGC 100 mm	ITS (dry) 3 rep. & ITS (wet) 3 rep.	STOA, LTOA, XLTOA	25	3×2×3=18
Mix B (2.2% residual Bitumen)	SGC 150 mm	SCB (3 notches, 4 rep.)	STOA, LTOA, XLTOA	-20, 0, 19	4×3×3×3=108
	SGC 100 mm	ITS (dry) 3 rep. & ITS (wet) 3 rep.	STOA, LTOA, XLTOA	25	3×2×3=18
Mix C (100% RAP no additives)	SGC 100 mm	ITS (dry) 3 rep. & ITS (wet) 3 rep.	STOA, LTOA, XLTOA	25	3×2×3=18

[1] STOA: short-term oven aging; LTOA: long-term oven aging; XLTOA: extra long-term oven aging

3.2 Materials

The materials investigated in the research are included typical asphalt paving materials and recycled materials used in the province of Quebec. To characterize the properties of the materials, Quebec standards (LC), as well as ASTM standards are employed in this study.

3.2.1 Reclaimed Asphalt Pavement (RAP)

Reclaimed Asphalt Pavement (RAP) is the main component of the mixtures, and it was obtained from Construction DJL inc site in Quebec (Figure 3.2). RAPs were stored in about 55 buckets of around 30 kg. To make sure that have a homogeneous batch of RAP, all available materials were mixed until reached homogenous batches. Then separated into smaller portions.

Figure 3. 2 Construction DJL inc - Quebec

Gradation of RAP is defined according to LC 21-040 (*Analyse granulométrique*) standard. Besides, the percentage of bitumen content in the RAP is calculated based on LC 26-006 (*Détermination de la teneur en bitume par ignition*). Water absorption of the RAP is characterized using LC 21-065 (*Détermination de la densité et de l'absorption du granulat fin*), LC 21-066 (*Détermination de la densité et de l'absorption du granulat fin de classe granulaire d/D*), and LC 21-067 (*Détermination de la densité et de l'absorption du GROS granulat*). General characteristics properties of the RAP are presented in Table 3.2.

Table 3. 2 Main properties of RAP

Property	**Standard**	**Value**
Binder content (by mass of aggregate)	LC 26-006	5.30%
Nominal maximum particle dimension	LC 21-010	14 mm
Maximum specific gravity: G_{mm}	LC 26-045	2.563
Water absorption (particles passing the 2.5 mm sieve)	LC 21-065	1.91%
Water absorption (fraction 2.5/5 mm)	LC 21-066	1.56%
Water absorption (particles retained on 5 mm sieve)	LC 21-067	0.99%

According to gradation analysis, Figure 3.2 illustrates the gradation curve based on LC 21-040.

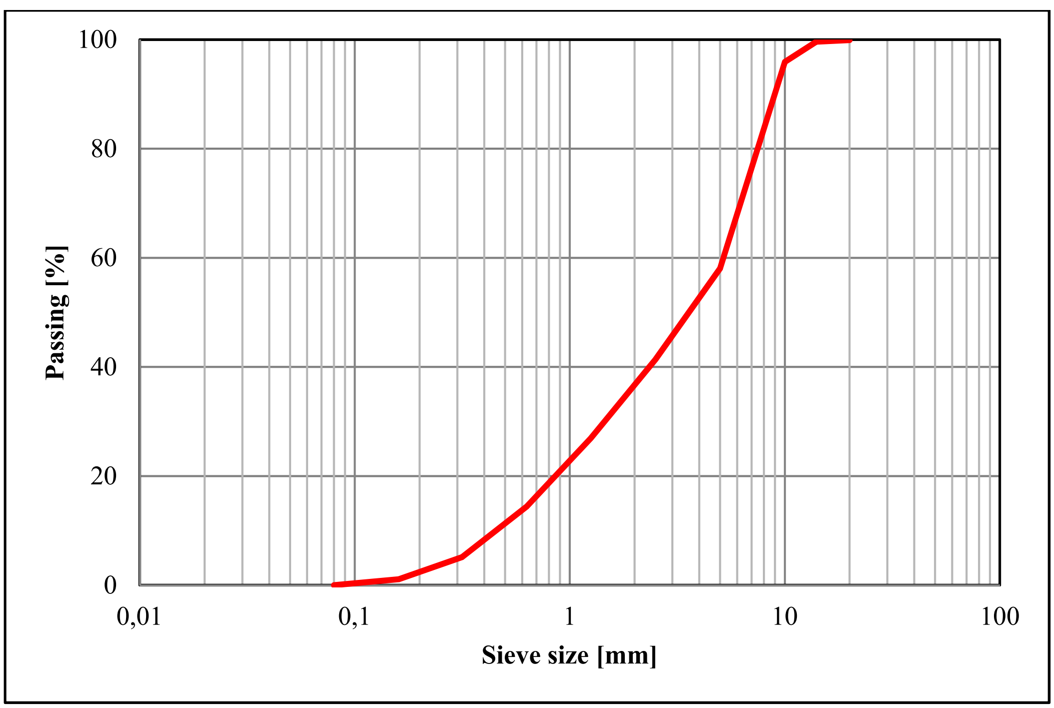

Figure 3. 3 RAP gradation curve

3.2.2 Filler

A mineral filler (virgin aggregate) characterized by 93% of passing material at 0.08 mm sieve size was selected to be added to the mixes. The presence of filler adjusts the RAP gradation to approach the maximum density curve.

3.2.3 Binder (Bitumen Emulsion)

Binder in form of bitumen emulsion was used for manufacturing CTBM. The type of emulsion used in this research is Cationic Slow Setting (CSS-1) with positive chemical charge which contains 63% residual bitumen. The grade of bitumen is PG 58-28. Selection of CSS-1 emulsion is based on the literature and previous research at ETS. The emulsified bitumen is provided from McAsphalt.

3.2.4 Cement

The type of cement employed in the mixtures is the general use (GU) which has compressive strength at 28 days of 43.9 MPa according to ASTM (C109). This type of cement has Blaine surface area of 410 cm^2/g.

3.3 Specimen Production

Two different mix designs are produced and investigated in this research. The only variable in the two mix design is the percentage of binder adjusted with the amount of added bitumen (residual bitumen from the emulsion). The reason to have different bitumen contents in the mixtures is to evaluate the effect of fresh bitumen on the performance of CRM. Therefore, other components are kept constant in both mixtures. Beside the main two mixes, the third mix design is manufactured with 100% RAP with no additives. This mix is defined in order to evaluate the hypothesis that RAP can be considered as "black rock". In other words, the aged bitumen in the RAP is assumed to be a part of aggregates. It does not play the role of binding phase to change the properties of RAP (Graziani et al., 2018). Thus, the third mix design aims to investigate the effect of old bitumen within the RAP on the performance of mixtures through the aging process. Table 3.3 presents the components of mix designs.

Table 3. 3 Mix design components

Mix components	**Mix A**	**Mix B**	**Mix C**
	Percentage according to dry RAP weight		
Water (including water in emulsion)	3.58%	3.58%	No additives 100% RAP
Residual bitumen	1.0%	2.2%	
Cement	0.5%	0.5%	
Filler	5.0%	5.0%	
Maximum specific gravity, G_{mm}	2.531	2.481	2.563

3.3.1 Mixing

The production of mixtures is divided into dry and wet phases. In dry phase, all dry ingredients of the mixture including dried RAP, filler, and cement are mixed until having a homogeneous material. The duration of mixing is 90 seconds until reaching a homogeneous mixture. The mixer that is used for this purpose has the capacity to mix approximately up to 15Kg of materials. Therefore, the total weight of specimens is less than 15kg in each production. In addition, limited proportion of material helps to keep the constant properties for each specimen by minimizing the time of production. After mixing the dry components, water is added to the mixture and mixed for another 90 seconds. When the added water is mixed and absorbed by the aggregates, bitumen emulsion is added to the mixture as the final ingredient and mixed until reaching a homogeneous material. The process of mixture preparation is conducted at room temperature and all of the materials are kept at room temperature before mixing. Figure 3.3 illustrates the procedure for specimens production in the laboratory.

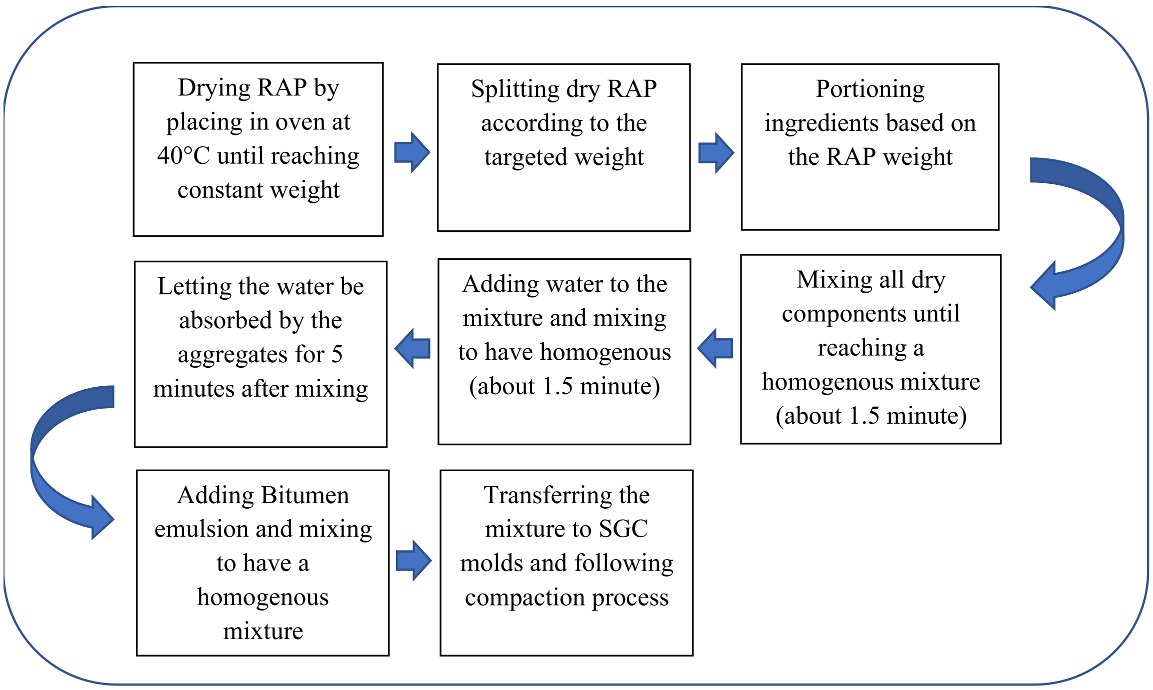

Figure 3. 4 Specimen production steps in the laboratory

To optimize the emulsion characteristic, the emulsion is stirred before adding it to the materials. In addition, limited amount of material is considered to produce specimens to minimize the potential water evaporation during the compaction.

3.3.2 Compaction

Superpave Gyratory Compaction (SGC) is selected as the compaction method since it has the potential to simulate compaction of pavement layers under a roller and traffic loading of pavements. SGC is configured with a 1.25° angle of gyration and 600 kPa compressive pressure (Figure 3.5). One of the most important features of SGC is the possibility to control the specimen height and reach the targeted air voids consequently. The targeted air voids (11%) is achieved by using the maximum specific gravity of the mixtures and controlling the height of the specimens. Since the molds diameters are either 100mm or 150mm, weight and height of specimens are selected to meet the targeted air voids. To address this point, compaction details of specimens are presented in Table 3.4.

Figure 3. 5 Superpave gyratory compactor (SGC)

Table 3. 4 SGC specimens details

Mix design	**Mold diameter**	**Height of specimens**	**Target air voids**	**Weight of the mixture**	**Number of gyrations**
Mix A	100mm	63.5mm	11% (+/- 0.5)	1159g	57-91
	150mm	120mm	11% (+/- 0.5)	4928g	76-102
Mix B	100mm	63.5mm	11% (+/- 0.5)	1135g	48-84
	150mm	120mm	11% (+/- 0.5)	4828g	64-72
Mix C	100mm	60mm	11% (+/- 0.5)	1076g	94-103

After having a homogenous mixture, loose materials are split by using the splitter apparatus regarding the proposed number of specimens to provide constant gradation and ingredient percentage for each specimen. Then, loose mixtures are scaled to reach the target weight and placed in the molds. In this research 150mm and 100mm cylindrical molds are employed.

3.3.3 Curing

Curing as a significant part of specimen preparation in the laboratory is conducted to imitate the field conditions. According to the previous studies and the common procedures for curing, the duration and temperature of curing process were selected to be 14 days at 40 °C. In addition, the presence of cement and water in the specimens was considered in determining curing duration. Therefore, water evaporation, emulsion breaking, and cement hydration are the main features that take place during curing process. The specimens are placed in a way that water could leave from the top surface and perimeter of specimens (Figure 3.6). Water evaporation is recorded in the duration of curing to observe the rate of water loss in both 100mm and 150mm specimens. According to the results of water loss, the percentage of residual water reaches a relatively constant amount within 14 days as shown in Figure 3.7 (less than 1%). Therefore, the curing duration of 14 days at 40 °C is considered to be sufficient for the specimens to reach a constant weight.

Figure 3. 6 Specimen (100mm)

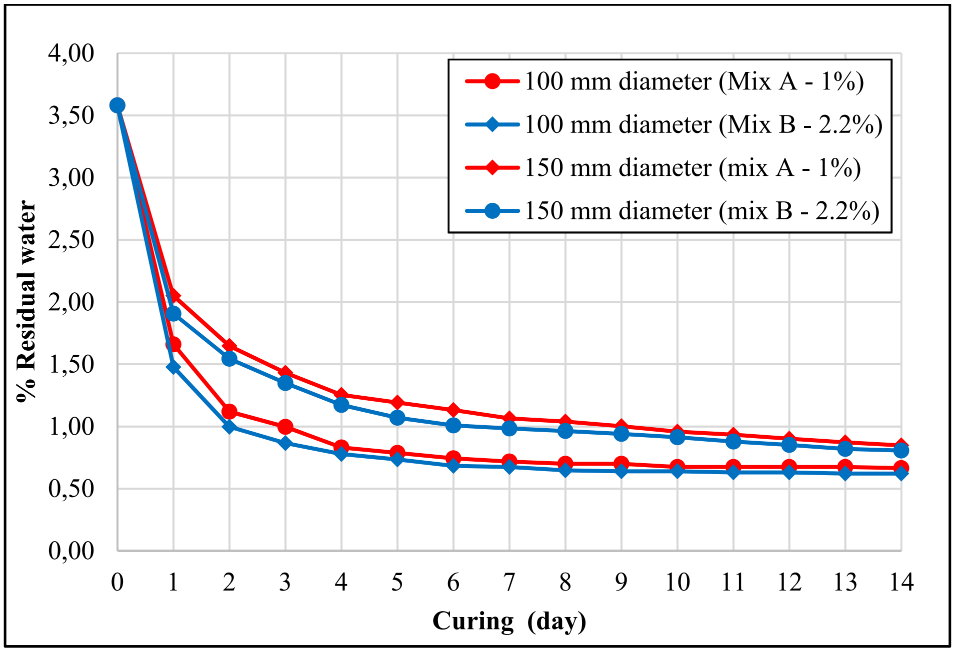

Figure 3. 7 Example of water loss during 14 days of curing (Mix a and B)

Residual water in representative specimens is calculated according to equation (3.1). Each sample is weighted daily to observe the amount of evaporated water.

$$\Delta W^{(t)} = \frac{W^0 - W^{(t)}}{W_{TOT}} \times 100 \tag{3.1}$$

Where;

$\Delta W^{(t)}$ is the percentage of residual water after a certain time,

W^0 is the weight of specimens right after compaction,

$W^{(t)}$ is the weight of specimens after a certain time,

W_{TOT} is the weight of added water for the production of specimens.

Since measuring the exact amount of water loss is not possible due to the particle loss during specimen production, the percentage of residual water at day 0 is assumed to be 3.58% for all samples.

3.3.4 Aging

One of the main objectives of this study is to evaluate the failure mechanism of CRM in different stages of aging. Therefore, a laboratory aging process was established to simulate field aging. According to the literature, a common approach to accelerate aging is to place the loose mixture at a high temperature for a limited time to let the bitumen be aged before compaction (Chen & Solaimanian, 2019). However, this approach is not applicable to the mixtures in this research because CRM needs to be manufactured at ambient temperature. Thus, another protocol is considered for aging which can be applied to the specimens after compaction and 14 days of curing. For this purpose, two main stages of aging including short-term and long-term are considered in this study. Both aging conditions are performed at the same temperature (60 °C) but in different durations. In short-term oven aging (STOA), specimens are placed in an oven at 60 °C for the duration of 4 hours. While long-term oven aging (LTOA) is performed by placing specimens in the same condition for 48 hours.

To better understand and clarify the effect of aging protocols, the third stage of aging was assumed by increasing the duration of aging process. Therefore, a longer period of oven aging is defined as extra long-term oven aging (XLTOA) which consists of placing specimens at 60 °C for 7 days. Specimens with 150mm diameters made for SCB test are cut and then placed in the oven for aging to increase the open surface of specimens and maximize the effect of oven aging. It is important to note that the three different levels of aging are not related to a given age or duration in the field since the field aging change significantly depending on the weather and the pavement structure.

CHAPTER 4

TEST AND RESULTS

This chapter is dedicated to the test details and results. In addition to the literature review, more information about each test is provided in this chapter to explain all procedures for the tests. There are 216 SCB and 54 ITS tests were conducted in this study.

4.1 Indirect tensile strength test (IDT)

There are two common standards available to perform indirect tensile strength test (IDT) also known as ITS. This test can be conducted according to ASTM D6931-17 and EN 12697-23. In this research, ASTM D6931-17 was selected over the European standard to conduct indirect tensile strength test. Since water sensitivity is another parameter to be investigated, saturating procedure of specimens is done based on LC standard. Then, the tests in dry and wet conditions are done according to ASTM D6931-17. The specimens are prepared using 100mm diameter SGC. Then, they are cured for 14 days at 40 °C in a draft oven to gain strength and evaporate the water. After curing, three stages of aging are simulated by defining different duration of oven aging to have short-term and long-term aging conditions. In general, three different mix designs in three aging stages are prepared for the test in dry and wet conditions. According to the standard, each sample must have at least three representatives. Therefore, 54 specimens are tested based on ASTM D6931-17. Figure 4.1 presents the specimens and test procedure.

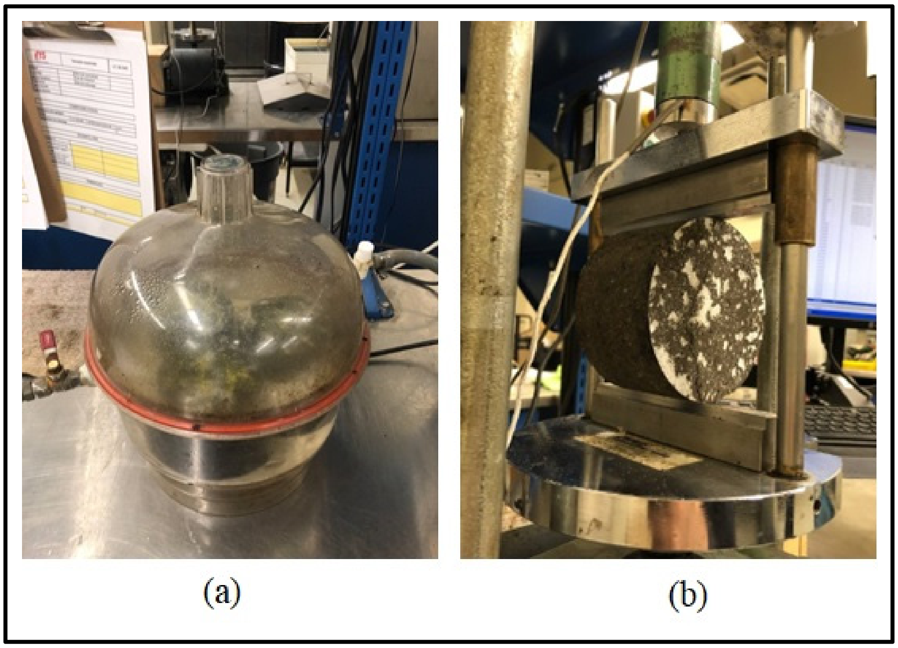

Figure 4. 1 Indirect tensile strength test procedure
a) saturation process b) IDT setup

The indirect tensile strength (S_t) is calculated according to equation 4.1, and the tensile strength ratio (TSR) is acquired by equation 4.2.

$$S_t = \frac{2000 \times P}{\pi \times t \times D} \tag{4.1}$$

Where;

S_t is IDT strength (kPa),

P is the maximum load (N),

t is the specimen height immediately before the test (mm),

D is the specimen diameter (mm).

$$ITR = \frac{ITS_{wet}}{ITS_{dry}} \tag{4.2}$$

Where;

ITS_{wet} is IDT strength wet (kPa),

ITS_{dry} is IDT strength dry (kPa),.

The specimens are manufactured using SGC by controlling height to reach the targeted air voids. Thus, the weight of each specimen is determined according to the maximum specific gravity of the mix designs, the targeted air voids (11%), and height of specimens. The results of indirect tensile strength test are presented in Table 4.1.

Table 4. 1 Results of indirect tensile strength test

Mix design	Aging	ITS_{dry} (kPa)	ITS_{dry} (average)	ITS_{wet} (kPa)	ITS_{wet} (average)	ITR
A	STOA	659.3	634.6	478.5	471.5	0.74
		594.6		453.5		
		649.8		482.4		
	LTOA	776.2	774.3	544.1	550.4	0.71
		764.7		552.7		
		781.9		554.4		
	XLTOA	742.1	812.4	686.1	683.1	0.84
		825.4		650.8		
		869.8		712.3		
B	STOA	650.5	623.7	590.4	571.7	0.92
		616.1		544.1		
		604.5		580.5		
	LTOA	778.72	789.6	628	641.7	0.81
		782.2		655.4		
		808		641.6		
	XLTOA	852.1	936.8	803.16	774.4	0.83
		1021		744.1		
		937.2		776		
C	STOA	292.8	272.1	142.8	122.1	0.45
		274.2		124.2		
		249.4		99.4		
	LTOA	286.6	319.8	136.6	169.8	0.53
		331.3		181.3		
		341.5		191.5		
	XLTOA	284.6	301.7	134.6	151.7	0.50
		302.7		152.7		
		317.9		167.9		

4.2 Semi-circular bending (SCB) test

Semi-circular bending (SCB) tests are conducted according to ASTM D8044-16. Thus, specimens are prepared with notches of 25mm, 32mm, and 38mm. According to the standard, each test should be performed with four representatives. Considering three notches and four representatives, each test requires 12 specimens (3×4=12). In addition, three temperatures along with three aging stages are applied to the test condition. As mentioned in the standard, specimens are compacted using SGC with 150mm dimension and 120mm height (Figure 4.2). After the compaction, specimens are kept in an oven for 14 days at 40 °C. Then, each specimen is cut in a way to have four semi-circular specimens with a thickness of 57mm. The aging process is started after this point by placing the semi-circular specimens in an oven at 60 °C. At the end, specimens are cut to put the notch in the middle of their flat surface. Figure 4.3 shows the preparation of specimens for SCB test.

Figure 4. 2 Compacted specimens with SGC

(a) (b) (c)

Figure 4. 3 Specimen preparation for SCB test

a) Cutting SGC specimens into semi-circular shape b) letting SCB specimens to be dried before cutting notches c) adding three different notches on SCB specimens

According to ASTM 8044 – 16, the test temperature is to be selected based on climatic intermediate temperature performance grade. The test temperature is obtained from equation 4.3.

$$PG\ IT = \frac{PG\ HT + PG\ LT}{2} + 4 \tag{4.3}$$

Where;

$PG\ IT$ is the intermediate performance grade temperature (°C),

$PG\ HT$ is climatic high performance grade temperature (°C),

$PG\ LT$ is climatic low performance grade temperature (°C).

In this study the grade of bitumen is PG 58-28, therefore the intermediate temperature was calculated as 19 °C. Moreover, SCB test is performed at two other temperatures to evaluate the performance of mixtures in colder temperatures. These temperatures are determined to be -20 °C and 0 °C. The specimens are placed in the climatic chamber prior to the test for at least 2 hours to reach the proposed temperatures. Then, specimens are placed on a three-point bending test fixture to run the test. The fixture has two cylindrical supports with a diameter of 25 mm which are placed in 127mm span length. The cylinders (support) are covered with polytetrafluoroethylene strips to reduce friction (Figure 4.4). By starting the test, a monotonic displacement with a rate of 0.5 mm/min is applied to the top center line of the specimen. Load and vertical displacement are monitored by a data acquisition system during testing. Before starting the test, a preload of 45 ± 10 N is applied to ensure the sample is seated properly.

Figure 4. 4 SCB test fixture

4.2.1 SCB fracture performance

Considering ASTM 8044 -16, one main fracture property is recommended to be investigated. Critical strain energy release rate is the parameter to rank the cracking performance of asphalt mixtures according to elastic plastic fracture mechanics (EPFM). This can be acquired by equation (4.4).

$$J_c = \frac{-1}{b}\left(\frac{dU}{da}\right) \tag{4.4}$$

Where;

J_c is the critical strain energy release rate (kJ/m^2),

b is the sample thickness (m),

a is the notch depth (m),

U is the strain energy to failure (kJ),

dU/da is the change of strain energy with notch depth (kJ/m).

To calculate the value of J_c, primarily it is required to obtain the strain energy to failure U. This parameter can be measured as the area under the load-displacement curve until the peak load. It can be calculated using the equation (4.5).

$$U = \sum_{i=1}^{n} (U_{i+1} - U_i) \times P_i + \frac{1}{2} \times (U_{i+1} - U_i) \times (P_{i+1} - P_i) \tag{4.5}$$

Where;

P_i is the applied load at the i load step (kN),

P_{i+1} is the applied load at the $i+1$ load step (kN),

U_i is the displacement at the i step (m),

U_{i+1} is the displacement at the $i+1$ step (m).

Since there are three different notches for each sample in the test application, the value of U is calculated for each notch. Then, dU/da is obtained by performing linear regression to the data represent the change of strain energy U with notch depth. Four replicates for each notch are tested to have more accurate linear fitting results.

In addition to EPFM approach, linear elastic fracture mechanics (LEFM) can be applied to the acquired data from SCB test to characterize fracture toughness K_{IC} which describes the material's resistance to fracture. These parameters can be obtained using equations 4.6 to 4.8.

$$\sigma_{max} = \frac{4.263\ P_{max}}{D \times t} \tag{4.6}$$

$$K_{IC} = \sigma_{max} \times f\left(\frac{a}{w}\right) \tag{4.7}$$

$$f\left(\frac{a}{w}\right) = -4.9965 + 155.58\left(\frac{a}{w}\right) - 799.94\left(\frac{a}{w}\right)^2 + 2141.9\left(\frac{a}{w}\right)^3 - 2709.1\left(\frac{a}{w}\right)^4 + 1398.6\left(\frac{a}{w}\right)^5 \tag{4.8}$$

Where;

σ_{max} = Maximum stress at failure (N/mm^2),

P_{max} = Maximum load at failure (N),

D = Diameter of specimen (mm),

t = Thickness of specimen (mm),

K_{IC} = Fracture toughness (N/mm$^{2/3}$),

a = Notch length of specimen (mm),

w = Width of specimen (mm).

Figure 4.5 shows sample specimens after failure and initiation of crack.

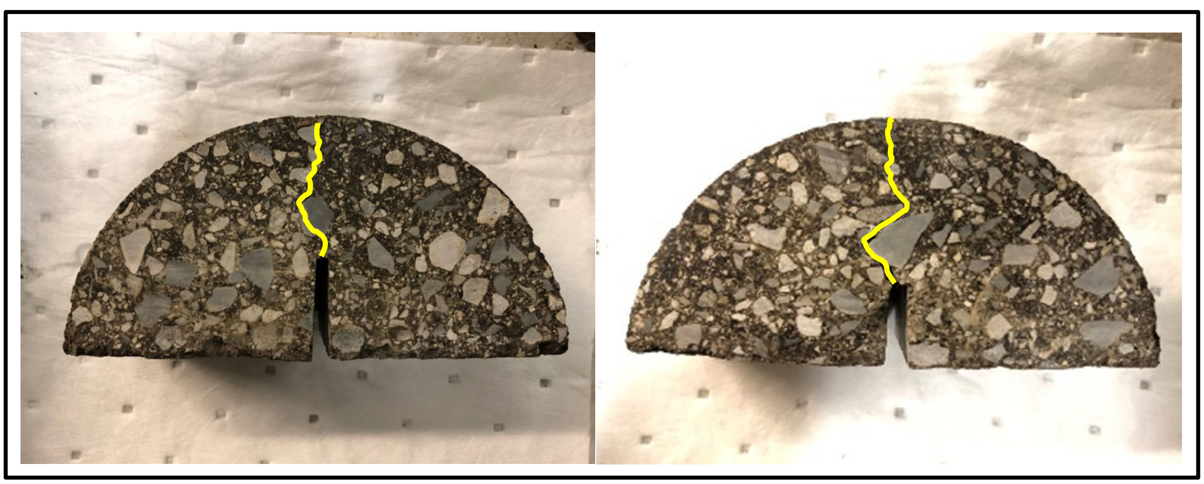

Figure 4. 5 Examples of crack propagation in SCB specimens
(the yellow line highlights the crack position)

Table 4.2 shows the results of SCB test at three temperatures and three aging conditions for mix A which has 1% residual bitumen.

Table 4. 2 SCB result of Mix A (1% residual bitumen)

Temperature	**Aging**	**Notch depth** (mm)	$\mathbf{F_{max}}$ (N)	**U** (average)	**dU/da**	**b** (mm)	$\mathbf{J_c}$ (kJ/m^2)
19 °C	short-term	25	574.4	0.000566	-0.0337	57.8	0.583
		32	349.6	0.000339			
		38	302.3	0.000255			
	long-term	25	579.3	0.000589	-0.0266	58.1	0.458
		32	413.2	0.000393			
		38	308.0	0.000245			
	Extra long-term	25	858.3	0.000486	-0.0208	58.1	0.358
		32	526.8	0.000234			
		38	488.9	0.000221			
0 °C	short-term	25	1111.4	0.000998	-0.0391	57.5	0.680
		32	875.7	0.000724			
		38	674.9	0.000491			
	long-term	25	1121.7	0.001028	-0.0391	57.6	0.679
		32	858.2	0.000655			
		38	714.4	0.000525			
	Extra long-term	25	1275.4	0.000664	-0.0275	58.1	0.473
		32	1010.0	0.000588			
		38	669.3	0.0003			
-20 °C	short-term	25	1545.6	0.000746	-0.0282	57.3	0.492
		32	1165.0	0.00063			
		38	886.2	0.000375			
	long-term	25	1747.2	0.00092	-0.0358	57.5	0.623
		32	1265.0	0.000716			
		38	940.9	0.000452			
	Extra long-term	25	1997.7	0.001194	-0.0518	58.1	0.892
		32	1526.3	0.000807			
		38	1104.7	0.000523			

Following the results for mix A, Table 4.3 represents the SCB output for mix B with 2.2% residual bitumen. The data presented in Tables 4.2 and 4.3 are the average of four representatives in each notch group. The complete results of SCB test are presented in the appendix.

Table 4. 3 SCB result of Mix B (2.2% residual bitumen)

Temperature	**Aging**	**Notch depth** (mm)	$\mathbf{F_{max}}$ (N)	**U** (average)	**dU/da**	**b** (mm)	$\mathbf{J_c}$ (kJ/m^2)
19 °C	short-term	25	716.0	0.000484	-0.0241	57.8	0.417
		32	491.5	0.000263			
		38	337.8	0.000174			
	long-term	25	721.8	0.000479	-0.0219	58.1	0.377
		32	533.7	0.000277			
		38	385.4	0.000197			
	Extra Long-term	25	768.1	0.000478	-0.0227	58.0	0.392
		32	579.5	0.000425			
		38	460.3	0.000237			
0 °C	short-term	25	1300.8	0.000806	-0.0507	57.5	0.882
		32	1070.9	0.000705			
		38	818.0	0.000507			
	long-term	25	1655.7	0.001094	-0.0543	57.6	0.942
		32	1055.6	0.000647			
		38	712.1	0.000392			
	Extra Long-term	25	1581.2	0.001242	-0.056	57.6	0.972
		32	1104.9	0.00075			
		38	846.9	0.000519			
-20 °C	short-term	25	2047.6	0.00132	-0.0571	58.1	0.983
		32	1457.3	0.000823			
		38	1197.9	0.000583			
	long-term	25	2062.5	0.001162	-0.0361	58.1	0.622
		32	1688.2	0.000949			
		38	1358.0	0.000691			
	Extra long-term	25	2308.3	0.001645	-0.0846	57.5	1.472
		32	1687.1	0.000917			
		38	1169.7	0.000553			

CHAPTER 5

RESULTS ANALYSIS AND INTERPRETATION

This chapter is dedicated to analyzing the results of experimental tests presented in chapter 4. The previous chapter represents the data from indirect tensile strength test and semi-circular bending test. To perfectly present and compare the results, various types of graphical figures and charts are employed along with the statistical analysis of data. First, water loss in curing process is discussed. Then, the results of indirect tensile strength test and semi-circular bending test considering the independent variables are vastly interpreted and discussed. Table 5.1 represents the independent and dependent variables in this study.

Table 5. 1 Independent and dependent variables

Experimental task	Independent variable	Dependent variable
Curing	Curing duration (day)	Residual water
Indirect tensile strength test	Bitumen content (0%, 1%, 2.2%)	ITS strength, ITR
	Moisture content (wet & dry)	
	Aging condition (STOA, LTOA, XLTOA)	
Semi-circular bending test	Bitumen content (1%, 2.2%)	Critical strain energy release rate (J_c), Fracture toughness (K_{IC}), Maximum Load (F_{max})
	Temperature (-20 °C, 0 °C, 19 °C)	
	Aging condition (STOA, LTOA, XLTOA)	

5.1 Curing

The curing process is included placing specimens in an oven at 40 °C for 14 days. Since there are two mix designs and two different specimen sizes with diameters of 100mm and 150mm, water loss in all samples needed to be monitored. Therefore, the residual water in the specimens is reordered right after compaction and every day during the curing process. Since there is inevitable particle loss while extruding specimens, the residual water for day 0 is considered equal to the initial water in the mixture. Thus, the initial residual water for all specimens is assumed 3.58%. Figures 5.1 and 5.2 present water loss trends for 100mm and 150mm specimens, respectively.

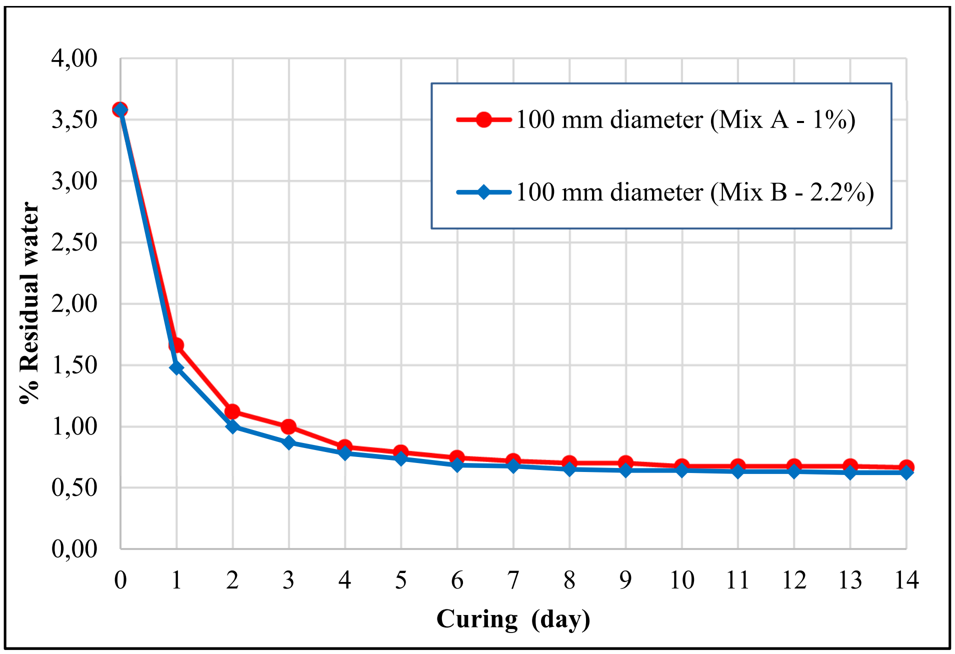

Figure 5. 1 Water loss for 100 mm diameter specimens

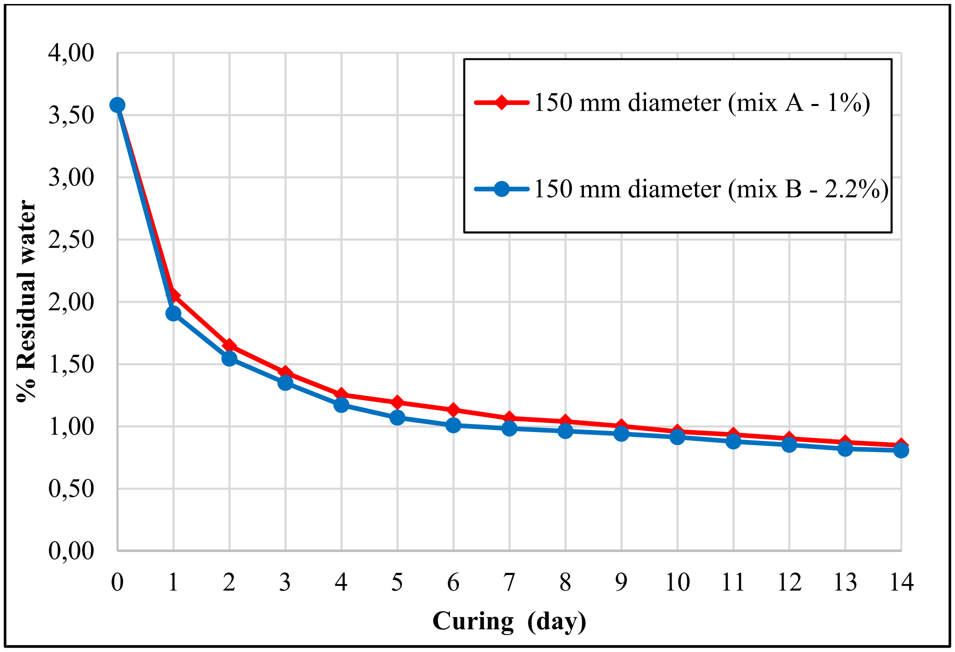

Figure 5. 2 Water loss for 150 mm diameter specimens

Bilinear regression analysis is utilized to predict water evaporation after 14 days. For this matter, the first regression is considered from the beginning to day 3. Then, the second regression is applied from day 3 to 14. The details of mentioned regressions are presented in Table 5.2. As expected, residual water after 14 days of curing in 150 mm specimens is more than for the 100mm specimens. By using bilinear regression, Δday shows the number of extra days after 14 days of curing to reach a constant number in residual water for 100mm and 150mm specimens.

Table 5. 2 Bilinear regression analysis of water loss

Specimen	Regression factors (day 0 to day 3)			Regression factors (day 3 to day 14)			Δdays
	C_1	I_1	R^2	C_2	I_2	R^2	
Mix A 150mm	-0.685	3.204	0.831	-0.039	1.373	0.973	21
Mix A 100mm	-0.829	3.082	0.801	-0.014	0.844	0.804	
Mix B 150mm	-0.705	3.153	0.802	-0.032	1.237	0.942	23
Mix B 100mm	-0.862	3.023	0.779	-0.013	0.786	0.778	

C is the slope of regression, I is the intercept value, R^2 is the regression coefficient

While the difference of residual water in mix A for 100mm and 150mm specimens is 0.183 % after 14 days of curing, it takes 21 more days in the same condition to reach the same content in residual water for both specimen sizes. For mix B, the difference in residual water is 0.186 % at the end of curing and it takes 23 more days to have the same percentage of residual water for both specimen sizes. Considering the effect of bitumen emulsion content on the water evaporation rate, it can be seen that both mixes reach relatively the same amount of residual water by the end of curing process. Therefore, the emulsion content does not have an impact on the water evaporation rate in curing process.

5.2 Indirect tensile strength (ITS) test

Indirect tensile strength test was utilized to evaluate the strength of asphalt mixtures. In this section, the results of ITS presented in chapter 4 are analyzed and interpreted. The test is conducted for three mix designs with single source RAP. The residual bitumen content is the only difference in mix A and B which is 1.0% and 2.2%, respectively. In addition, the third mix design with 100% RAP and no additives is prepared as mix C. The test is performed at 25 °C in wet and dry conditions for three different aging stages. The values in this part are the average of three representatives for each sample.

5.2.1 Effect of aging

Three aging conditions are applied to the specimens before testing. Specimens gained more strength by increasing the duration of oven aging. This effect can be observed for all three stages of aging in mix A and B. In other words, there is an increasing trend in ITS strength of specimens by increasing the duration of oven aging. Figures 5.3 and 5.4 show the results of ITS in dry and wet conditions for three mix designs regarding their different aging stages. Error bars in Figures 5.3 and 5.4 represent the standard deviation of ITS results in dry and wet conditions.

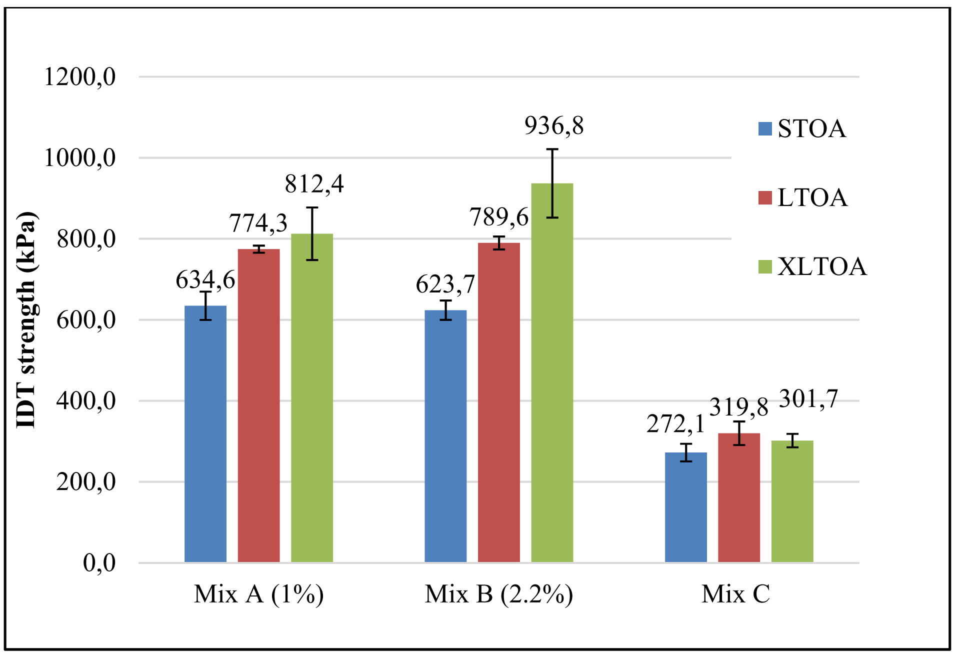

Figure 5. 3 ITS results comparison in dry condition

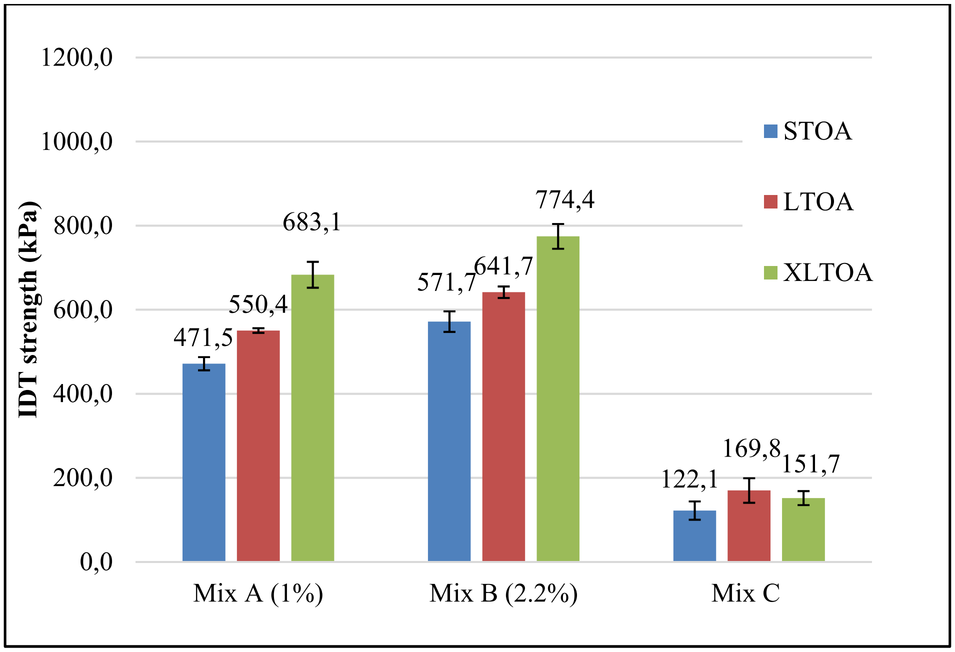

Figure 5. 4 ITS results comparison in wet condition

According to the results of ITS for dry and wet specimens, ITS strength increases with extending the duration of oven aging. Moreover, it can be observed from Figure 5.4 that the increase of ITS strength in mix B with 2.2 percent residual bitumen is more than mix A. This fact is more noticeable in the transition from long-term aged to extra long-term aged specimens. ITS strength for mix C with 100% RAP and no other additives has increased over aging, however, there is no prominent difference between its ITS results in LTOA and XLTOA conditions. This result for mix C shows that the assumption to consider RAP as black rock is not accurate. As Table 5.3 shows, there is a 50% increase in ITS_{dry} (Mix B) from STOA to XLTOA condition. While the corresponding rise is 28% for Mix A.

Table 5. 3 ITS analysis for different aging conditions

Mix design	ITS_{dry} (kPa)			% of increase from STOA to XLTOA[1]
	STOA	LTOA	XLTOA	
Mix A	634.6	774.3	812.4	28%
Mix B	623.7	789.6	936.8	50%
Mix C	272.1	319.8	301.7	11%

[1] % increase = (ITS_{XLTOA} - ITS_{STOA}) / ITS_{STOA}

Despite no additives in mix C, an 11% increase happened from STOA to XLTOA condition. This result shows that for the given RAP, a longer duration of oven aging leads to a higher amount of ITS regardless of existing fresh bitumen in the mixtures. This phenomenon could happened due to increasing temprerature and re-coating the aggregates with the old bitumen. Overall, the results shows that with longer oven aging, the RAP does deem to increase the ITS. This fact needs more tests in different aging conditions to better study the aspect. Considering all potential reasons, it can be derived that RAP cannot be assumed as black rock. To better visualize this increasing trend in the results of these three mixtures, Figure 5.5 illustrates the data from Table 5.3. similar to other figures, error bars in Figure 5.5 show the stadnard deviation of ITS results from three representatives specimens.

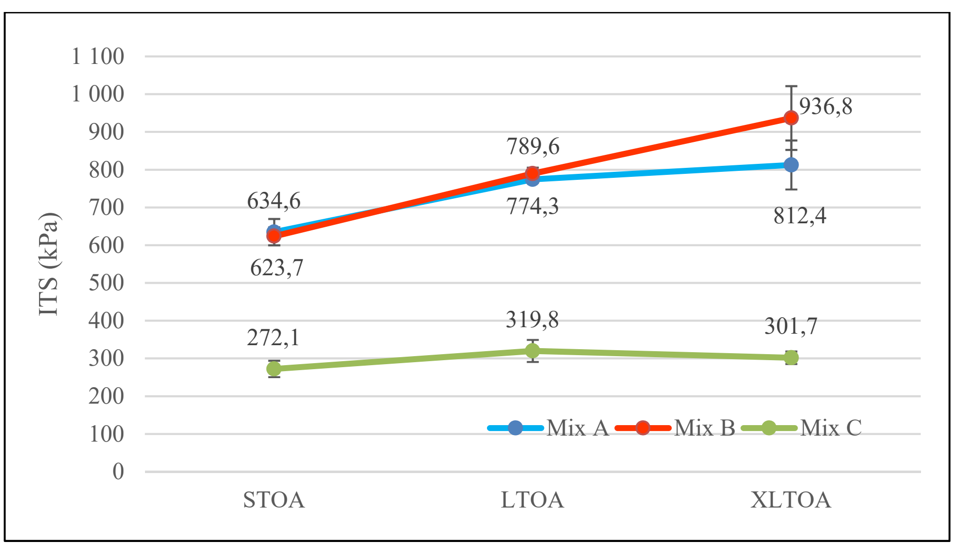

Figure 5. 5 ITS results in different aging conditions

5.2.2 Effect of binder content

As presented in chapter 3, the mixtures are made of a single source RAP and the only difference between mix A and B is the binder content. Besides, the compaction energy in production of both mixes is designed in a way to have the same air voids (11%). Since there is no added binder in mix C, this mixture is out of the scope of this section. Therefore, the effect of binder content is subjected to be analyzed in this part. Table 5.4 represents the comparison in ITS results of mix A and B regarding aging conditions.

Table 5. 4 ITS results for different binder contents

Aging condition	**ITS (kPa)**		**Percentage of increase**[1]
	Mix A	**Mix B**	
STOA	634.57	623.70	-2%
LTOA	774.27	789.64	2%
XLTOA	812.43	936.77	15%

[1] % increase = $(ITS_{\text{Mix B}} - ITS_{\text{Mix A}}) / ITS_{\text{Mix A}}$

Given the data from the last column in Table 5.4, there are insignificant differences between the results of mix A and B for STOA and LTOA conditions. On the contrary, for a longer duration of oven aging the difference in ITS results is more notable. It shows that binder content has more effect on the ITS results when the duration of aging in longer.

5.2.3 Water sensitivity (ITR)

ITR as an indicator ratio for evaluating water sensitivity of Mixtures is employed. The same conditions are applied in the production, curing, and aging of specimens in dry and wet conditions. Table 5.5 represents the results of ITR for the different mixes.

Table 5. 5 ITR results of mixtures

Aging condition	Mix design		Change percentage[1]
	Mix A	Mix B	
STOA	0.74	0.92	23%
LTOA	0.71	0.81	14%
XLTOA	0.84	0.83	-2%

[1]change percentage = ($ITR_{Mix\ B}$ – $ITR_{Mix\ A}$) / $ITS_{Mix\ A}$

Although the value of ITR for mix B is slightly less than mix A in XLTOA condition, mix B showed better resistance to moisture in STOA and LTOA conditions. Moreover, the maximum difference in ITR results between mix A and B occurred in STOA condition. The value of ITR is 23% higher for Mix B in STOA condition.

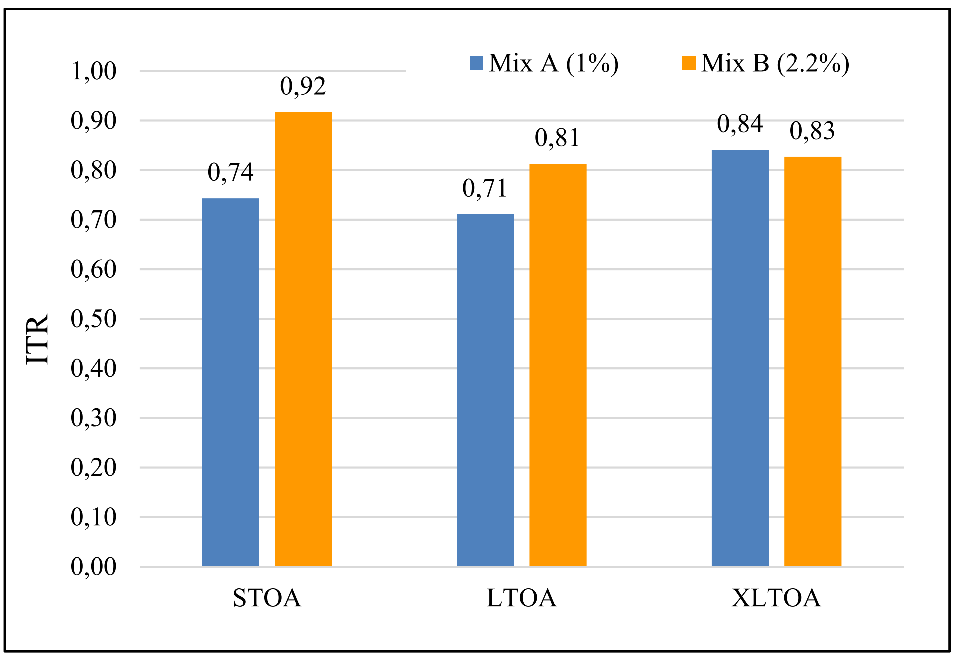

Figure 5. 6 ITR results comparison for mix A and B

5.2.4 Data quality

Quality of testing and analyzing data is a foundation to build the conclusions and findings from the research work. In other words, the conclusions of research could be reliable in case of having accurate data. One measure of data quality is the coefficient of variation (COV) that is used in this study. Since COV measure relative dispersion or variation in a data set, it is better to have lower values for COV which means less variation in the data set. For ITS test, Figure 5.7 presents the COV of data for different mixtures.

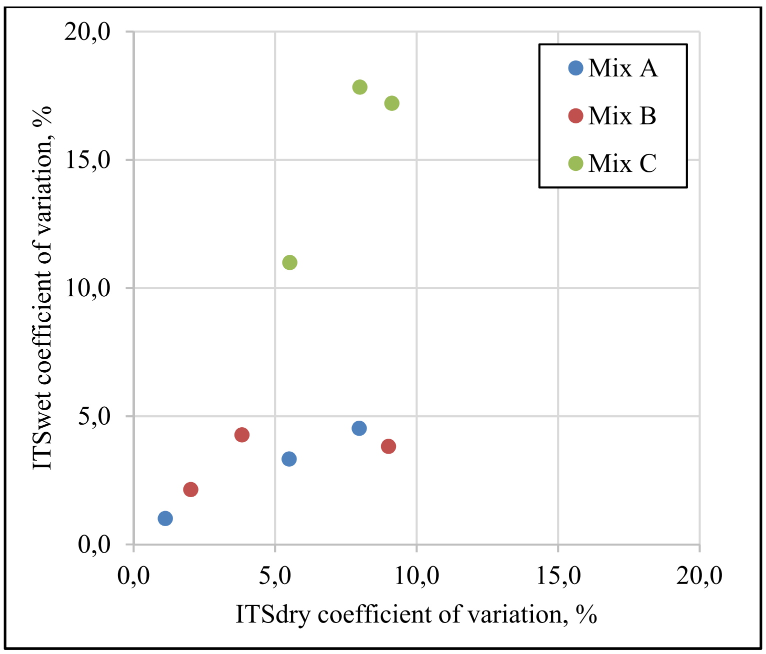

Figure 5. 7 Coefficient of variation comparison for ITS

According to Figure 5.7, the COV of ITS for all mixtures are less than 20%. The quality of data can be affected by specimen preparation, air voids, material source, quality of test equipment, test operation, and interpretation methodology.

5.3 Semi-circular bending (SCB) test

The results of SCB tests are analyzed and discussed in this chapter. SCB test in this study is conducted in three different temperatures with specimens in three aging conditions. For each sample, three notches are prepared and for each notch, four representatives are tested. Therefore, the results which are presented in this chapter are the average of four specimens.

Since the SCB test is performed at -20 °C, 0 °C, and 19 °C, the behaviour of mixtures is expected to be more brittle at lower temperatures in comparison with intermediate temperature. Thus, the assumption for analyzing data is considered based on both approaches of elastic plastic fracture mechanics (EPFM) and linear elastic fracture mechanics (LEFM). According to the common practices of SCB test J_c and J-integral are defined based on EPFM approach while, fracture toughness K_{IC} and fracture energy are based on LEFM. A vast explanation of calculating each parameter was mentioned in chapter 2. In this chapter, the effect of temperature and aging is investigated on cracking resistance of the mixtures with two different binder contents. Table 5.6 shows the results of SCB for mix A at three temperatures and three aging conditions. The numbers in the tables represent the average of four representatives for each notch depth.

Table 5. 6 SCB result for mix A

Temperature	**Aging**	**Notch depth** (mm)	F_{max} (N)	σ_{max} (N/mm^2)	K_{IC} ($N/mm^{2/3}$)	**Jc** (kJ/m^2)
19 °C	short-term	25	574.4	0.282	2.71	0.583
		32	349.6	0.173	2.09	
		38	302.3	0.149	2.27	
	long-term	25	579.3	0.283	2.73	0.458
		32	413.2	0.202	2.45	
		38	308.0	0.114	1.73	
	Extra long-term	25	858.3	0.420	4.04	0.358
		32	526.8	0.258	3.13	
		38	488.9	0.239	3.65	
0 °C	short-term	25	1111.4	0.549	5.28	0.680
		32	875.7	0.435	5.28	
		38	674.9	0.333	5.07	
	long-term	25	1121.7	0.551	5.31	0.679
		32	858.2	0.421	5.11	
		38	714.4	0.354	5.40	
	Extra long-term	25	1275.4	0.626	6.02	0.473
		32	1010.0	0.494	5.99	
		38	669.3	0.327	4.97	
-20 °C	short-term	25	1545.6	0.763	7.34	0.492
		32	1165.0	0.578	7.01	
		38	886.2	0.442	6.73	
	long-term	25	1747.2	0.863	8.31	0.623
		32	1265.0	0.622	7.54	
		38	940.9	0.469	7.14	
	Extra long-term	25	1997.7	0.980	9.43	0.892
		32	1526.3	0.748	9.06	
		38	1104.7	0.539	8.21	

Similar to Table 5.6, SCB results of mix B are presented in Table 5.7. In these tables, the parameters including J_c, σ_{max} K_{IC}, based on LEFM and EPFM are calculated.

Table 5. 7 SCB results for mix B

Temperature	**Aging**	**Notch depth** (mm)	$\mathbf{F_{max}}$ (N)	$\boldsymbol{\sigma}$ $_{\mathbf{max}}$ (N/mm^2)	$\mathbf{K_{IC}}$ (N/mm$^{2/3}$)	**Jc** (kJ/m^2)
19 °C	short-term	25	716.0	0.351	3.38	0.417
		32	491.5	0.242	2.94	
		38	337.8	0.166	2.53	
	long-term	25	721.8	0.353	3.40	0.377
		32	533.7	0.261	3.17	
		38	385.4	0.189	2.88	
	Extra Long-term	25	768.1	0.378	3.63	0.392
		32	579.5	0.284	3.44	
		38	460.3	0.226	3.44	
0 °C	short-term	25	1300.8	0.642	6.17	0.882
		32	1070.9	0.533	6.46	
		38	818.0	0.403	6.13	
	long-term	25	1655.7	0.814	7.83	0.942
		32	1055.6	0.519	6.29	
		38	712.1	0.353	5.38	
	Extra Long-term	25	1581.2	0.778	7.48	0.972
		32	1104.9	0.543	6.59	
		38	846.9	0.420	6.40	
-20 °C	short-term	25	2047.6	1.005	9.67	0.983
		32	1457.3	0.713	8.64	
		38	1197.9	0.585	8.91	
	long-term	25	2062.5	1.012	9.73	0.799
		32	1688.2	0.826	10.02	
		38	1169.7	0.570	8.69	
	Extra long-term	25	2308.3	1.141	10.97	1.293
		32	1687.1	0.830	10.06	
		38	1358.0	0.677	10.31	

5.3.1 Effect of aging

Similar to ITS, three aging conditions are proposed for SCB test. Since the SCB test is performed by controlling the axial displacement rate at 0.5mm/min, the output is in form of a load-displacement curve which all parameters are derived from it. The tests are run until reached 100 N after the peak load. The effect of aging on the cracking performance parameters of mixtures is targeted in this section.

5.3.1.1 Effect of aging on F_{max}

The effect of aging on the maximum load can be observed in Tables 5.6 and 5.7. By increasing the duration of oven aging, the maximum load that specimens can withstand increases. This trend is the same for both mixtures. To better illustrate the effect of aging on F_{max}, Figures 5.8, 5.9, and 5.10 show the maximum load in different aging conditions. The error bars in all figures represent the standard deviation for the test results.

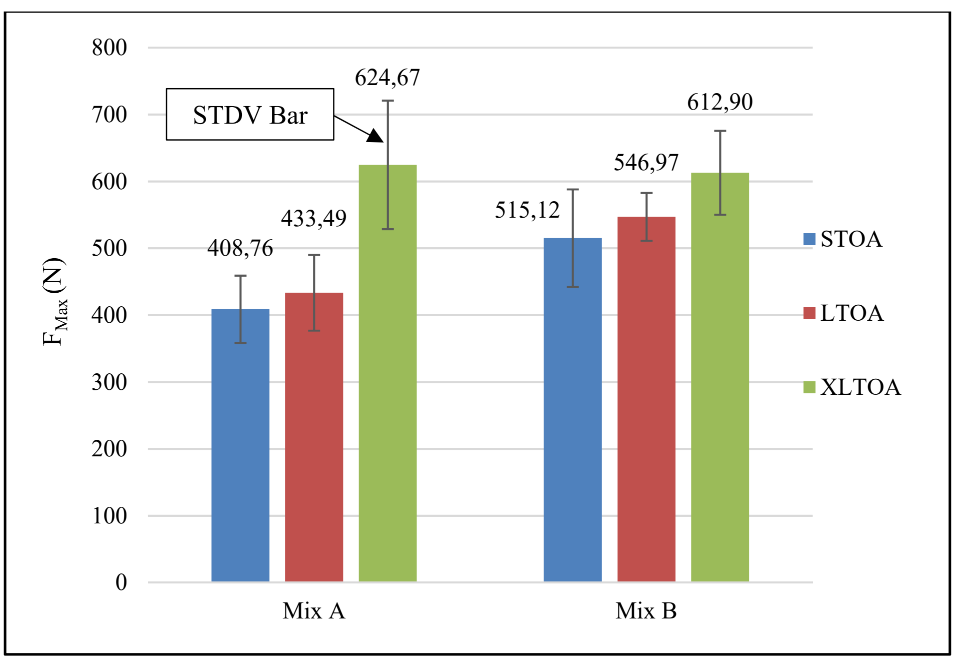

Figure 5. 8 SCB maximum load at 19 °C

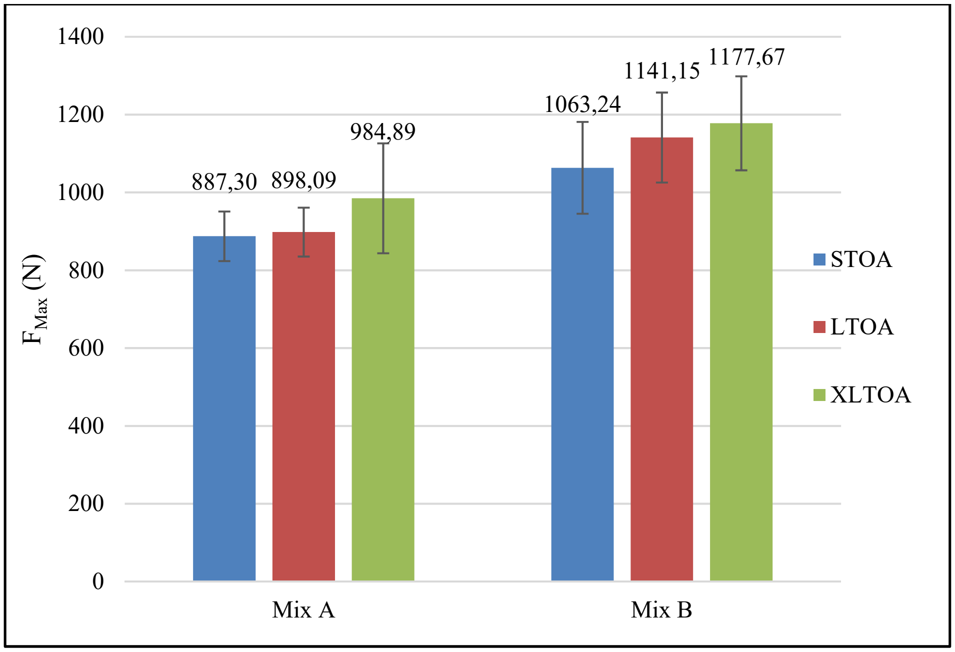

Figure 5. 9 SCB maximum load at 0 °C

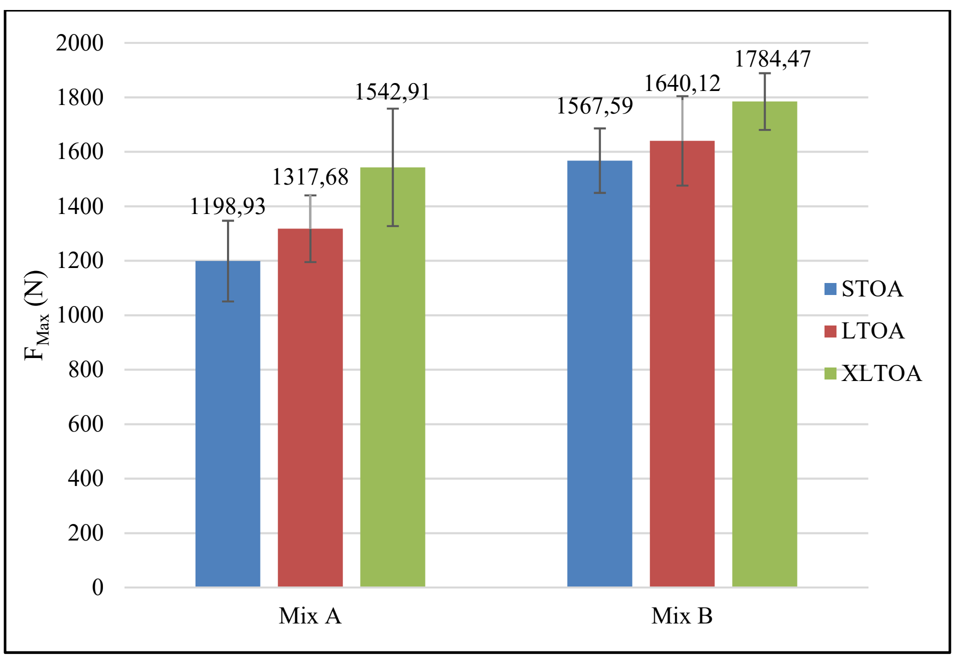

Figure 5. 10 SCB maximum load at -20 °C

As presented in Figures 5.8, 5.9, and 5.10, maximum load rises with the increase of oven aging duration. This trend exists for both mix designs, however, the maximum load has its highest increase at 19 °C for XLTOA. The values from the previous figures are converted to Table 5.8 to statistically compare and analyze the data. As expected, the maximum force increases when the temperature decreases.

Table 5. 8 Effect of aging on maximum load in SCB test

Temperature (°C)	**Mix Design**	**Maximum Force (N)**			**Change percentage[1]**
		STOA	LTOA	XLTOA	
19	Mix A	408.70	433.40	624.60	53%
	Mix B	515.10	546.90	613.00	19%
0	Mix A	887.30	898.00	984.80	11%
	Mix B	1063.20	1141.10	1177.60	11%
-20	Mix A	1198.90	1317.60	1542.90	29%
	Mix B	1567.50	1640.12	1784.47	14%

[1]change percentage = ($F_{max(XLTOA)}$-$F_{max(STOA)}$)/ $F_{max(STOA)}$

According to the data from Table 5.8, the highest increase in maximum load happened at 19 °C which is 53% and 19% for mix A and B respectively.

5.3.1.2 Effect of aging on the critical strain energy release rate (J_c)

J_c as a part of EPFM approach is calculated accounting for the area under the load-displacement curve until the peak load and then plotted against the notch depths. The complete description for calculating J_c is presented in chapter 4. Research studies proved that J-integral showed higher degree of sensitivity in comparison with peak load, strain energy, and deformation with respect to asphalt binder, and nominal maximum aggregate size (NMAS) (Saha & Biligiri, 2016). Due to the definition of J-integral, when a mixture has higher J_c, it means higher energy requires to initiate the crack. In other words, the mixture has higher crack resistance. An example procedure to calculate J_c according to ASTM D8044 is presented in Figures 5.11 and 5.12. Calculation of the critical strain energy release rate is derived from load-displacement curves of mix B at 19 °C and LTOA.

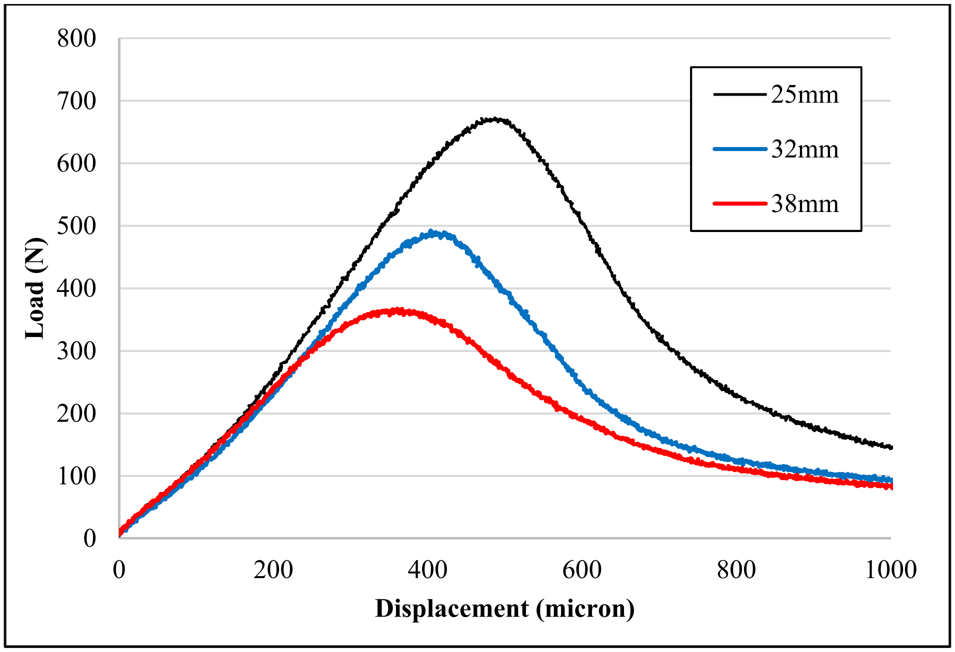

Figure 5. 11 SCB load-displacement curve (Mix B-19°C-LTOA)

The three curves in Figure 5.11 are the average of four representatives for each notch depth. Moreover, the area under the load-displacement curve (U) is measured using equation 4.5, then plotted strain energy to failure (U) against the notch depths and modelled the data with a linear regression line in Figure 5.12. The slope of the linear regression line represents the change of strain energy with notch depth (dU/da). Finally, the critical strain energy release rate is calculated using equation 4.4.

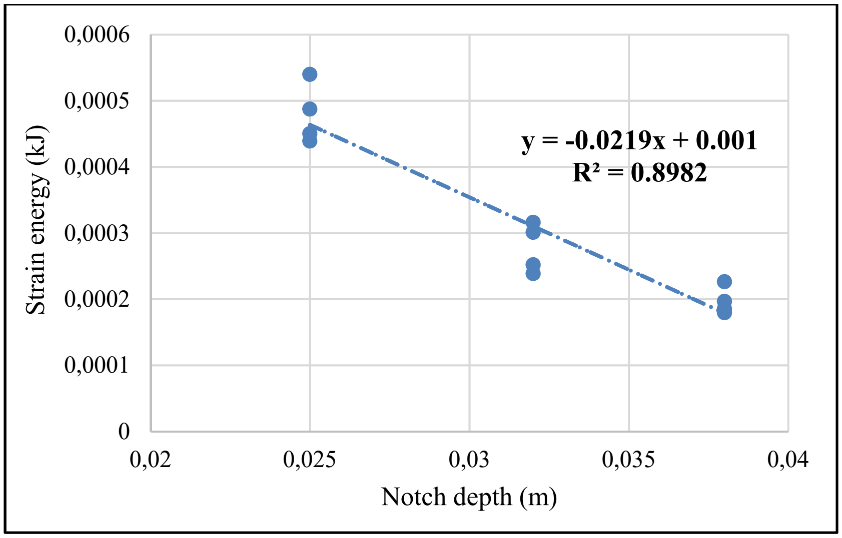

Figure 5. 12 Notch depth versus strain energy plot (Mix B-19°C-LTOA)

Figures 5.13 and 5.14 illustrate the value of J_c for both mixes in different aging. According to the equation for calculating J_c (see chapter 4, equation 4.4) Standard deviation bars are not applicable to J_c. Instead, the accuracy of data is evaluated by using the coefficient of variation (COV) for peak loads and the areas under load-displacement curves in section 5.4.

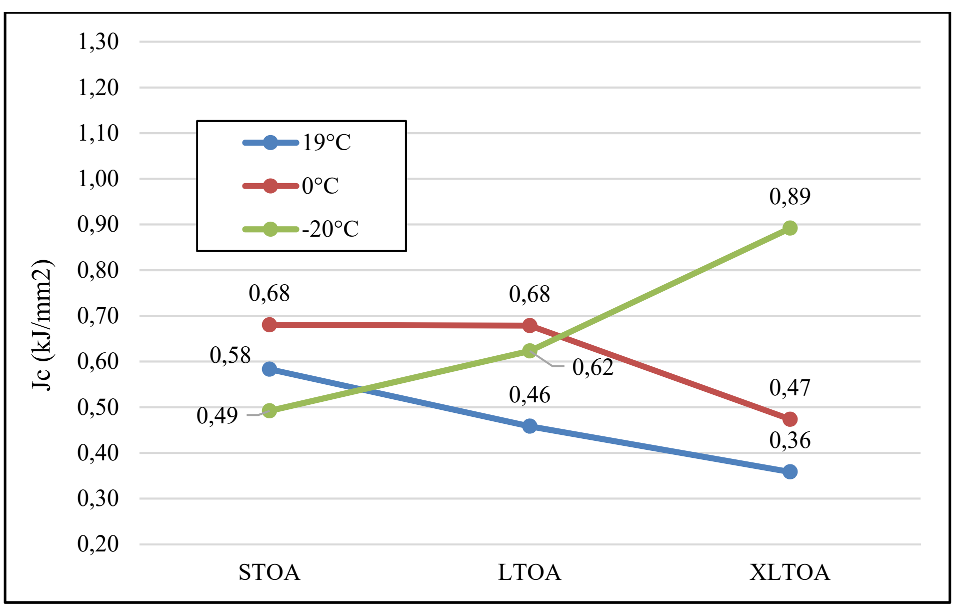

Figure 5. 13 J_C results for mix A

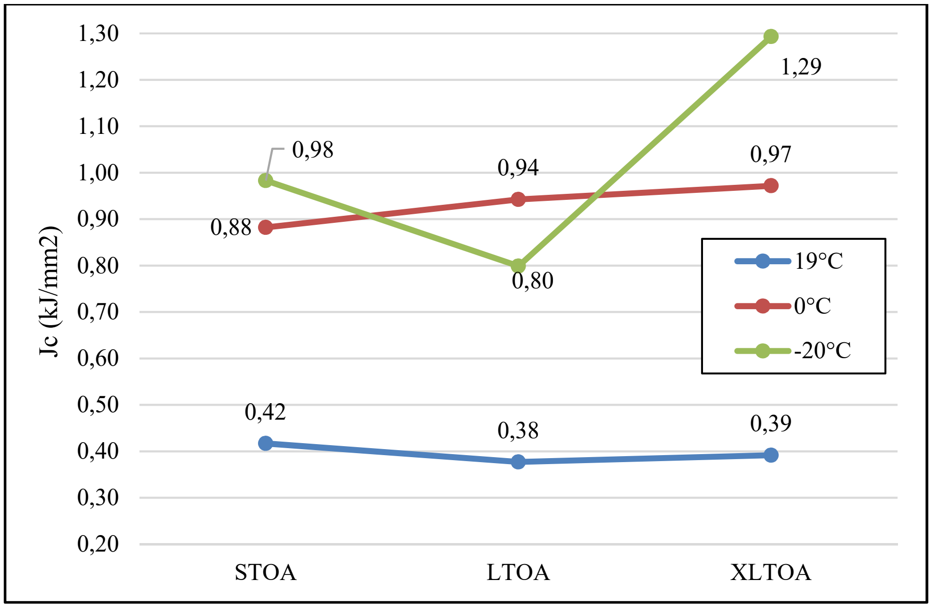

Figure 5. 14 J_c results for mix B

Data from Figures 5.13 and 5.14 are transferred to Table 5.9 to better represent the values and trends of J_c in different conditions.

Table 5. 9 J_c results in different aging conditions

Temperature	**Mix Design**	**J_c (kJ/mm^2)**			**Change percentage**[1]
		STOA	LTOA	XLTOA	
19 °C	Mix A	0.58	0.46	0.36	-39%
	Mix B	0.42	0.38	0.39	-6%
0 °C	Mix A	0.68	0.68	0.47	-30%
	Mix B	0.88	0.94	0.97	10%
-20 °C	Mix A	0.49	0.62	0.89	81%
	Mix B	0.98	0.80	1.29	32%

[1] Change percentage = $(\mathbf{J_{c\,XLTOA}} - \mathbf{J_{c\,STOA}}) / \mathbf{Jc_{STOA}}$

The value of critical strain energy release rate (J_c) decreased by increasing oven aging duration at 19 °C and 0 °C for mix A. As presented in Table 5.9, J_c has an extreme increase from STOA to XLTOA at -20 °C. The decreasing trend of J_c indicates lower energy is required to initiate crack in aged asphalt mixtures in this study. This fact happened at intermediate temperature (19 °C). However, cracking behaviour of the specimens does not follow the same trend in lower temperatures. Since the concept of J_c is defined based on EPFM, at lower temperatures the behaviour of asphalt mixtures is more brittle than at intermediate temperatures. Therefore, EPFM approach at lower temperatures is not valid. Thus, this fact has the potential to be the reason for the difference in the trend of J_c value at 19 °C and -20 °C. As a result, another parameter is required to analyze all temperatures.

5.3.1.3 Effect of aging on Fracture toughness (K_{IC})

Fracture toughness (K_{IC}) as a parameter to evaluate the material's resistance against fracture, is determined based on linear elastic fracture mechanics (LEFM). Fracture toughness (K_{IC}) explains stress distribution near the crack tip at failure. According to the definition of fracture toughness, this parameter better covers the behaviour analysis of the mixtures in compare with J_c. Since the procedure to prepare the specimens and conduct the SCB test is done according to ASTM D8044, three different notch depths are made in SCB test. Given the fact that calculating K_{IC} depends on the geometry of specimens, fracture toughness has a different value for each notch depth. Therefore, an average of K_{IC} for the three notch depths is considered to analyze and compare this parameter in various conditions. Figures 5.15 and 5.16 Show the changes in fracture toughness for mix A and B in different conditions.

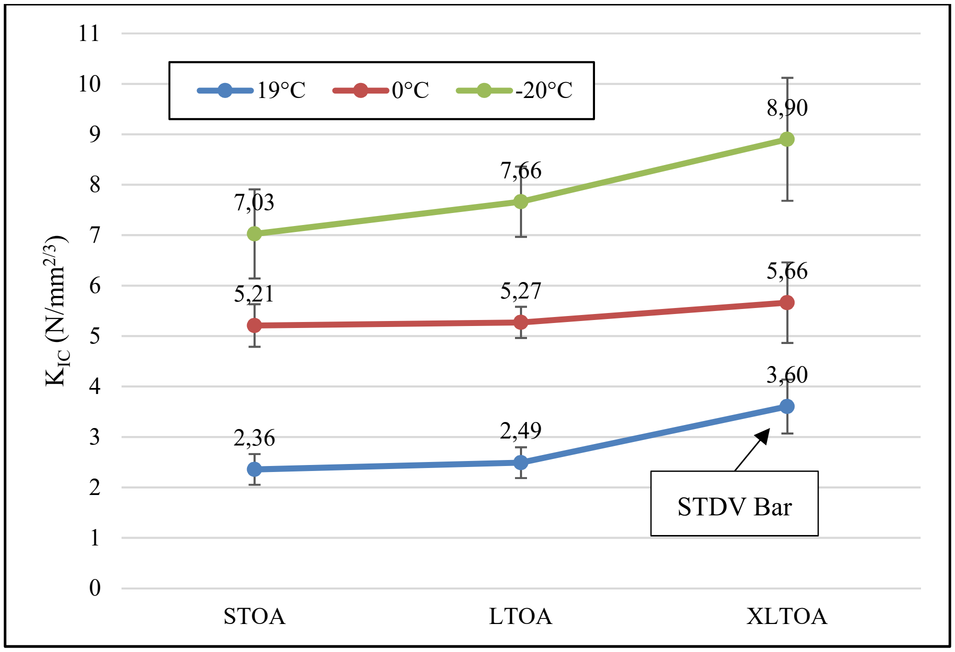

Figure 5. 15 K_{IC} results for mix A

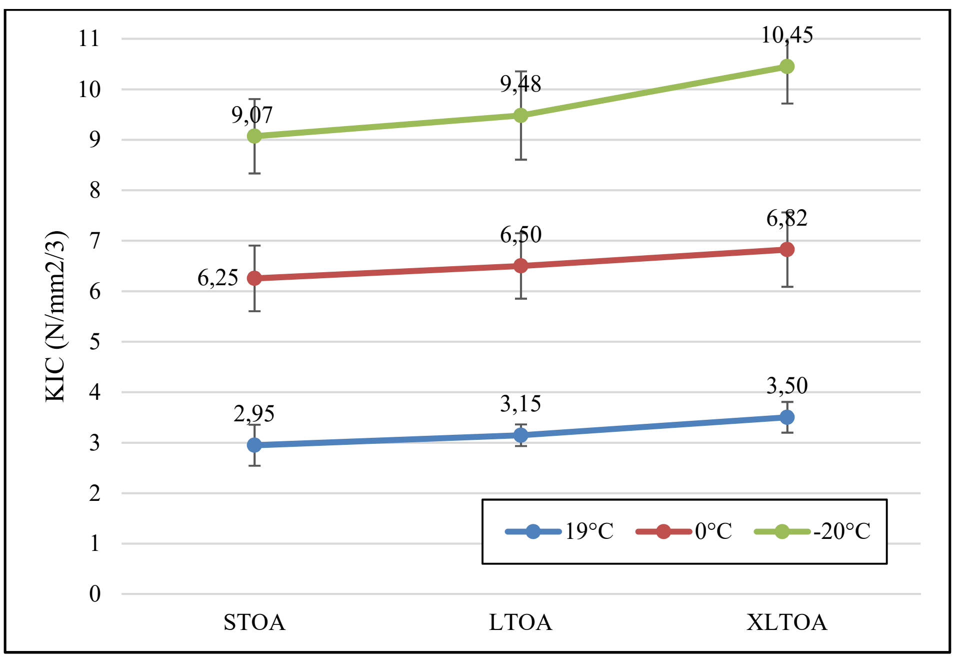

Figure 5. 16 K_{IC} results for mix B

To statistically compare the K_{IC} results, data from Figures 5.15 and 5.16 are imported into Table 5.10. The increasing percentages of K_{IC} from STOA to XLTOA are highlighted in the table. In general, K_{IC} increases by increasing the oven aging duration. This trend exists in both mixtures in all three temperatures.

Table 5. 10 K_{IC} results in different aging conditions

Temperature	**Mix Design**	**K_{IC} (N/mm$^{2/3}$)**			**Change percentage**
		STOA	LTOA	XLTOA	
19 °C	Mix A	2.36	2.49	3.60	53%
	Mix B	2.95	3.15	3.50	19%
0 °C	Mix A	5.21	5.27	5.66	9%
	Mix B	6.25	6.50	6.82	9%
-20 °C	Mix A	7.03	7.66	8.90	27%
	Mix B	9.07	9.48	10.45	15%

According to Table 5.10, aging has more effect on fracture toughness at 19 °C in both mixtures. Fracture toughness on average increased by 36% for Mix A and B at 19 °C.

5.3.2 Effect of temperature

Apart from the recommended SCB test temperature based on ASTM D8044, two other temperatures are considered to evaluate the behaviour of asphalt mixtures at low temperatures. Therefore, 19 °C as an intermediate temperature regarding the performance grade of asphalt binder was selected along with 0 °C and -20 °C as the common low temperatures for SCB. Conditioning specimens were performed according to ASTM D8044 to reach the targeted temperature prior to testing.

5.3.2.1 Effect of temperature on F_{max}

The maximum loads corresponding to different temperatures are presented in Tables 5.6 and 5.7. To better show the effect of temperature on F_{max}, Figures 5.17, 5.18, and 5.19 visualize the data from SCB results.

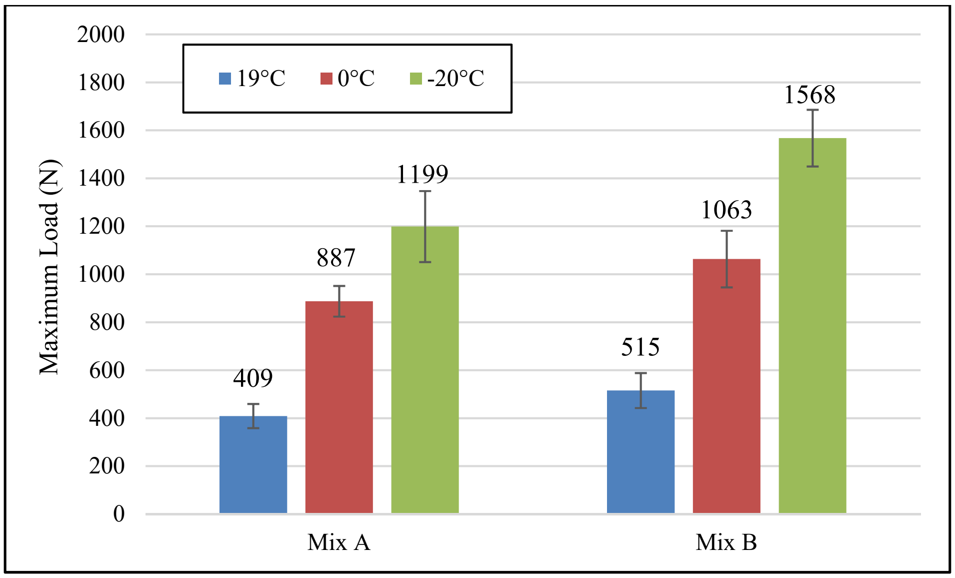

Figure 5. 17 SCB maximum load for STOA

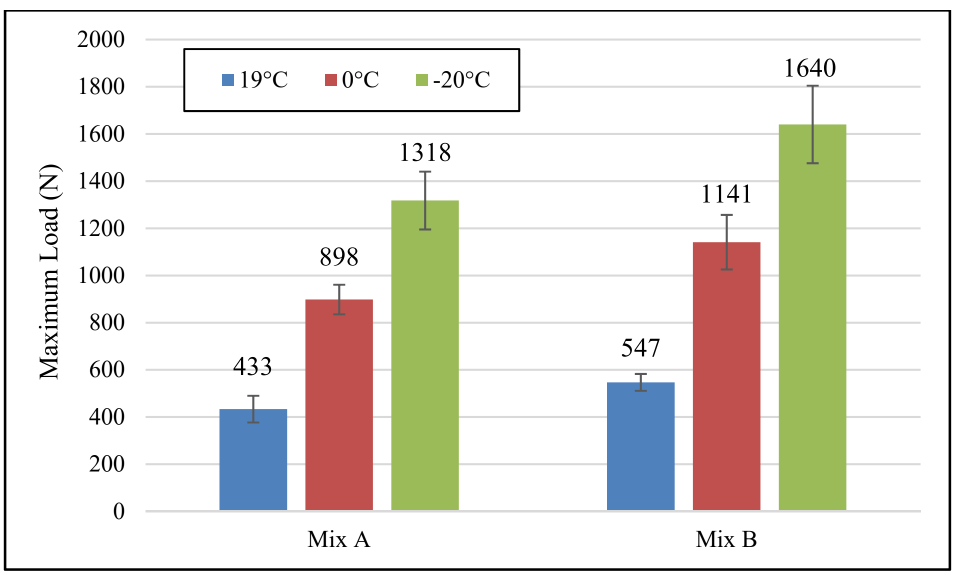

Figure 5. 18 SCB maximum load for LTOA

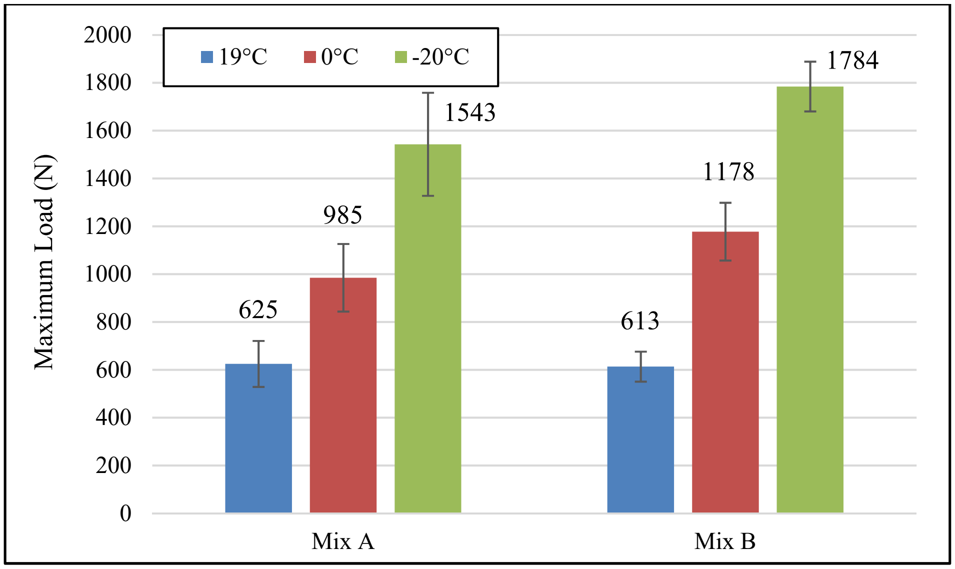

Figure 5. 19 SCB maximum load for XLTOA

As it can be observed from Figures 5.18, 5.19, and 5.20, by decreasing temperatures from 19 °C to -20 °C, the maximum load increases consequently. This fact has more effect on the mixture with higher content of asphalt binder. Since the mix designs are the same except for the binder content, which is different in mix A and B, changing temperature affects mix B (higher binder content) more than Mix A. Table 5.11 shows statistical data for maximum load and its difference regarding temperatures.

Table 5. 11 Effect of temperature on maximum load in SCB test

Temperature (°C)	Mix Design	Maximum Force (N)			Change Coefficient [1]
		19 °C	0 °C	-20 °C	
STOA	Mix A	408.7	887.3	1198.9	2.9
	Mix B	515.1	1063.2	1567.5	3.0
LTOA	Mix A	433.4	898	1317.6	3.0
	Mix B	546.9	1141.1	1640.124	3.0
XLTOA	Mix A	624.6	984.8	1542.9	2.5
	Mix B	613	1177.6	1784.471	2.9

[1] Change coefficient = Maximum Force $_{-20\,°C}$ / Maximum Force $_{19\,°C}$

The change coefficient calculated in Table 5.11 shows that decreasing temperature from 19 °C to -20 °C causes increasing the maximum load between 2.5 and 3 times. In addition, the rise of the maximum load is higher for mix B in all aging conditions in comparison with mix A.

5.3.2.2 Effect of temperature on the critical strain energy release rate (J_c)

To illustrate the effect of temperature on the critical strain energy release rate (J_c), related results are derived from Tables 5.6 and 5.7 and present in Figures 5.21 and 5.22. These figures show the change of J_c at different temperatures.

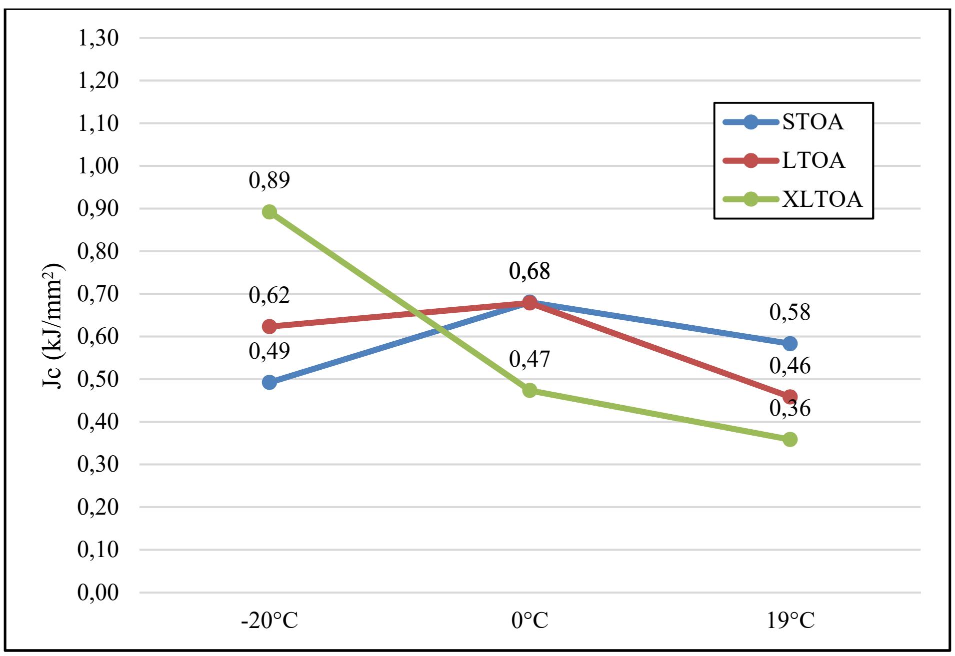

Figure 5. 20 Effect of temperature on J_c for mix A

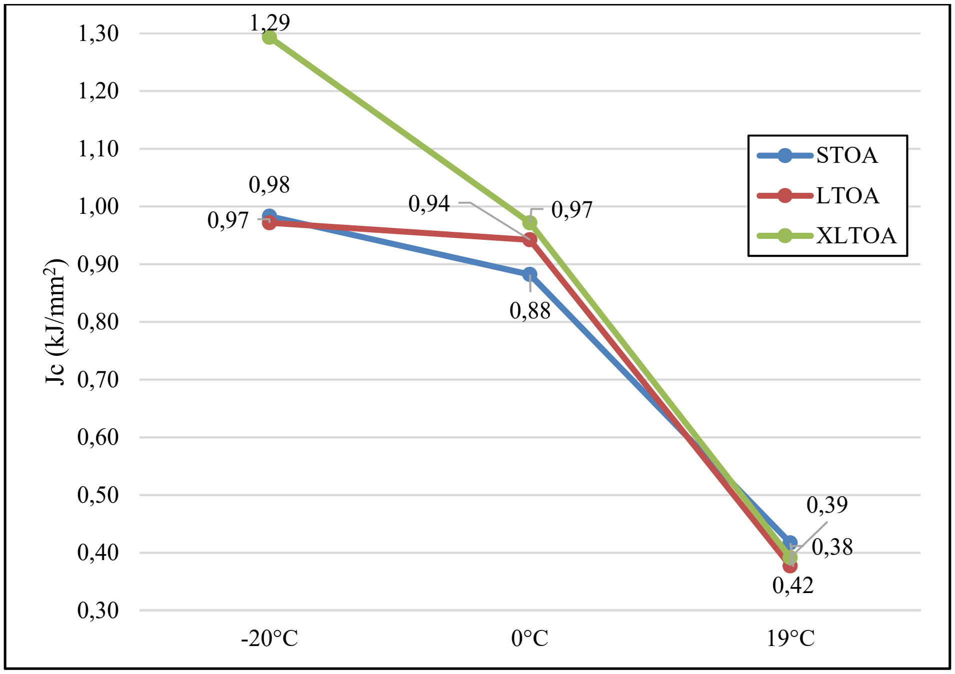

Figure 5. 21 Effect of temperature on J_c for mix B

The value of J_c has its maximum at -20 °C for both mixtures, while mix B has higher J_c at -20 °C and 0 °C. In both mixes, the critical strain energy release rate (J_c) decreases when the temperature increases from 0 °C to 19 °C. Table 5.12 presents the J_C results and their changes with different temperatures.

Table 5. 12 J_c results in different temperatures

Temperature	**Mix Design**	**Jc (kJ/mm^2)**			**Change percentage** [1]
		-20 °C	0 °C	19 °C	
STOA	Mix A	0.49	0.68	0.58	19%
	Mix B	0.98	0.88	0.42	-58%
LTOA	Mix A	0.62	0.68	0.46	-26%
	Mix B	0.97	0.94	0.38	-61%
XLTOA	Mix A	0.89	0.47	0.36	-60%
	Mix B	1.29	0.97	0.39	-70%

[1] Change percentage = (**Jc** $_{19\,°C}$ **- Jc** $_{-20\,°C}$) / **Jc** $_{-20\,°C}$

In general, the critical strain energy release rate (J_c) decreases by increasing the temperature from -20 °C to 19 °C. Table 5.12 shows J_c results and their change percentages from -20 °C to 19 °C. Except for mix A in STOA condition which increased by 19%, J_c for other specimens decreased from 26% to 70%. It is notable that J_c in mix B has been more affected by temperature in comparison with mix A due to its higher bitumen content.

5.3.2.3 Effect of temperature on Fracture toughness (K_{IC})

Figures 5.15 and 5.16 illustrate the value of fracture toughness (K_{IC}) for mix A and B in different temperatures. These figures help to better compare fracture toughness (K_{IC}) in different conditions.

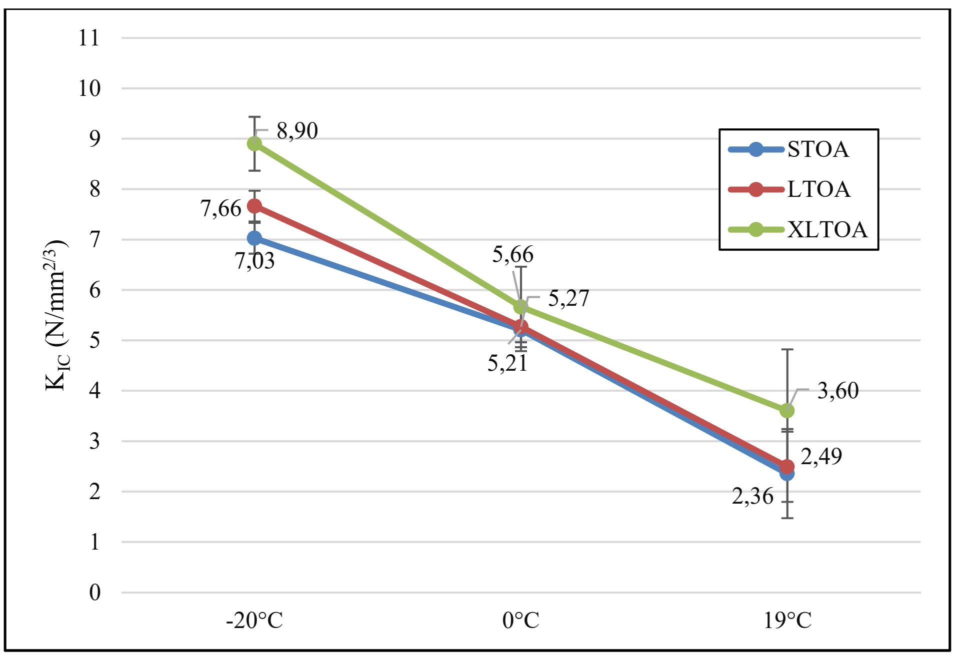

Figure 5. 22 K_{IC} results for mix A in different temperatures

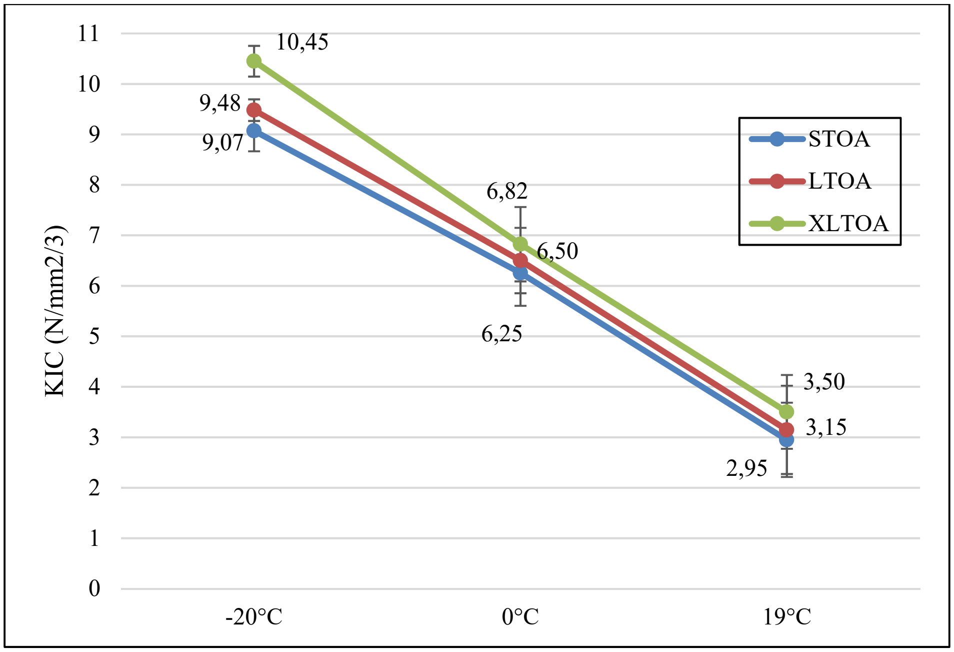

Figure 5. 23 K_{IC} results for mix B in different temperatures

Fracture toughness (K_{IC}) decreased by increasing the temperature from -20 °C to 19 °C. This trend is similar for both mixtures, however, mix B has a relatively higher K_{IC} at each temperature. Fracture toughness (K_{IC}) values for both mixes are converted to Table 5.13 to better highlight differences in various temperatures.

Table 5. 13 K_{IC} results in different temperatures

Temperature	**Mix Design**	**K_{IC} (N/mm$^{2/3}$)**			**Change percentage[1]**
		-20 °C	0 °C	19 °C	
STOA	Mix A	7.03	5.21	2.36	-66%
	Mix B	9.07	6.25	2.95	-67%
LTOA	Mix A	7.66	5.27	2.49	-67%
	Mix B	9.48	6.50	3.15	-67%
XLTOA	Mix A	8.90	5.66	3.60	-60%
	Mix B	10.45	6.82	3.50	-66%

[1] Change percentage = ($\mathbf{K_{IC}}$ $_{19\,°C}$ - $\mathbf{K_{IC}}$ $_{-20\,°C}$) / $\mathbf{K_{IC}}$ $_{-20\,°C}$

Given the change percentage column in Table 5.13, K_{IC} decreased at the relatively same rate for all samples. Based on Table 5.13, it can be concluded that aging does not have noticeable effect on K_{IC}.

5.3.3 Load-displacement curves

Load-displacement curve as the main output of SCB test is the reference for all parameters along with the specimen's geometry. As a result, SCB applied load and its corresponding vertical displacement are the foundation of fracture analysis in this method. Besides, the behaviour of samples in various temperatures and aging conditions can be monitored by comparing load-displacement curves. The fact that mixtures in this study behave more brittle at lower temperatures and aged conditions, can be seen in load-displacement curves.

Load-displacement curves for mix A and B in different aging conditions and temperatures are illustrated in figures 5.26 to 5.34.

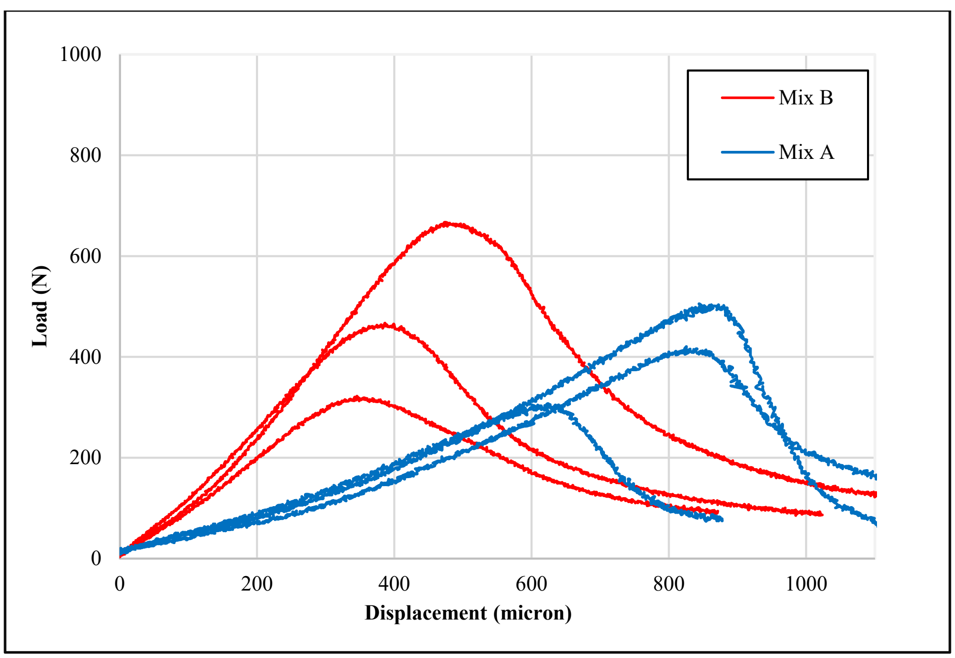

Figure 5. 24 SCB load-displacement curve (STOA at 19 °C)

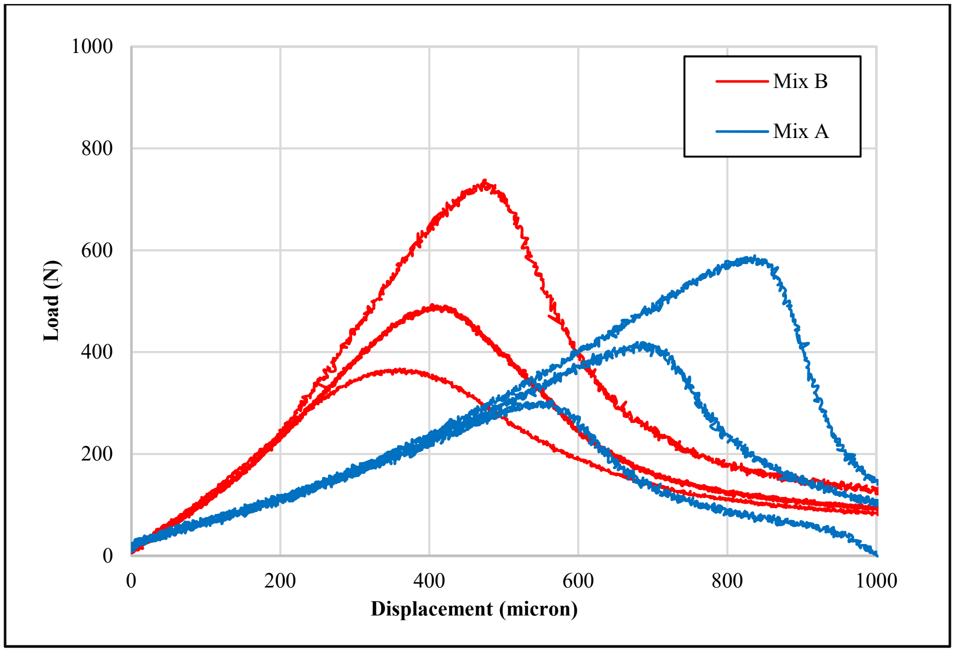

Figure 5. 25 SCB load-displacement curve (LTOA at 19 °C)

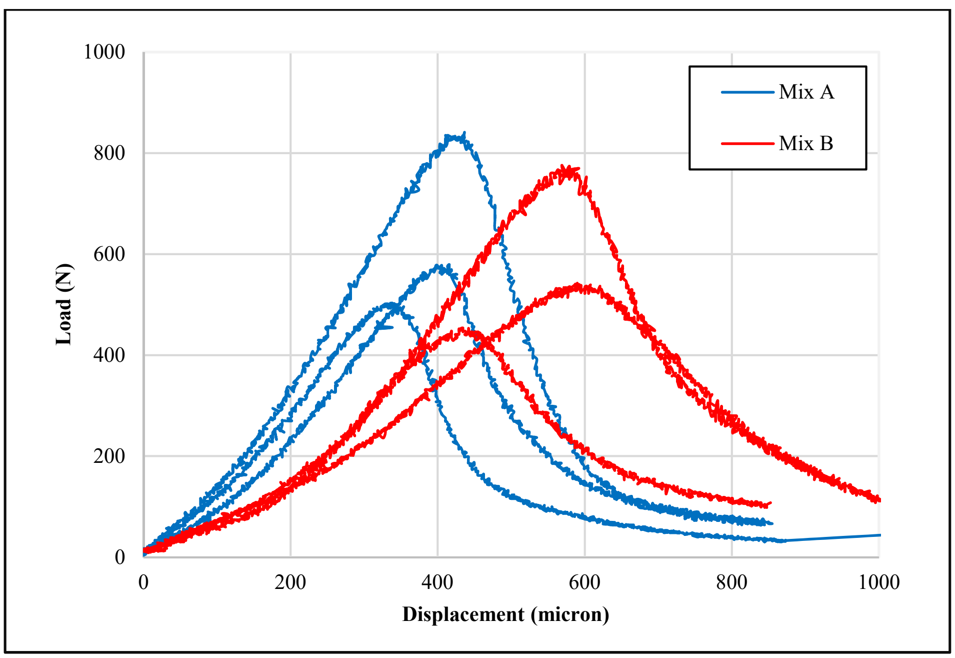

Figure 5. 26 SCB load-displacement curve (XLTOA at19 °C)

According to the last three figures, the post peak behaviours of mix A (blue curves) are showing more brittle performances at 19 °C. This fact can be seen from the higher slopes in blue curves after reaching the highest load. This phenomenon shows the fact that mix A with less binder content, behaved more brittle than mix B. in addition, there is considerable difference in displacement of mix A and B in peak load at STOA and LTOA conditions. Due to the fact that mix B has higher binder content, it is expected to have more displacement before failure. However, the curves show different behaviour which indicate another reason for this phenomenon. Based on the test results in this study there is not enough information to analyze the reason for this displacement difference.

The load-displacement curves for the mixtures at 0 °C are presented in Figures 5.28 to 5.30.

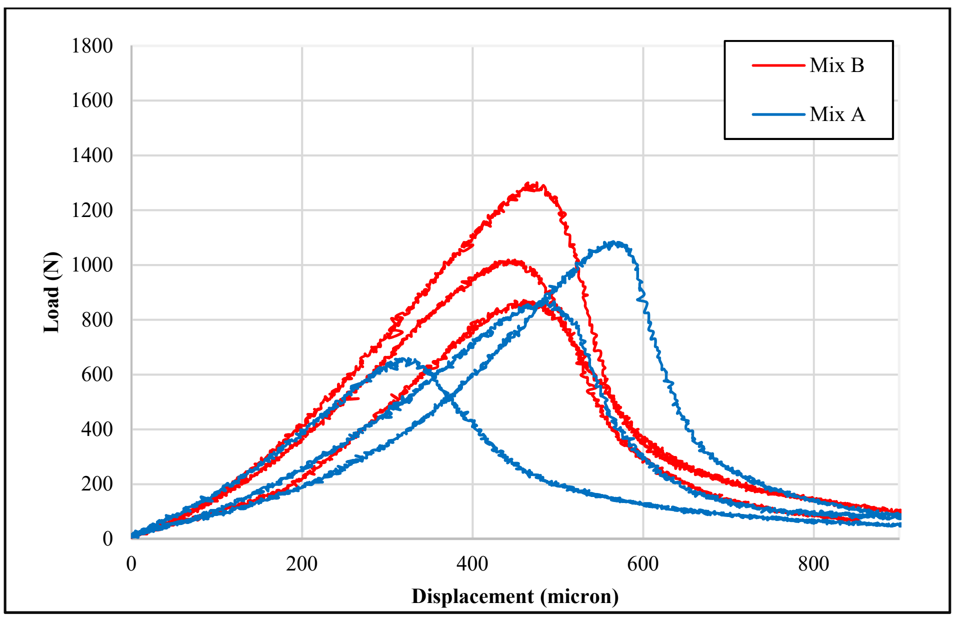

Figure 5. 27 SCB load-displacement curve (STOA, 0 °C)

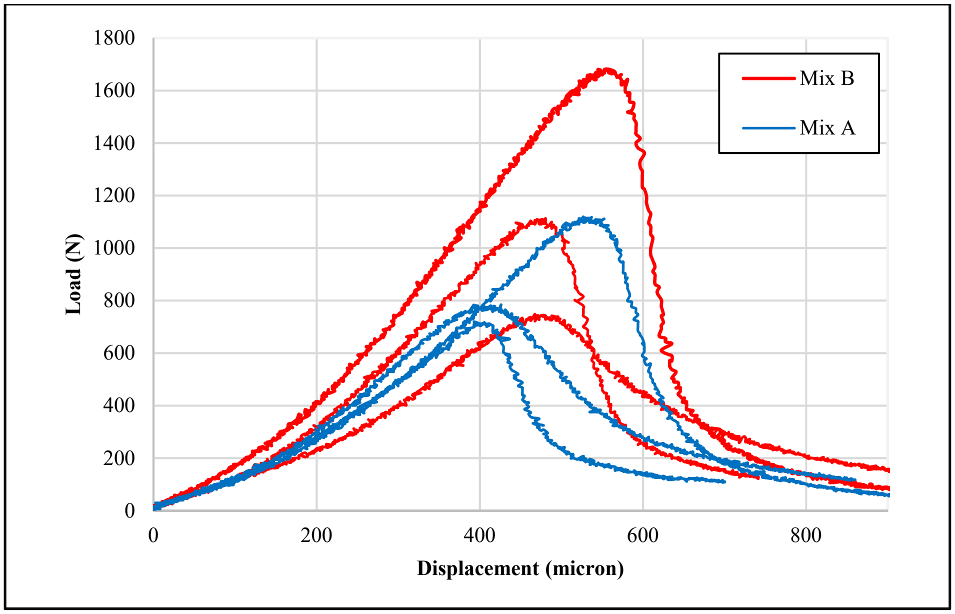

Figure 5. 28 SCB load-displacement curve (LTOA, 0°C)

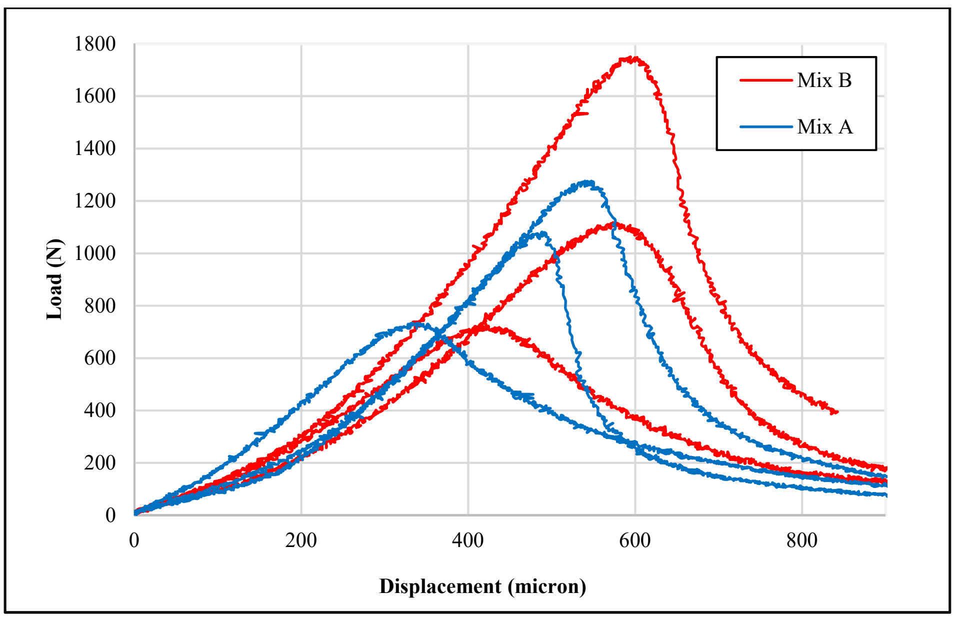

Figure 5. 29 SCB load-displacement curve (XLTOA, 0°C)

Based on load-displacement curves at 0 °C, both mixtures show more strength to the applied load. In terms of brittleness behaviour, both mixtures are relatively similar, and more brittle behaviour can be observed for aged specimens in Figures 5.29 and 5.30. However, statistical analysis was presented earlier in this chapter.

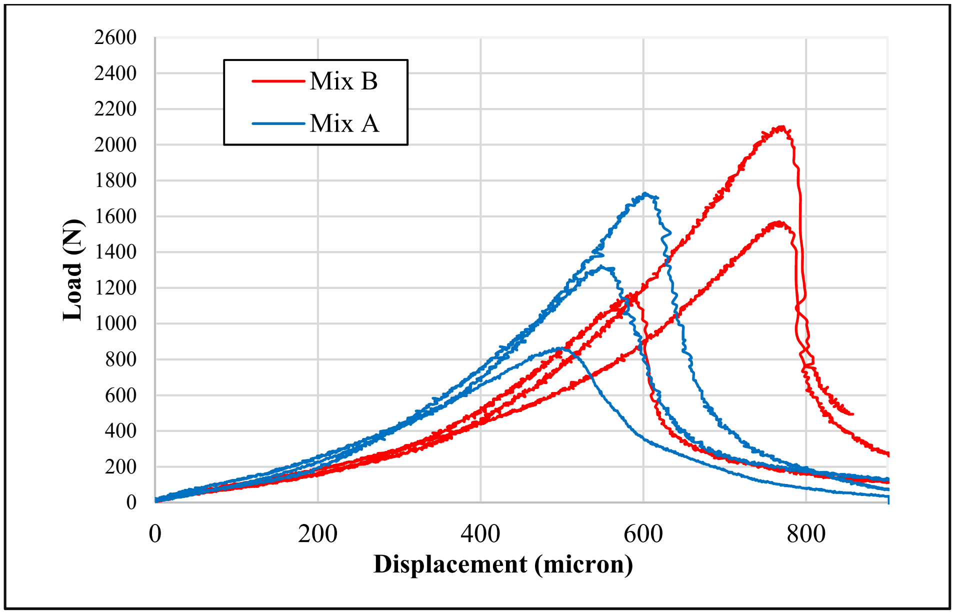

Figure 5. 30 SCB load-displacement curve (STOA, -20 °C)

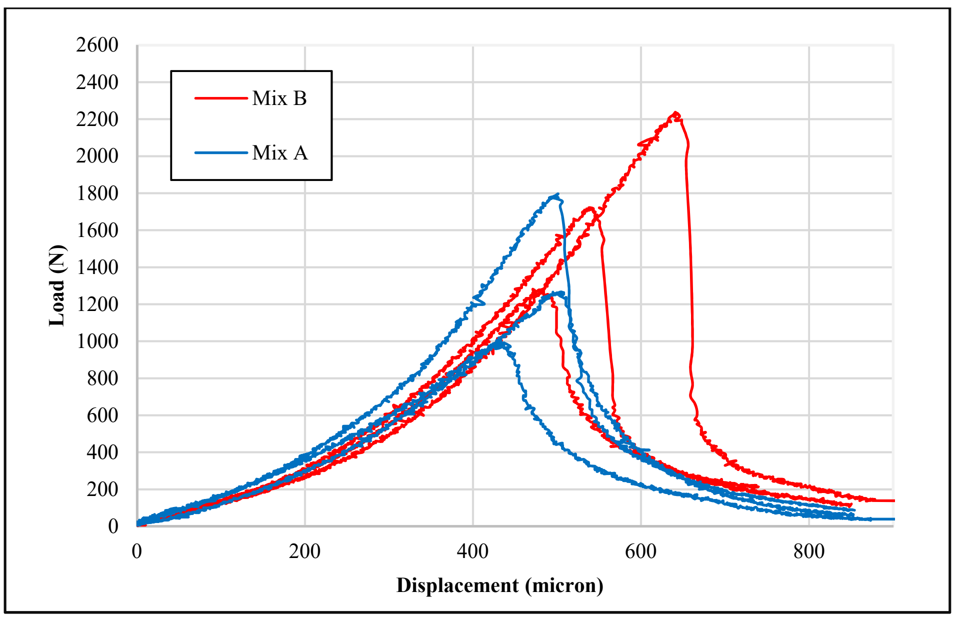

Figure 5. 31 SCB load-displacement curve (LTOA, -20 °C)

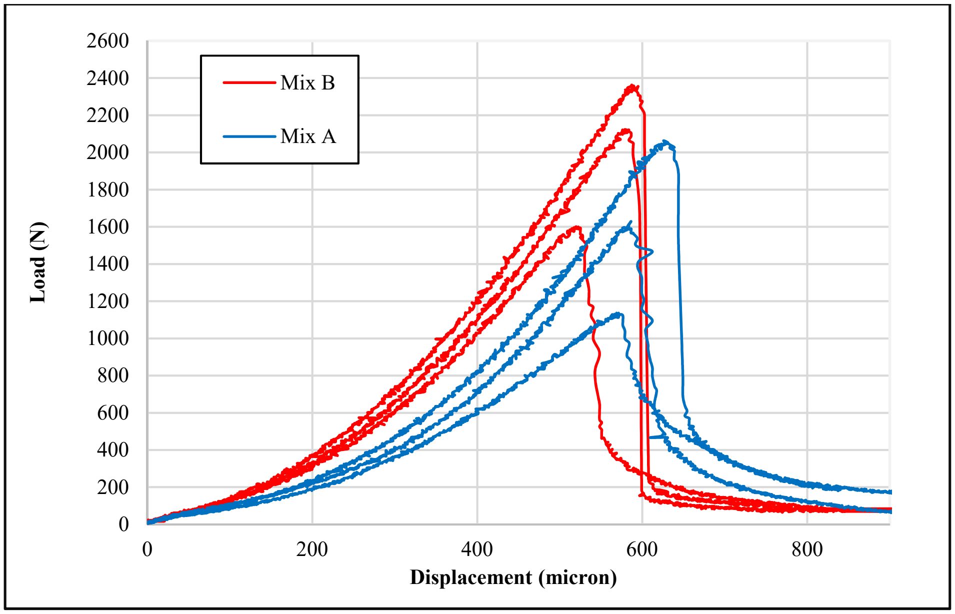

Figure 5. 32 SCB load-displacement curve (XLTOA, -20 °C)

Load-displacement curves at -20 °C show extreme brittleness behaviour for the mixtures. According to the figures, mix B with 2.2% residual bitumen shows different behaviour than mix A in all aging conditions. As the figures present, the curves for mix B drops after reaching the peak load which means it behaves relatively more brittle than mix A. in addition, both mixtures tend to behave more brittle by increasing the oven aging duration from STOA to XLTOA. It can be observed from Figure 5.33 that XLTOA has more effect on the performance of mix B which contains more residual bitumen. The reason for this fact is that new bitumen ages faster than RAP bitumen and that new bitumen are more sensitive to change in temperature and to aging than old bitumen.

5.4 Data quality

As mentioned earlier in this chapter, data quality is evaluated using coefficient of variation (COV). Since three parameters are used in data analysis, coefficient of variation was measured for the rationale data of these parameters separately to evaluate the variation in data sets. Given that SCB parameters discussed in this study are based on axial load and displacement, COV is measured for maximum load and the area under load-displacement curve (U). Therefore, the coefficient of variation (COV) for maximum load is presented in Figure 5.33 for mix A and B.

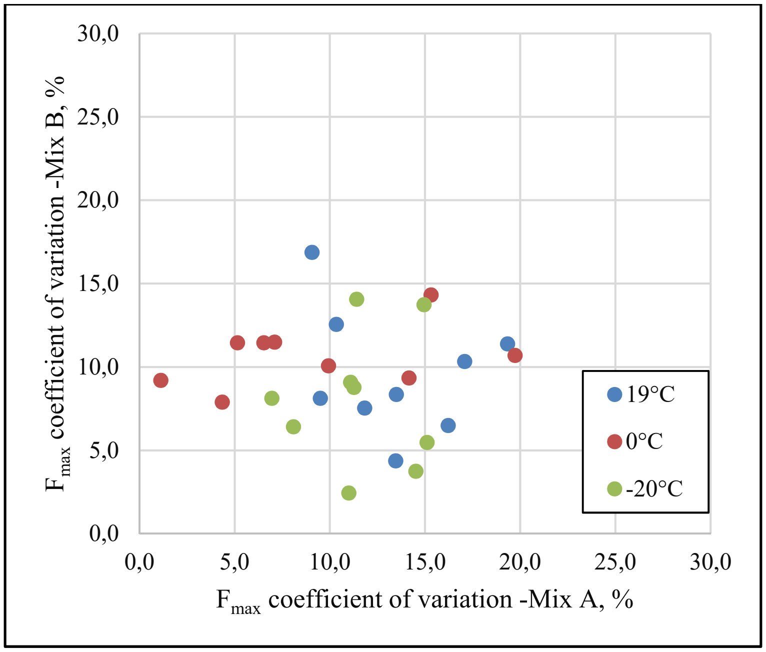

Figure 5. 33 Coefficient of variation for the maximum load (SCB)

According to Figure 5.24, coefficient of variation (COV) for maximum load are less than 20%. Since maximum stress at failure (σ_{max}) and fracture toughness (K_{IC}) is measured based on maximum load and specimen dimensions, the accuracy of data presented in Figure 5.24 can be applied to these parameters.

Critical strain energy release rate (J_c) as another parameter in this study is calculated based on the area under the load-displacement curve. Strain energy to failure (U) which is defined as the area under load-displacement curve is related to vertical displacement and its corresponding load. The coefficient of variation (COV) for Strain energy to failure (U) is presented in Figure 5.34. Each value in this figure represents COV for a data set including four specimens.

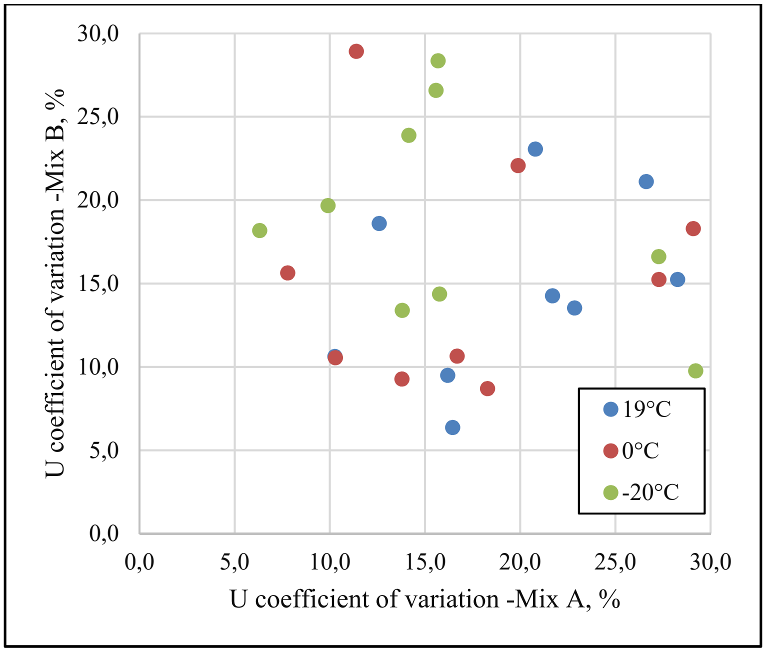

Figure 5. 34 Coefficient of variation for Strain energy to failure (SCB)

According to Figure 5.25, coefficient of variation (COV) for Strain energy to failure (U) are less than 30%.

5.5 Summary

In this chapter, the results of water evaporation in the curing process, indirect tensile strength test, and semi-circular bending test were analyzed.

- **Curing:** Assuming two types of mixture and specimen size, the amount of residual water was recorded for the sample specimens. According to the data, residual water for both mixes (Mix A and B) are relatively the same at the end of the curing process. Based on Figure 5.3 there is 0.04 percent difference between the residual water percentages of mix A and B at the end of curing. In both cases, mix B had 0.04% more residual water in comparison with mix A on day 14. Thus, binder content does not have an important effect on water evaporation rate during curing process. Since there was a

0.18 difference in the percentage of residual water for 100mm and 150mm specimens, a bilinear regression was used to estimate water evaporation after 14 days of curing. According to the data from Table 5.2, it would have taken at least 23 more days to reach the same percentage of residual water for 100mm and 150mm specimens.

- **Indirect tensile strength test:** Three types of mix designs with different binder contents were used for ITS test. The percentages of binder content were assumed to be 2.2, 1, and 0 for mix A, B, and C, respectively. Indirect tensile strength test was performed based on ASTM D6931-17. In addition, the test was conducted in saturated condition to investigate moisture sensitivity of specimens. Therefore, the effect of aging and binder content on the tensile strength of specimens was evaluated using the results of ITS.

  As a result, oven aging leads to improving strength in mix A and B, however, mix C showed a fluctuation in ITS value with an increasing trend over aging. The same behaviour was observed in wet condition for all mixes, but with slightly lower values of ITS. It can be seen that the mixture with higher binder content (mix B) was more affected by oven aging in ITS test. In other words, mix B has the highest increase in ITS results from STOA to XLTOA in comparison with other mixtures. In addition, mix B which has the highest binder content showed less sensitivity to moisture. Increasing binder content leads to having more aging effects on the performance of mixtures because the proportion of unaged bitumen is increased. Then, the process of aging the added bitumen has more effect on the performance of mixture.

- **Semi-circular bending test:** Due to the preparation measures for SCB test, only mix A and B were proposed for this test. The test was conducted according to ASTM 8044-16. The results were analyzed considering different temperatures, aging, and binder contents. Since the mixtures have different behaviour depending on test temperature and binder content, both elastic plastic fracture mechanics (EPFM) and linear elastic fracture mechanics (LEFM) approaches were considered to analyze the data. Thus, maximum load, critical strain energy release rate (J_C), and fracture toughness (K_{IC}) are the main parameters in analyzing SCB test results. Primarily, it was observed that maximum load increases by increasing oven aging duration or decreasing test temperature. J_C which represents the required energy to initiate the crack was decreased by increasing aging duration for mix A at test temperatures of 19 °C and 0 °C, but it increased at -20 °C. However, J_C does not follow the same trend for Mix B. At temperatures of 19 °C and 0 °C, J_C fluctuated around constant values in various aging conditions, but it increased by increasing aging duration at -20 °C. Fracture toughness (K_{IC}) as a parameter to evaluate the material's resistance against fracture, is determined based on linear elastic fracture mechanics (LEFM). By increasing aging, fracture toughness rises. Also, it decreases by increasing the test temperature. In addition, increasing binder content causes higher fracture toughness (K_{IC}). According to load-displacement curves, the behaviour of mixtures is going to be more brittle by decreasing temperature and this can be monitored in Figures 5.26 to 5.34.

CONCLUSION

The main goal of this thesis is to investigate the failure mechanism of cold recycled mixtures in various aging conditions and temperatures. Mixture preparation, curing, and aging was designed to highly simulate field conditions. To test the performance of mixtures, indirect tensile strength and semi-circular bending tests were selected. to evaluate the performance of asphalt mixtures, aging, temperature, and binder content were taken as independent variables to monitor related parameters in each laboratory test as the dependent variables. The following conclusions are drawn:

- Oven aging at 60̊C can affect the performance of the asphalt mixture made of 100% RAP. In other words, the aged binder within RAP can be affected over time and change the performance of mixtures. This fact was tested using indirect tensile strength test with mix C (100% RAP with no additives). Thus, the assumption to consider RAP as black rock is not accurate. In other words, if RAP could be black rock, it would not have enough cohesion to make a specimen and it would not be sensitive to aging.
- Aging has more influence on the mix with higher binder content in a way that ITS_{dry} of mix B increased 50% from STOA to XLTOA while mix A raised 28%.
- Mixtures with higher binder content (Mix B) showed better overall moisture resistance concerning different aging conditions.
- The maximum load that specimens can withstand in SCB test increases with longer aging duration.
- Higher binder content causes higher maximum loads in SCB test.
- Aging leads to decreasing J_c at 19°C, while at lower temperature it does not follow any trends. Based on the concept of J_c (EPFM), this parameter cannot be used at lower temperatures. Thus, another parameter (K_{IC}) is required to analyze the performance of mixtures in different temperatures.
- K_{IC} increased as an effect of lower test temperature and more aged specimens.
- Aging has more effect on fracture toughness at 19 °C than lower temperatures.
- Mixtures behaved more brittle by increasing aging and decreasing temperature.

RECOMMENDATIONS

1. The first recommendation is to practice the experiments in this study on both laboratory made specimens and field samples with the same materials and mix design. This helps us to compare the results in order to evaluate the accuracy of laboratory compaction, curing, and aging protocols which are supposed to represent field conditions.
2. Conducting laboratory tests on the mixture after curing to observe the effect of curing on different mixtures.
3. More RAP source helps to dominate the results of experimental tests and establish a more confident conclusion.
4. Different aging protocols and monitoring water evaporation during oven aging might lead to different results.
5. Using different mix designs helps better clarify the effect of mix components on the performance of asphalt mixture.

ANNEX I

Table A-I. 1 Complete SCB results for Mix A

Aging	Temperature	Notch depth (mm)	F_{max} (N)	Average F_{max} (N)	U	U (average)	dU/da	b (mm)
short-term	19 °C	25	505.5	574.4	0.00059212	0.0005657	-0.0337	58.2
			565.7		0.00043222			57.6
			600.2		0.00071229			57.7
			626.0		0.00052616			58.2
		32	264.4	349.6	0.00042734	0.00033871		59
			383.2		0.00029386			57.3
			330.7		0.00036876			56.9
			420.2		0.0002649			57.5
		38	256.7	302.3	0.00027266	0.00025507		58.1
			307.5		0.00029055			58
			319.5		0.00023639			57.4
			325.5		0.00022067			57.6
long-term		25	589.5	579.3	0.00061341	0.0005885	-0.0266	58.6
			529.9		0.00050215			58.8
			685.4		0.00071229			57.8
			512.4		0.00052616			57.2
		32	419.4	413.2	0.00051895	0.0003927		58.2
			415.1		0.00030923			59.1
			477.1		0.0003848			57.9
			341.0		0.00035782			56.9
		38	273.9	307.5	0.0002106	0.00024703		57.7
			345.3		0.00026961			57.4
			304.9		0.00025396			58.4
			306.0		0.00025396			58.6
Extra long-term		25	708.7	858.3	0.0003867	0.00048577	-0.0208	58.6
			1059.9		0.00067313			58.8
			823.2		0.00046818			57.8
			841.3		0.00041505			57.2
		32	469.3	526.8	0.00018428	0.00023415		58.2
			531.3		0.00022881			59.1
			612.3		0.00027624			57.9
			494.3		0.00024728			56.9
		38	502.9	488.9	0.00020098	0.00022145		57.7
			385.8		0.0001534			57.4
			488.3		0.00022831			58.4

			578.7		0.00030311			58.6
short-term	0 °C	25	1011.0	1111.4	0.00132481	0.00099824	-0.0391	57.3
			1085.7		0.00056974			58.7
			1168.4		0.00079793			56.8
			1180.4		0.00130046			57.4
		32	851.6	875.7	0.00096318	0.00072393		57.1
			838.6		0.00095794			58.2
			940.2		0.00046401			56.8
			872.2		0.00051058			56.6
		38	609.7	674.9	0.00074593	0.00049056		57.6
			662.2		0.00055959			57.2
			658.7		0.00027929			59
			768.9		0.00037743			57
long-term		25	1173.5	1121.7	0.0013428	0.00102761	-0.0391	59.1
			1056.5		0.00143736			57.4
			1119.3		0.00058221			56.8
			1137.4		0.00074805			57.9
		32	1053.9	858.2	0.00081302	0.00065484		59.1
			817.2		0.00104567			57.4
			777.5		0.00044772			56.8
			784.4		0.00031296			57.9
		38	722.7	714.4	0.00073689	0.00052488		57.3
			709.6		0.00071146			56.8
			719.5		0.00033345			58.1
			705.6		0.00031771			56.9
Extra long-term		25	1276.0	1275.4	0.00063996	0.00066435	-0.0275	59.2
			1106.4		0.00048802			57.7
			1525.7		0.00091231			57.6
			1193.4		0.00061709			57.2
		32	1081.5	1010.0	0.00054484	0.00058805		59
			1022.9		0.0005046			58.9
			732.8		0.00033183			57.2
			1202.8		0.00097092			57.1
		38	661.3	669.3	0.00030163	0.0003003		58.6
			633.8		0.0002525			59
			732.8		0.00033132			57.9
			649.3		0.00031575			57.6
short-term	-20°C	25	1730.6	1545.6	0.00096814	0.0007464	-0.0282	58.2
			1350.9		0.00074931			56.9
			1465.0		0.00081894			58.4
			1636.0		0.0004492			56.8
		32	1324.2	1165.0	0.00066497	0.00063015		57.1
			1281.2		0.00064422			57.3
			942.8		0.00063862			57.9
			1111.6		0.00057281			57
		38	864.5	886.2	0.00038623	0.00037547		57.1
			997.0		0.00041028			56.9

			760.3		0.00029736			57.4
			923.0		0.00040799			56.7
long-term		25	1796.9	1747.2	0.00083765	0.00092031	-0.0358	58.2
			1632.4		0.0008289			56.8
			1554.1		0.00113561			58.4
			2005.2		0.00087907			56.8
		32	1409.4	1265.0	0.00075073	0.00071604		57.9
			1192.5		0.00079942			57.3
			1191.6		0.00065455			57.2
			1266.5		0.00065948			58.8
		38	993.7	940.9	0.00051079	0.00045203		57.1
			998.8		0.00049672			56.9
			903.2		0.00042145			57.4
			867.9		0.00037914			56.7
Extra long-term		25	1916.6	1997.7	0.00119294	0.00119417	-0.0518	59.2
			2367.7		0.0013514			57.7
			2063.8		0.00130137			57.6
			1642.8		0.00093097			57.2
		32	1722.8	1526.3	0.00103566	0.00080712		59
			1213.2		0.00053583			58.9
			1629.9		0.0009255			57.2
			1539.5		0.00073147			57.1
		38	1011.7	1104.7	0.00041075	0.00052268		58.6
			1264.8		0.00057508			59
			1005.7		0.00051452			57.9
			1136.6		0.00059036			57.6

Table A-I. 2 Complete SCB results for Mix B

Aging	Temperature	Notch depth (mm)	F_{max} (N)	Average F_{max} (N)	U	U (average)	dU/da	b (mm)
short-term	19	25	713.8	716.0	0.0005218	0.0004843	-0.0241	58.2
			620.0		0.0003537			57.6
			886.9		0.0006162			57.7
			643.3		0.0004454			58.2
		32	497.8	491.5	0.0002956	0.0002627		59
			567.5		0.0002769			57.3
			440.9		0.000209			56.9
			459.9		0.0002691			57.5
		38	394.4	337.8	0.0001738	0.0001741		58.1

			311.8		0.0002152		58
			299.7		0.000136		57.4
			345.4		0.0001715		57.6
long-term		25	676.0	721.8	0.0004874	0.0004791		58.6
			745.7		0.0005398			58.8
			738.0		0.0004391			57.8
			727.6		0.0004501			57.2
		32	583.8	533.7	0.000316	0.0002769	-0.0219	58.2
			538.2		0.000301			59.1
			475.4		0.0002518			57.9
			537.4		0.0002386			56.9
		38	355.7	385.4	0.0002263	0.0001969		57.7
			372.9		0.0001967			57.4
			428.9		0.0001852			58.4
			384.1		0.0001793			58.6
Extra Long-term		25	775.8	768.1	0.0004424	0.0004777		58.6
			685.4		0.0003944			58.8
			738.0		0.0004496			57
			873.1		0.0006244			57.01
		32	616.6	579.1	0.0004477	0.0004249	-0.0227	58.2
			542.5		0.0004018			59.1
			617.1		0.0004487			57.9
			540.2		0.0004011			56.9
		38	426.3	460.3	0.0002017	0.0002371		57.7
			498.6		0.0002812			57.4
			462.5		0.000251			58.4
			453.8		0.0002143			58.6
short-term	0	25	1219.2	1300.8	0.0008036	0.0008056		57.3
			1510.2		0.0008926			58.7
			1172.7		0.0008049			56.8
			1301.0		0.0007211			57.4
		32	1224.4	1070.9	0.0009051	0.0007046	-0.0507	57.1
			937.7		0.0007181			58.2
			1103.0		0.0006654			56.8
			1018.6		0.0005298			56.6
		38	871.4	818.0	0.0004337	0.0005068		57.6
			708.7		0.0004217			57.2
			890.3		0.0005677			59
			801.7		0.0006042			57
long-term		25	1682.4	1655.7	0.0011122	0.0010943	-0.0543	59.1
			1464.6		0.0009277			57.4
			1726.3		0.0011884			56.8

			1749.5		0.0011488			57.9
		32	1234.7	1055.6	0.000683	0.0006474		59.1
			1113.3		0.0006625			57.4
			987.6		0.0006977			56.8
			886.9		0.0005465			57.9
		38	748.3	712.1	0.0004359	0.0003918		57.3
			620.0		0.0003471			56.8
			713.0		0.0003901			58.1
			767.2		0.000394			56.9
Extra Long-term		25	1504.2	1581.2	0.0010579	0.0012421	-0.056	59.1
			1419.8		0.001485			57.4
			1651.4		0.0013087			56.8
			1749.5		0.0011168			57.9
		32	1117.6	1104.9	0.0006773	0.0007496		59.1
			1264.8		0.0010724			57.4
			1042.7		0.0006318			56.8
			994.5		0.0006168			57.9
		38	722.5	846.9	0.0003343	0.0005192		57.3
			870.5		0.0006023			56.8
			837.8		0.0005712			58.1
			956.6		0.0005691			56.9
short-term	-20	25	2069.8	2047.6	0.0015015	0.0013195	-0.0571	59.2
			2031.1		0.001284			57.7
			1987.1		0.0011983			57.6
			2102.5		0.0012943			57.2
		32	1611.8	1457.3	0.0008661	0.0008232		59
			1570.5		0.0009996			58.9
			1477.5		0.0007846			57.2
			1169.3		0.0006426			57.1
		38	1135.7	1197.9	0.0005242	0.0005826		58.6
			1134.0		0.0004307			59
			1168.4		0.000619			57.9
			1353.5		0.0007566			57.6
long-term		25	2237.7	2062.5	0.0013301	0.0011623	-0.0464	59.2
			1759.9		0.0009954			57.7
			1879.5		0.0010444			57.6
			2372.8		0.0012793			57.2
		32	1623.8	1688.2	0.0009545	0.0009494		59
			1824.4		0.0011999			58.9
			1582.5		0.0007539			57.2
			1722.0		0.0008893			57.1
		38	1299.3	1169.7	0.0006342	0.0005529		58.6

			1163.2		0.0005963			59
			1072.0		0.0004991			57.9
			1144.3		0.0004821			57.6
Extra long-term		25	2342.7	2308.3	0.0022972	0.0016455	-0.0743	58.2
			2403.8		0.0016642			56.8
			2123.2		0.0012706			58.4
			2363.4		0.00135			56.8
		32	1678.9	1687.1	0.0008445	0.0009167		57.9
			1740.1		0.0010613			57.3
			1728.0		0.0010232			57.2
			1601.5		0.0007377			58.8
		38	1462.0	1358.0	0.0008218	0.0006912		57.1
			1281.2		0.0005696			56.9
			1225.2		0.0005004			57.4
			1463.7		0.0008728			56.7

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