Evaluating Fatigue Resistance of the Fiber-reinforced 100% RAP Content Asphalt Mixture Rejuvenated with Waste Vegetable Oil

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EVALUATING FATIGUE RESISTANCE OF THE FIBER-REINFORCED 100% RAP 
(RECLAIMED ASPHALT PAVEMENT) CONTENT ASPHALT MIXTURE 
REJUVENATED WITH WASTE VEGETABLE OIL 
A dissertation submitted in partial fulfillment of 
the requirements for the degree of 
DOCTOR OF PHILOSOPHY 
in 
CIVIL ENGINEERING 
by 
Farshad Haddadi 

2022
To:  Dean John L. Volakis  
     College of Engineering and Computing

This dissertation, written by Farshad Haddadi, and entitled Evaluating Fatigue Resistance of the Fiber-reinforced 100% RAP (Reclaimed Asphalt Pavement) Content Asphalt Mixture Rejuvenated with Waste Vegetable Oil, having been approved in respect to style and intellectual content, is referred to you for judgment.

We have read this dissertation and recommend that it be approved.

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Nipesh Pradhananga

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Atorod Azizinamini, Co-Major Professor

_______________________________________
Hesham Ali, Co-Major Professor

Date of Defense: March 29, 2022

The dissertation of Farshad Haddadi is approved.

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Dean John L. Volakis  
College of Engineering and Computing

_______________________________________
Andrés G. Gil  
Vice President for Research and Economic Development  
and Dean of the University Graduate School

Florida International University, 2022
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ABSTRACT OF THE DISSERTATION

EVALUATING FATIGUE RESISTANCE OF THE FIBER-REINFORCED 100% RAP (RECLAIMED ASPHALT PAVEMENT) CONTENT ASPHALT MIXTURE REJUVENATED WITH WASTE VEGETABLE OIL

by

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Florida International University, 2022

Miami, Florida

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The use of reclaimed asphalt pavement (RAP) in the pavement industry continues to grow as it is an economically and environmentally beneficial proposition. However, a survey conducted by the Federal Highway Administration shows that the average RAP content in the hot mix used in the United States is only 10–20%, even though specifications allow up to 30%. The primary performance drawback of using a high percentage RAP is cracking distresses. This research is an effort to investigate whether basalt fiber and waste vegetable oil (WO) can improve the low and intermediate temperature cracking behavior of a 100% RAP mixture. Bending beam rheometer (BBR) and the indirect tension asphalt cracking (IDEAL-CT) tests were conducted to measure cracking performance at low and intermediate temperatures, respectively. The rutting performance was controlled using an asphalt pavement analyzer (APA). Four control mixtures and 18 types of mixture consisting of two nominal maximum aggregate sizes (NMAS), three different fiber lengths (3, 6, and 12mm), and three fiber content (0.10, 0.30, and 0.60% by the weight of the mixture), were prepared. It was observed that for mixtures containing fiber, the inflection point at the post-peak part of the load-displacement curve at 25°C occurs at 69% percent
of the peak load. Also, a novel method to determine mixture thermal cracking using BBR
test results was proposed; \( \Delta T_c \) concept, formerly used for binder thermal evaluation, was
utilized for mixture thermal characterizations. Results indicate that \( \Delta T_c \) and rutting depth
are not significantly affected by fiber length (FL). On the other hand, \( CT_{\text{index}} \) was found a
function of both fiber content (FC) and FL. Results showed that NMAS-to-FL ratio of
approximately 1.6, results in the highest \( CT_{\text{index}} \) for all basalt FC. Also, using proper FC
and FL, the fatigue and thermal cracking resistance could be improved up to 110 and 136%,
respectively. Finally, a performance-based mix design procedure is prepared for designing
a 100%RAP mixture containing fiber and rejuvenator.
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CHAPTER I. INTRODUCTION

1.1 Background

Asphalt recycling is one of the current challenges in road construction and asphalt industries. Due to the increase in the price of asphalt and the costly nature of asphalt operations, road and transportation departments and researchers are constantly looking for a method for optimal and better use of more percentage of RAP. However, this goal faces many challenges including the lack of a reliable reference or standard for classification and analysis of RAP materials, the lack of an appropriate and codified mixture plan, and finally, the poor performance of these mixtures. Adding RAP which contains aged asphalt significantly increases the stiffness of the mixture. This increase could be desirable for performance of pavement at high temperature. However, at low and medium temperatures it can cause thermal and fatigue cracks earlier during the service life of the pavement.

Thermal cracks are known as one of the most important demolitions of asphalt paving. Creation and propagation of cracks in asphalt paving usually occur due to environmental factors (such as temperature cycles of cooling and heating resulting from daily or seasonal changes of temperature as well as gradual changes in mechanical properties of asphalt over time). This phenomenon can be considered as the result of the weaknesses of asphalt mixtures against stresses caused by the propagation and contraction of asphalt due to cycles of heat and cold.

The use of asphalt modified with polymer or asphalt with desirable degrees of functionality is considered a method to improve the resistance of asphalt mixtures against cracks. However, since the amount of new asphalt added to recycled asphalt mixtures with high percentages of RAP is not too high, moreover since the new asphalt and the old asphalt
existing in the RAP materials do not mix completely, the use of the aforementioned methods to improve asphalt resistance against cracks can cause various problems. Therefore, the only way possible is the use of additives that are added to the mixture in dry form. Among these additives, are the fibers.

The primary purpose of this project is to investigate the cracking behavior of basalt fiber-reinforced 100% RAP. The first application of basalt fiber was in the 1990s and since then become a popular type of fiber in the pavement industry. This is primarily because of its better mechanical performance and higher work temperature, in addition to its higher-level durability [1].

1.2 Problem Statement

As mentioned earlier, the most important associated with using a high percentage of RAP in the new mixture is reduction in cracking resistance at low and intermediate temperatures. Many researchers are trying to address these problems which vital of them are mentioned in the literature review. The results confirmed that the increasing percentage of the RAP could reduce the cracking resistance of asphalt [2].

Various types of rejuvenators and modifiers are introduced to improve the cracking performance of asphalt mixtures containing a high percentage of RAP. Zaumanis et al. used six different rejuvenators to evaluate their effect on the performance of the 100% RAP mixture. Results showed that the performance grade (PG) of aged binders is recoverable using waste vegetable oils, waste vegetable grease, and organic oils [3]. Among the various type of rejuvenators, it is concluded that using bio-based oils is not only superior to other types of generic rejuvenators but also promotes pavement sustainability. Although bio-based oils are highly susceptible to aging, waste vegetable oil (WO) is much less
susceptible due to undergoing the high temperature during the cooking process which reduces lots of volatile components of oil [3]. The availability of the WO could be another advantage of this rejuvenator. There are roughly 32 million gallons of WO generated in the United States alone each year [4]. Although the rejuvenators have many positive impacts on the recycled mixture, the fracture work of rejuvenated mixtures containing 100% RAP is significantly lower than that of the virgin mixture [5].

Previous studies have used polymer-modified virgin binders in combination with rejuvenators to restore the performance of recycling asphalt mixture. However, as the content of the RAP increases, it is impossible to cooperate enough percentage of the polymer in the mix especially when the percentage of RAP is as high as 100%. Furthermore, it is not possible to use polymer modified binders when hot in-place recycling is used. Thus, using a modifier that could be directly added to the mixture can be an excellent option to improve 100% RAP mixture fatigue resistance. Based on the previous research, it is shown that it is possible to add fibers directly to the asphalt mixture and it has the potential to enhance the rutting and cracking performance [6]. Therefore, fiber can be a suitable replacement for polymer modifiers when it is not possible to use them in a 100% RAP mixture. In this study, the ability of fibers when using it in the 100% RAP mixture rejuvenated with WO was evaluated.

1.3 Objectives and Hypotheses

The objectives of this research are as follows:

1. Evaluate the effect of FC and FL on intermediate- and low-temperature cracking performances.
2. Proposing a method to evaluate the resistance of asphalt mixtures against cracking using different percentages of FC and FL.

It is speculated that when using fibers in the mixture, the tension strain is resisted both by the bond strength of the binder and by the friction between the particle and fiber (Figure 1). On the other hand, by increasing the compressive stress the interlock and the friction between fiber and aggregates increase.

![Figure 1 Role of fiber in the asphalt mixture [7]](image)

The hypotheses of the research are as follows:

1. The use of fibers in 100%RAP asphalt mixtures can increase the resistance of the mixture against cracking and rutting.

2. FL and FC affect the resistance of the mixture at different temperatures

1.4 Organization of Dissertation

This dissertation includes six chapters. Chapter 1 is an introduction that defines the objectives of the study and the knowledge gap. Chapter 2 is a comprehensive literature review. Chapter 3 describes the methodology and the tests that were conducted to perform this study. Chapter 4 provides information about the source of material, type of material, and samples preparation process. Chapter 5 presents performance tests results and analyses. Finally, Chapter 6 is the conclusion of this study. Also, proposed methods and recommendations for future studies are presented in this chapter.
CHAPTER II. LITERATURE REVIEW

2.1 Introduction

In this chapter, different types of RAP materials and the methods for producing samples and mixing plan tests are investigated, and the study of different types of rejuvenators for use in recycled mixtures, as well as the methods of mixing the rejuvenators, along with the study of various fibers used in asphalt and their performances, are conducted; moreover, various types of tests performed on the fibers are introduced and the resources are studied to gather the existing information on the mechanical behavior of asphalt in the process of crack initiation and growth.

2.2 History of Asphalt Recycling

In early 1900s, the concept of sustainability and recycling gained popularity owing to high carbon emission from industrial and construction sectors. In particular, the construction industry which contributes a significant portion of carbon dioxide, had to make sufficient changes to adapt to lower emissions. Some of the changes reflected were efficient construction, improved materials and in some instances introduction of accelerated construction procedures [8–10]. Asphalt recycling is also not a new concept. In 1970, the desire to recycle pavements intensified with the oil sanctions which had increased the price of asphalt. Significant advances in the development of heavy equipment and manufacturing processes have been part of the recycling evolution path. The development of the powerful mill tool enabled executors to recycle materials from damaged asphalt pavements and mix them with new materials. Warm asphalt companies were modified to use recycled asphalt pavements. The mixture plan, structural design, and construction operations were modified
to use RAP in case required. The trend towards recycling began in the early days of 2000 due to the rapid increase of costs and declining resources of quality materials [11].

2.3 Asphalt Recycling Methods

In general, asphalt recycling is performed in two warm and cold ways. Each of these methods can be performed on a factory or on-site basis. The asphalt recycled using the warm method is generally more durable and resistant compared to the asphalt recycled using a cold method. However, high temperatures (above 150 °F) in warm recycling provide two major problems: 1) re-aging of the asphalt binder in asphalt aggregate materials that are already aged and hardened, and 2) production of toxic and harmful gases, which in turn limits the percentage of asphalt aggregate materials. Extensive research has been conducted in recent years on recycling using semi-warm methods so that in addition to resolving the aforementioned problems, recycled asphalt with desired performance can be obtained. Moreover, this method can use a high percentage (50% to 100%) of the asphalt aggregate materials in the recycling process [12].

2.4 Experience of United States in Using RAP

In 1978, 30% RAP was used on an intercity road in Panama City. Then, in 1979, 35% of RAPs were used in 31500 tons of asphalt in an experiment in the city of Okala, which, considering the operating conditions and the quality of the recycled materials produced at that time, no problems occurred on that road for 12 to 13 years according to the monitoring committee of the time. After adequate testing and studies, the license to use a percentage of RAPs higher than 60% was issued in 1980. In the assessment of the roads of Florida, 60 percent of the roads are estimated to be made from recycled asphalt. The Illinois Department of Transportation has used about 62300 tons of RAP on its highways by 2001.
Since 2007, about 40 million tons of RAP have been used in pavement layers annually, from 100 million tons of RAP produced.

States that use high percentages of RAP in pavement layers are highlighted in Figure 2. As can be seen, more than half of the states have used high percentages of RAP in their asphalt mixtures in 2011.

![Figure 2. States that use more than 20 percent RAP in HMA layers. [13]](image)

Since 2008, due to the sharp rise in oil prices, the increase in RAP consumption rate has been emphasized, such that according to the table below, the RAP consumption in different states reaches 50%. Table 1 shows that 10% to 50% of all RAP produced in the United States in 2007 and 2008 have been reused.

**Table 1. Consumption amount of recycled RAP in US [13]**

<table>
<thead>
<tr>
<th>State</th>
<th>RAP%</th>
<th>Date of execution</th>
</tr>
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<tbody>
<tr>
<td>North Carolina</td>
<td>40</td>
<td>September 2007</td>
</tr>
<tr>
<td>South Carolina</td>
<td>30-50</td>
<td>October 2007</td>
</tr>
<tr>
<td>Wisconsin</td>
<td>25</td>
<td>November 2007</td>
</tr>
<tr>
<td>Florida</td>
<td>45</td>
<td>December 2007</td>
</tr>
<tr>
<td>Kansas</td>
<td>30-40</td>
<td>May 2008</td>
</tr>
<tr>
<td>Delaware</td>
<td>35</td>
<td>June 2008</td>
</tr>
<tr>
<td>Minnesota</td>
<td>30</td>
<td>2008</td>
</tr>
<tr>
<td>Illinois</td>
<td>10-50</td>
<td>2008</td>
</tr>
</tbody>
</table>
2.5 Features of Recycled RAP

During the service life of asphalt pavements, the composition of stone and asphalt binder materials is affected by many physical and geophysical changes that must be taken into consideration during the design phase of the recycled asphalt mixture to ensure the hot mix asphalt (HMA) with RAP, performs similar to HMA mixture with intact stone materials. Knowing the amount of asphalt binder in RAP, as well as the physical characteristics of the stone materials in RAP, such as granulation and fracture, is of great importance.

Various methods have been developed to separate asphalt binder and stone materials or to determine the amount of asphalt, which include solvent separation, nuclear measurement method, automatic recording, and combustion method.

2.5.1 Asphalt Binder Content of RAP

When the aged asphalt binder in the RAP is combined with the new asphalt, it affects the performance degree of the new asphalt. When low percentages of RAP (below 15%) are used, this impact is negligible, but at high percentages, the effect is significant [14]. The asphalt binder inside RAP can be recovered using different methods such as using a rotary evaporator, or the Abson method. Recovering the asphalt binder inside RAP is generally performed for two purposes. 1- to determine the geophysical properties of the aged asphalt. 2- to determine the percentage and type of the rejuvenating substance.

Asphalt binder recovery can affect its characteristics. Therefore, firstly, it is necessary to perform this procedure with the utmost care, and secondly, the recovery method must be chosen carefully.

The rotary evaporator method is known as one of the reliable methods, to recover binder. A brief description of this test is that first the asphalt binder of RAP materials is washed...
using the asphalt binder decomposition method (according to ASTM D2 standard) and using methyl chloride as a solvent, and then the resulting solution is placed inside the evaporator to evaporate its solvent. This experiment is performed according to EN-1269703 standard. The components of the rotary evaporator are illustrated in the figure below [15]. In this study, asphalt binder recovery is performed to conduct tests of penetration degree, softening, and viscosity points with rejuvenators to determine the optimal percentage of rejuvenator.

2.5.2 Rejuvenators

Rejuvenators are generally made of lubricating and expanding oils. Much of this substance is made of malts and aromatic compounds. These compounds return the malts that have been lost in the asphalt binder over time and during the aging process. Moreover, aromatic compounds are used for their compatibility with asphaltene and their uniform distribution [15].

When combined with aged asphalt, rejuvenators improve the performance of that asphalt. The behavior of the asphalt binder modified by the rejuvenator directly impacts and improves the behavior of the asphalt compound. The selection of the rejuvenator material is performed according to the ASTM D4552 standard. Among the characteristics that should be considered regarding rejuvenators are their viscosity before and after short-term aging, their degree of ignition, and percentage of solvent [11].

2.5.3 Combining asphalt binder with rejuvenator

Research has shown that the combination of aged asphalt binder with rejuvenators is time-consuming and it takes a while for the aged asphalt binder to thoroughly mix with the
rejuvenator, depending on the type of rejuvenator and the degree of asphalt binder aging [16].

One question is whether RAP materials act like black aggregates, i.e., the aged asphalt binder in the RAP does not mix with the fresh asphalt binder and does not change its characteristics. This question is addressed in project NCHRP 9-12:

The issue that asphalt aggregate materials act like black aggregates and do not participate in mixture with new asphalt is correct to some extent. Research has shown that in practice neither of these two situations are correct and some of the asphalt binders present in RAP participate in the mixture, the percentage of which differs proportionate to the stiffness of the available asphalt. The effects for low RAP percentages are still negligible. However, at higher RAP percentages asphalt binder existing in RAP increases the stiffness of the final product.

At higher RAP percentages, further studies have to be conducted on what grade of asphalt binder is required or what is the maximum useable RAP.

In general, it can be stated that RAP percentages between 15 to 30%, do not require extensive experiments. However, more tests and controls must be conducted for higher percentages. It should be noted that the asphalt binder that does not participate in mixing is not ineffective in the mixture. Asphalt binder generally plays three primary roles in the mixture: 1) coating stone materials to prevent moisture damages, 2) lubricating the mixture in case of compaction, and 3) gluing the mixture together after compaction. In fact, asphalt binder that does not participate in the mixture has lower effects on 2 and 3, but is quite effective regarding case 1, and prevents the absorption of pure asphalt binder by RAP materials [17].
2.5.4 Stone materials in RAP

The aggregates within the RAP affect the volume and performance characteristics of the mixture. The structure of material aggregates, broken aggregates, dust and soil present in them, and angled and lean aggregates must be considered in the plan. The effects of these parameters are also negligible for low percentages of RAP.

In case of using a solvent in the asphalt binder separation stage, to determine the granulation size and characteristics of stone materials, the materials must be completely dried in a furnace or in front of a fan. If a combustion chamber is used, the stone materials must be completely cooled. After the combustion, some stone materials are broken or destroyed and the granulation changes. Due to the widespread use of separation methods using combustion chambers, some US states have not allowed the use of combustion chambers for determining the granulation of stone materials. Regulations in some states relate the application of combustion chambers to the type of stone materials in the area [11].

2.6 Effects of RAP on Characteristics of Asphalt Mixtures

Since extensive research has been conducted on the issue of recycled asphalt aggregates, and today many of their findings have reached the stage of industrialization, this Section is attempted to focus on studies that use high percentages of recycled RAP and investigate the crack characteristics.

In studies performed by NCAT institution in 2012, zero to 50% of the RAP obtained from RAP and reclaimed asphalt shingles (RAS) along with a percentage of roof RAP were put at the temperature of 10 °C with and without rejuvenators and in different short- and long-term aging simulation conditions using the dynamical module, resilience module and
energy rate tests. Furthermore, in this study, the asphalt binder existing in the RAPs were extracted and their low- and medium-temperature performances were investigated using Bending beam rheometer (BBR) and dynamic shear rheometer (DSR) tests [18].

Among the most important results obtained from these studies, is the negative effect of using RAP on the resistance of asphalt binder and asphalt mixtures against exhaustive and heat cracks. It was shown in these results that the use of rejuvenators can compensate for this deficiency to some extent, however, it does not still reach the quality of the conventional RAP-free mixture. These experiments were conducted at two levels on the asphalt binder recycled from RAP and RAS and the mixtures containing RAP and RAS. In addition, all experiments were conducted with and without the rejuvenator known as a recycling agent (RA).

In the remainder of this study, the energy rate (ER) test, which includes three tests of resilience module, creep compliance, and indirect tensile test was developed at the University of Florida to evaluate the resistance of asphalt mixtures against top-down cracks, was used at 10 ºC. The asphalt mixture with a higher energy rate has higher energy loss at the time of fraction, higher minimum creep energy loss, and higher fracture energy, and thus exhibits higher resistance against cracking.

In 2012, Al-Qadi et al. studied the behavior of recycled asphalt mixtures containing 0 to 50% RAP in a research project, at the request of the Illinois department of transportation. Among the experiments performed by them was the fracture test of asphalt mixtures using SCB samples at temperatures -24 and -12 ºC. In this experiment, they investigated the diagram of force in terms of crack opening. Moreover, a 4-point beam test was also performed to predict the fatigue of asphalt mixtures containing RAP. The results of fatigue tests suggested that mixtures containing 40 and 50 percent RAP, have lower fatigue
durations compared to mixtures with 0 to 30 percent RAP. Additionally, the results of fracture tests showed that the fracture energy reduces at minus 12 °C by adding RAP to the mixture.

But afterward, a relative improvement is observed in the fracture energy by adding a percentage of the RAP. The fracture energy at -24 °C improves a little by adding RAP. Figure 3 illustrates the results of this experiment for asphalt binder PG 64-22 [19].

![Fracture energy for mixtures containing different percentages of RAP](image)

*Figure 3. Fracture energy for mixtures containing different percentages of RAP [19]*

One of the interesting and remarkable results of this research is that the trend of fracture energy changes at two temperatures can be different with the increase of RAP percentage. At -12 °C, the fracture energy was decreased significantly by adding 30% RAP, while adding 40% and 50% RAP slightly increased the fracture energy. However, it was still be lower than the control mixture lacking RAP. This is while adding RAP to the asphalt mixture at -24 °C, the fracture energy remained almost intact while adding 40% and 50% RAP increased the fracture energy compared to the RAP-free control mixture. The asphalt
binder was not yet completely fragile at -12°C. Mixtures containing higher RAP percentages have both higher stiffness and lower deformations and have lower fracture energy compared to RAP-free control mixtures. However, at -24°C, the mixtures are totally fragile and deformation due to load is almost non-existent. Therefore, with the increase of RAP, the mixture becomes harder, and fractures harder and the fracture energy increases. During the project NCHRP 9-46, fracture toughness and fracture energy of asphalt mixtures containing 0, 25, and 55 percentage RAP at -9, -19, and -29 ºC were investigated using SCB samples at mode one of the fractures. The results demonstrated an increase in fracture toughness with the increase of the percentage of RAP at all temperatures. On the one hand, the fracture energy at -9 ºC reduces with the increase of the percentage of RAP, however, an increase in the fracture energy is observed with the increase of the percentage of RAP. This is since at lower temperatures, the fracture energy is affected more than the maximum load, and since the maximum load increases with the increase of RAPs percentage, the fracture energy decreases with the increase of the percentage of RAP at lower temperatures.

In a statistical study on data from the information bank of long-term pavement performances (LTPP), West et al. showed that the most damages in the pavements containing recycled RAP correspond to fatigue and heat cracks [20].

In a study in 2007, conducting an indirect traction test on asphalt mixtures containing 0, 15, 25, and 40% RAPs at -10°C, Shah et al. showed that increasing the percentage of RAP increases the stiffness of the mixture, consequently increasing the critical crack temperature of the mixtures. [21].
In 2016, Zhou et al., attempted to improve the behavior of asphalt mixtures containing 50% RAP using Latex polymer and two different types of rejuvenators. Performing fracture tests on the SCB sample at -12 °C, as well as fatigue tests on the SCB sample at 15 °C, they investigated the behavior of these mixtures. The results demonstrated an improvement in the behavior of recycled mixtures against fatigue and fracture [22]. During these experiments they showed that the fracture energy and the tensile strength for the case with rejuvenator was higher than the case where no rejuvenators are used and that the fracture energy and tensile strength for the case where both rejuvenators were used and also Latex polymer is used as the modifier additive for 50% RAP, are almost equal to the fracture energy and tensile strength of the RAP-free control mixture [21]. In the remainder of this study, fatigue tests in stress control mode with cyclic loads were conducted on SCB samples with the same geometry of the fracture test. This experiment, known as the R-SCB test, was performed at a temperature of 15°C. The trend of changes in the fatigue life is also identical to the fracture test and the fatigue life decreases by adding RAP. However, the addition of rejuvenator and Latex polymer compensate for this effect.

In 2018, Song et al. suggested that the resistance of recycled mixtures is reduced compared to cracking of asphalt mixtures by adding RAP. However, although the use of rejuvenators resolves this problem to some extent, on the other hand, rejuvenators reduce the grooving resistance of the mixtures. They then showed that the grooving problem can be resolved using semi-warm technology [23].

It was shown in the project NCHRP 9-58 that the use of RAP reduces the resistance of asphalt mixtures against cracking. This deficiency can be compensated for to some extent, by using rejuvenators, and softer asphalt binder and increasing the amount of virgin asphalt.
Nevertheless, studies have shown that the use of softer asphalt binder cannot restore the total lost resistance and increasing the percentage of fresh asphalt binder also lowers the economic justification of the project. Therefore, the use of rejuvenators was recommended as an effective solution [24].

In 2015, Giustozzi et al., investigated a warm asphalt mixture containing 40% RAP and the possibility of its modification using different additives such as composite fibers, cellulose fibers, SBS polymer, rejuvenator, and anti-strippers. To this end, they investigated the impacts of these modifiers on the viscoelastic behavior of the mixture, as well as its fatigue and grooving performances. They demonstrated that the use of composite and cellulose fibers has a positive effect on the fatigue and grooving behaviors of the mixtures. However, when these fibers are combined with SBS polymer, their positive effects become doubled [14].

In their research in 2016, Walla Mogawer et al. attempted to improve the behavior of asphalt mixtures containing 50% RAP. To this end, they used different rejuvenators, different asphalt binders, and SBS polymer. According to their results, all of the abovementioned solutions are effective in improving the resistance of asphalt mixtures containing 50% RAP against cracks and fatigue. This is despite the fact that unmodified recycled mixtures had lower resistance compared to the control mixture.

In this study, the 4-point beam test was conducted in form of constant strain to control fatigue at 15°C, and the TSRST test was also performed with a cooling rate of 10°C to control cracking in asphalt mixtures.
2.7 Using High Percentages of RAP in Asphalt Mixture

After the oil crisis and the sharp rise in prices of asphalt binder and oil derivatives in the 1970s, extensive research was conducted in the United States and Europe in the field of asphalt recycling and further use of recycled asphalts. At the time, using RAP in the mixture was limited to 30 to 70 weight percent of asphalt mixtures. In 1978, in a project commissioned by the Arizona department of transportation, for the first time, mixtures containing 100 percent RAP were produced. However, the experience of using asphalts containing 100% RAP was not so successful at the time. Nevertheless, the use of mixtures containing 100% RAP was not forgotten until many years later, when this issue was used in on-site warm recovered mixtures [25].

The use of recycled asphalt mixtures containing 100% RAP was first proposed in the annual TRB conference by Rajab Malik et al., in an article titled “why not to use mixtures containing 100% RAP”. In this research, producing completely recycled samples and conducting tests such as dynamic module, and creep compliance prediction, they suggested that the use of mixtures containing 100% RAP is practical and feasible [26].

In 2014, Zaumanis et al. performed extensive research on the effects of different types of rejuvenators on the characteristics of mixtures containing 100% recycled RAP. They used indirect traction tests in their experiments to find the critical crack temperature ($T_{crit}$) in asphalt mixtures.

The critical crack temperature is an estimation of the temperature where heat tension at the sample level reaches its resistance. This temperature can be estimated for any asphalt mixtures, using the diagram of creep compliance and indirect tensile strength. This
estimation is obtained using the mechanistic model derived from the United States’ Strategic Highway Research Program (SHRP) and presented in form of Excel software [5]. It is demonstrated in this research that recycled mixtures have higher critical crack temperatures and the use of rejuvenators significantly resolves this deficiency. The results of this experiment are presented in Figure 4. As can be seen from this figure, the critical crack temperature for the recycled mixture without rejuvenators is 10 ºC higher than the RAP-free control mixture. Adding some rejuvenators can reduce the critical temperature to the level of the critical temperature of the control mixture.

**Figure 4. Critical crack temperature for recycled mixtures with various rejuvenators [5]**

However, they then showed by conducting a coaxial cutting fatigue test that the use of recycled mixtures increases the fatigue life span [5]. This is because this test was conducted only at a relatively low strain level, and naturally, samples with higher stiffness show better resistance against fatigue at low strain levels, and these tests should be performed for different strain levels.

In 2016, Dinis Almeida et al. investigated warm asphalt mixtures made from 100% RAP. They used two samples with RAP obtained from two different sources, once with granulation modification with 20% admix and once without granulation modification.
They used the 4-point beam fatigue test and the results showed that fatigue resistance reduces when using mixtures containing 100% RAP materials.

During the test in the tension control mode, the number of cycles required for achieving 100 micro strains for each RAP source are 82 and 74 percent of the mixture without RAP, respectively [27].

2.8 Use of Fibers to Improve Cracking in Asphalt Mixtures

The use of mineral and synthetic fibers to improve the cracking behavior of asphalt mixtures has a long history. Fibers have been used in asphalt mixtures for various reasons. Among the reasons for using fibers in asphalt mixtures, one can mention asphalt binder stabilization and preventing asphalt binder from falling in stone mastic asphalt (SMA) and open graded asphalt (OGAP) mixtures, improvement of moisture sensitivity, grooving, reflective cracks, and resilience module of asphalt mixtures [24].

In 2010, Kaloush et al. conducted a comprehensive study on polypropylene and Forta polyaramid fibers at the University of Arizona. They conducted dynamical module, dynamic creep, fatigue, and low-temperature fracture tests on control samples and samples containing fibers. The fatigue test was performed by the three-point beam apparatus in form of constant strain at three different strain levels. The results showed that the performance of mixtures containing fibers is great at low strain levels, however, at high strain levels, the performances of these mixtures drop and their resistance against fatigue also decreases. This shows that these mixtures can be used in roads with higher mean velocities. However, constant strain tests with high strains for samples with higher stiffness are not fair, since a much higher tension is applied to it compared to softer samples [28]. In the remainder of the research above, the indirect tensile test has been used to measure the resistance of
mixtures reinforced with fiber, against cracking. This test was performed at 0, -10 and 10 °C and the results were reported. The results indicate a significant improvement in the resistance of asphalt mixtures reinforced by fibers (Figure 5) [25].

![Figure 5 Fracture energy of asphalt mixtures with and without fiber][28]

Fazayeli et al. used polyaramid fiber equal to 0.05 weight percent of the mixture to improve the characteristics of warm and semi-warm asphalt mixtures in a study in 2016. They conducted fatigue, indirect traction, and fracture tests by continuous loading at 0 °C and using SCB samples. The results indicated an improvement in the resistance of asphalt mixtures against fatigue and cracking when fibers were used [29].

Considering the provided items, most researchers have reported a negative impact on the resistance of asphalt mixtures against fatigue and cracking, when using recycled RAP. When high percentages of recycled RAP are used, the use of asphalt binder additives such as waxes and polymers faces with limitations. Therefore, the use of fibers as additives to the mixture for compensating the reduction of resistance of recycled mixtures against fatigue and cracking can be a viable option.
In 2018, in their study on the crack resistance of asphalt mixtures containing glass fibers, Morea and Zerbino showed that the use of these fibers improves the cracking resistance and grooving resistance of asphalt mixtures [30]. Asphalts reinforced with Polyolefin and glass fibers were studied by Abu-Talebi et al. in 2018. They suggested that this fiber generally improves the fatigue, cracking, and grooving resistances of asphalt mixtures [31]. In research in 2018, Al-Hadidi et al. suggested that the use of Polypropylene fibers in the amount of 3% of the asphalt binder mixture improves the indirect tensile strength and marshal strength of asphalt mixtures. They then showed that reinforcing the consumable asphalt binder with Polypropylene fibers reduces the moisture sensitivity and increase the cracking resistance in asphalt mixtures [32]. Moreover, in a study in 2016, Kathari et al. had verified the reduction of heat sensitivity and improvement of grooving resistance in mixtures containing Polypropylene [33].

2.9 Concepts of Fracture Mechanics and Fracture Tests on Asphalt Mixtures

In general, fractures in the pavements are due to weak resistance of pavements against factors causing shear and tensile stresses. If tensions of one point inside one body are alternately increased and decreased, it is said that the structure has under the effect of dynamic forces and can experience fatigue cracking. Therefore, one of the reasons for cracks in pavements and asphalt covers is the reduction of resistance under the effect of fatigue and increase of tensions applied compared to the final resistance of the asphalt. Considering the fatigue phenomenon and rules corresponding to this phenomenon to investigate the fatigue life span of pavements and the paths of crack growth resulting from loading stages would be useful. However, due to the complexities of asphalt mixtures, the
methods and equations used to predict the fatigue life span of asphalt mixtures are always faced with limitations, and it may be possible that they do not provide an appropriate prediction with the change in structure and type of the mixture. On the other hand, most of these models are not presented based on the fracture mechanics of materials. The use of principles of fracture mechanics can be effective in predicting crack growth and distribution. Using the principles of fracture mechanics, it becomes possible to evaluate tension changes at the crack’s tip caused by crack geometry, boundary conditions, and characteristics of the asphalt mixture. The use of this method is based on the principle that crack initiation is due to the existence of an initial deficiency in the material. The extent and manner of distribution of this deficiency significantly affect crack growth and distribution.

With the growth of the crack such that it reaches a critical limit, the crack suddenly widens and it becomes an unstable and disunited instance.

In general, the following three factors can be stated as the reasons for crack initiation and growth.

a) Atmospheric factors and thermal stresses: consecutive cooling and heating of pavement layers throughout the year and the non-uniformity of thermal conductivity coefficients of the layers cause an inconsistency in the deformations created in the pavement layers. This causes stresses and cracking of the layers as well as widening of the cracks throughout the pavement.

b) Traffic loads and passage of vehicle wheels over the pavements cause various tensions in the pavement layer which is itself a factor in creating fatigue due to
frequency of traffic load and it will also increase tensile and shear stresses in the pavement body, and thus widen the cracks.

c) Subsidence in the lower layers: lack of proper implementation of pavement layers and lack of sufficient compactions in the layers, resulting in vertical subsidence in the base, and sub-base layers and even in the bed soil, which can, in turn, be effective in widening the cracks due to the elimination of the support of the upper layers [34].

2.9.1 Using the Energy Method for Estimation of Crack propagation

When a cracked object undergoes load and tension, a fracture plastic zone (FPZ) is formed about the edges of the crack. In theory, the formation of LEFM and the crack initiation begin with the emergence of the FPZ zone (Figure 6). This zone is generally small for elastic materials and has a small share in crack propagation. However, in semi-brittle asphalt mixtures, this zone is primarily large and has a significant effect in asphalt fracture characteristics. In this case, the use of the LEFM theory also becomes an issue for further consideration. Therefore, researchers try to use the EPFM theory for asphalt mixtures. The use of energy methods and calculation of opening of the tip or edges of the crack are among the simple methods for using this theory in asphalt mixtures. The fracture energy which is computed from the multiplication of load and displacement is defined as Equation 1.

\[
fracture \ energy = \frac{work}{unit \ of \ surface \ traveled \ by \ crack} = \frac{\int P \, du}{unit \ of \ surface \ traveled \ by \ crack} \quad Equation \ 1
\]

*Figure 6. The FPZ zone at the tip of the crack*
Figure 7 presents the schematic graph of load-crack opening deformation for a sample asphalt. As seen from this figure, the graph is separated into two Sections. The first Section corresponds to before initiation of the crack and the second Section corresponds to after the emergence and propagation of the crack. In the first Section, the tension tolerated by the sample increases at a significant rate, while the deformation applied to the sample does not demonstrate a great change. In the second stage, the tension starts to decrease at a lower rate, while the increased rate of deformation is too big. At this moment, the crack continues its stable propagation with the energy stored at the tip of the crack reaching the surface energy required for fracture [35].

![Graph showing load-crack opening deformation for asphalt mixtures](image)

**Figure 7. The load-crack opening deformation type diagram for asphalt mixtures [35].**

In 2010, Saha and Biligiri investigated crack propagation in asphalt mixtures with continuous, hollow, and open aggregates. They performed the fracture test at 35 ºC and plotted the load-crack opening deformation diagrams. They showed in their study that the crack onset occurs sooner in mixtures with hollow and open aggregates, however, the crack propagation in these mixtures takes a longer time to happen. In the remainder of this research, to investigate the trend of crack propagation, the fracture energies are plotted and
compared to each other for different intervals. To this end, the fracture energies were calculated for 50%, 75%, and 100% deficiencies (deficiencies were calculated using the concept of dissipated energy) and compared at different temperatures. The results of this study demonstrated that there exists an identical trend for deficiency propagation of different mixtures [35].

2.10. Summary and Conclusion

This chapter was an attempt to present a comprehensive summary of the studies on recycled asphalt mixtures, fiber mixtures, and various methods for evaluating asphalt mixtures against cracking. The summary of the content presented in this chapter is as follows:

1. The use of RAP weakens the performance of asphalt mixtures at medium and low temperatures and reduces the resistance of asphalt mixtures against cracks. Studies have shown that this reduction in resistance can be compensated for to some extent using additives such as rejuvenators and polymers. However, when high percentages of RAP are used, the aforementioned solutions are not be effective due to the reduction in the amount of new asphalt.

2. Studies have suggested that the use of fibers can increase the resistance of asphalt mixtures against cracks to a great extent. Therefore, a combination of fibers with recycled mixtures is capable of compensating for their lost resistance against cracking. However, this solution has not been extensively studied.

Thus, this study investigates the crack resistance of 100%RAP mixtures reinforced with fibers and evaluates the effect of various FL and FC on the mixture at different temperatures.
CHAPTER III. METHODOLOGY

This chapter gives an outline of the research methods that were followed in the study. It describes the research design that was chosen for this study and the reasons for this choice. The instrument that was used for data collection is also described and the procedures that were followed to carry out this study are included. The researcher also discusses the methods used to analyze the data.

3.1 Research Plan

The most significant problem associated with using a high percentage of RAP is reduction in cracking resistance at low- and intermediate-temperature. Furthermore, it is speculated that by the increase of fiber content, the binder content should be increased. This can lead to a greater rutting susceptibility. Thus, the research focuses on the fatigue and rutting performance of the asphalt mixture made of 100% RAP which is rejuvenated with cooking oil and reinforced with basalt fiber. According to the literature, FL and FC also have a major impact on performance. Therefore, the sample preparation is designed to include a variety of FL and FC. The research plan contains four tasks. The outline of the research plan is illustrated in Figure 8.
Based on the literature, there is no definite method to assess the thermal cracking resistance of the mixture using BBR test outputs. This research study also proposes a method to assess the thermal cracking potential of the mixture using the BBR test results.

3.2 Performance Tests Introduction

In this Section, each test required for this research study is introduced and the reason for choosing each of them is explained.

3.2.1 Intermediate Temperature Crack Test

National Cooperative Highway Research Program (NCHRP) 9-57 recognized the seven most common cracking tests used in the US as bending beam fatigue (BBF) test, overlay test (OT), disk-shaped compact tension (DCT) test, indirect tensile creep and strength test (IDT-CST), and three versions of semi-circular bend (SCB) tests [36]. The advantages and disadvantages of each test are summarized in Table 2.

Figure 8 Tests and analysis program
Table 2. Comparison of seven existing cracking tests [36]

<table>
<thead>
<tr>
<th>Cracking tests</th>
<th>Test limitations and equipment cost</th>
</tr>
</thead>
</table>
| DCT                             | • Specimen prep: 3 cuts, 1 notch, and 2 holes  
• Instrumentation: glue 2 studs, mount 1 clip gauge  
• Equipment cost: $50,000 |
| SCB-AASHTO TP105                | • Specimen prep: 3 cuts and 1 notch  
• Instrumentation: glue 3 studs, mount 1 extensometer + 1 clip gauge  
• Testing: 30 min.  
• Equipment cost: $50,000 |
| SCB-Louisiana transportation research center | • Specimen prep: 9 cuts and 3 notches  
• Testing: around 30 min.  
• Equipment cost: less than $10,000 |
| SCB-Illinois                    | • Specimen prep: 3 cuts and 1 notch  
• Equipment cost: $10,000-$18,000 |
| IDT-CST                         | • Specimen prep: 2 cuts  
• Instrumentation: Glue 8 studs, mount 4 extensometers  
• Testing: 1-2 hours  
• Equipment cost: more than $50,000 |
| OT                              | • Specimen prep: 4 cuts, glue specimen to bottom plates  
• Testing: 30 min. - 3 hr.  
• Equipment cost: $40-50,000 |
| BBF                             | • Specimen prep: large slab, 4 cuts  
• Instrumentation: glue 1 stud and mount 1 linear variable differential transformer  
• Specimen testing: 1 hour to days  
• Equipment cost: more than $100,000 |
| IDEAL-CT                        | • No cutting, notching, drilling, gluing, or instrumentation  
• Test completion within 1 min.  
• Repeatable (or low variability) with COV<25 percent  
• Practical for routine uses in Departments of Transportation (DOTs) and contractors’ laboratories  
• Low cost test equipment (<$10,000)  
• Sensitive to asphalt mix composition  
• Cracking performance related |

NCHRP 9-57 recognized seven advantages of using the IDEAL-CT test, as listed below [36]:

1. The process of sample fabrication and test preparation is straightforward. (no need for cutting, drilling, gluing, notching, or instrumentation)
2. The test is not time-consuming (the duration of the test is less than 1 minute)
3. The test is relatively cheap (it costs less than $10,000)

4. COV of the results for replicates of the same type of mixture is low (less than 25)

5. The test result is highly sensitive to asphalt compositions such as additives, binder, and aggregates

6. Results show a good correlation with field cracking performance

Considering these advantages, the IDEAL-CT test was selected to measure intermediate cracking performance in this research study.

The important parameters of the IDEAL-CT test are derived from the measured load vs. displacement curve. The cracking parameter is adopted from the well-known Paris’ law [37] and the work of Bazant and Prat [38]. The cracking test index (CT\textsubscript{index}) is presented in Equation 2 [36].

\[ CT_{\text{index}} = \frac{G_f}{m} \times \left( \frac{l}{D} \right) \]  \hspace{1cm}  \textit{Equation 2}

where fracture energy \( G_f \) is the work of fracture (the area of the load-displacement curve) divided by the area of cracking face; \( m \) is the slope of the load-displacement curve, and \( l/D \) represents the strain

The larger the CT\textsubscript{index}, means cracks grow at a lower rate. Due to the visco-elastic-plastic behavior of asphalt mixture, \( m \), and \( l/D \) along the load-displacement curve. Consequently, the CT\textsubscript{index} value changes at each point on the curve. According to the work of Zhou et al., the average CT\textsubscript{index} should be calculated at the post-peak point (PPP\textsubscript{75}) where the load is reduced to 75 percent of the peak load (Figure 9). Zhou et al. propose Equation 3 to calculate \( m \) at PPP\textsubscript{75} considering a standard deviation of 5 [36].

\[ |m_{75}| = \left| \frac{P_{85}+P_{65}}{l_{85}+l_{65}} \right| \]  \hspace{1cm}  \textit{Equation 3}
Parameters of the equation are illustrated in Figure 9.

![Figure 9. Average slope at PP75(|m75|)](image)

### 3.2.2 Low-Temperature Thermal Cracking

The indirect tension (IDT) test is used to conduct creep and strength tests on asphalt mixture according to AASHTO T322-02 [39,39,40]. IDT testing has the following disadvantages [41]:

- The required equipment (loading frame, extensometers) are expensive,
- Maintenance and calibration are expensive and time-consuming.
- Relatively thick test specimens make it impossible to obtain the characteristic of mixture with pavement depth.

A research study conducted at the University of Minnesota [42] showed that the BBR not only can be used to specify asphalt binder, but also can characterize creep properties of asphalt mixture. The BBR testing has several important advantages over the IDT test [43]:
- Testing equipment has a reasonable price and it is a common instrument in asphalt mixture labs.
- BBR calibration is simple and due to the simple test procedure and sample preparation, the repeatability of the test results is very high
- Duet to the thin test specimens, it can be extracted from different Sections of the field core. Therefore, it allows evaluating the properties of asphalt pavement along with the depth of the pavement profile.

Additionally, by conducting a simple back calculation on the results, the effective properties of the asphalt binder could be extracted from mixtures contacting RAP [42]. The maximum stress and strain at the bottom of the beam are a function of the applied load and dimensions of the beam (Equations 5 and 6):

\[ \sigma_N = \frac{3PL}{2bh^2} \quad \text{Equation 4} \]
\[ \varepsilon(t) = \frac{6\delta(t)h}{L^2} \quad \text{Equation 5} \]

where \( \sigma_N \) is the nominal strength (MPa), \( \varepsilon \) is the strain at failure, \( P \) is the maximum measured load (N), \( L \) is the span length (mm), \( b \) is the width of the beam (mm), \( h \) is the thickness of the beam (mm) and \( \delta_N \) is the deflection (mm) of the beam corresponding to

Figure 10. BBR Strength test sample geometry [44]
the maximum load. Parameters related to the geometry of the sample are presented in Figure 10.

Few research studies are using the BBR test to evaluate the low temperature cracking resistance of the mixture [41,43,45]. However, there is no method to define a critical value for a mixture using the BBR test. In this research, the study author tried to define a criterion to be able to evaluate and compare the low-temperature thermal cracking of the samples.

### 3.2.3 High-Temperature Rutting Resistance

Increasing the fiber content increases the mixture airvoid, it could become more susceptible to permanent deformation. One of the most widely verified test methods to measure the deformations distress of the asphalt mixture is the asphalt pavement analyzer (APA) test [46]. According to a survey conducted by NCHRP 20-07/Task 406, the second most selected test used by states DOTs is the APA (Figure 11) [47].

![Figure 11. Number of agencies using a particular rutting test](image)

The test procedure is elaborated in AASHTO TP 63-03, “determining rutting susceptibility of Hot-Mix Asphalt (HMA) Using the Asphalt Pavement Analyzer”. In this research study, APA was used to investigate the rutting performance of the mixtures at various FC and FL.
CHAPTER IV. MATERIAL AND SPECIMEN PREPARATION

This chapter provides information about the source of material, type of material, and samples preparation process. The mix design procedure is also explained and information about geometric and volumetric characteristics of specimens is provided.

4.1 Consumed Materials

The materials and substances used in this research include stone materials, fibers, recycled RAP materials, rejuvenators, and fillers, the specifications and characteristics of each of which are explained below in detail:

4.1.1 Virgin asphalt binder and aggregate

The aggregates and asphalt binder used in this study were obtained from General asphalt in Miami, Florida. Some important characteristics of the lime aggregates and gradation are presented in Table 3. Upon DSR testing, the asphalt binder’s PG was 74-22°C

<table>
<thead>
<tr>
<th>Physical characteristics</th>
<th>Standard</th>
<th>Stone materials</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specific weight of fine-grained materials (g/cm³)</td>
<td>ASTM C127</td>
<td>2.63</td>
</tr>
<tr>
<td>Water absorption (%)</td>
<td>ASTM C127</td>
<td>0.9</td>
</tr>
<tr>
<td>Specific weight of coarse-grained materials (g/cm³)</td>
<td>ASTM C128</td>
<td>2.62</td>
</tr>
<tr>
<td>Los Angeles erosion (%)</td>
<td>ASTM C131</td>
<td>22.5</td>
</tr>
<tr>
<td>Percentage of fractures on both sides (%)</td>
<td>ASTM D5821</td>
<td>98</td>
</tr>
<tr>
<td>Percentage of fractures on one side (%)</td>
<td>ASTM D5821</td>
<td>100</td>
</tr>
<tr>
<td>Sand value (%)</td>
<td>ASTM D2419</td>
<td>60</td>
</tr>
</tbody>
</table>
4.1.2 RAP

A RAP sample weighing approximately 3,000 pounds was obtained from General Asphalt Co. Inc. It was the depot gathered by scraping the asphalt in NW 36th St, Miami, Florida. After being stored in the Green Paving Laboratory at Florida International University, it was mixed thoroughly to produce a consistent sample. Three samples of RAP were tested to characterize the material. The aggregate was extracted from the samples, and the binder content was determined using an ignition oven, following ASTM D6307. In addition, the maximum specific gravity (G_{mm}) of the samples was determined per ASTM D2041.

The RAP binder was recovered using a centrifuge extractor and a rotary evaporator, under ASTM D2172 and ASTM D5404. Table 4 shows the average gradation from three RAP samples. Binder content was equal to 7.20% and 7.10 for nominal maximum aggregate size (NMAS) 12.5, and 9.5 respectively. G_{mm}, of the samples, was 2.40 and 2.35.

Table 4. Gradation, Binder Content, and Maximum Specific Gravity, of the RAP

<table>
<thead>
<tr>
<th>Sieve Size</th>
<th>Average</th>
<th>NMAS 12.5mm</th>
<th>NMAS 9.5mm</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Lower</td>
<td>Upper</td>
</tr>
<tr>
<td>19 mm</td>
<td>100</td>
<td>100</td>
<td>-</td>
</tr>
<tr>
<td>12.5 mm</td>
<td>99</td>
<td>90</td>
<td>100</td>
</tr>
<tr>
<td>9.5 mm</td>
<td>93</td>
<td>-</td>
<td>90</td>
</tr>
<tr>
<td>No.4</td>
<td>67</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>No.8</td>
<td>52</td>
<td>28</td>
<td>58</td>
</tr>
<tr>
<td>No. 16</td>
<td>39</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>No. 30</td>
<td>32</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>No. 50</td>
<td>22</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>No. 100</td>
<td>7</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>No 200</td>
<td>4</td>
<td>2</td>
<td>10</td>
</tr>
</tbody>
</table>
4.1.3 Rejuvenator oil

Before performing the main experiments, a set of general tests were conducted on the WO rejuvenator according to the ASTM D4552 standard. The characteristics of the consumed rejuvenator along with its corresponding standard are provided by its producer and presented in Table 5.

Table 5. Physical characteristics of the used WO material.

<table>
<thead>
<tr>
<th>Test title</th>
<th>Results</th>
</tr>
</thead>
<tbody>
<tr>
<td>Special weight at 25°C</td>
<td>0.965</td>
</tr>
<tr>
<td>Degree of ignition, °C</td>
<td>215</td>
</tr>
<tr>
<td>Kinematic Viscosity at 135°C (mm².S⁻¹)</td>
<td>5.17</td>
</tr>
<tr>
<td>Moisture, Impurities</td>
<td>&lt;2%</td>
</tr>
</tbody>
</table>

4.2 Mixture Design Plan

According to comprehensive research studies conducted by Zaumanis et al., it was recognized that even fulfilling volumetric requirements would not ensure the expected mixture performance when it comes to 100% RAP mix design [25]. Therefore, the mixture design process used in this study was performance-based. Performance was evaluated for both binder and mixture. Binder tests were conducted to determine the rejuvenator content that satisfies the required high and low PG (Section 3.4.1). However, for comparison purposes, binder content was kept constant for all mixtures (Section 3.4.2). Furthermore, the air void of the samples with the same fiber content was controlled to be the same. The control sample was fabricated using a virgin aggregate of similar type and gradation as the aggregate contained in the RAP and the virgin binder.
4.2.1 Asphalt binder/rejuvenator performance

The performance grade of the RAP binder was determined based on the asphalt binder's performance at high and low temperatures. The asphalt binder existing in the RAP was recovered using a rotary evaporator device. The recovered asphalt binder was combined with 0, 3, 6, and 10.5% rejuvenator by weight of the binder. DSR test was conducted to determine the maximum possible amount of rejuvenator. The maximum rejuvenator amount was determined by the temperature at which the rutting parameter \( \left( \frac{G^*}{\sin \delta} \right) \) obtained from DSR test on original and short term aged asphalt binder were equal to 1.0 and 2.2, respectively [20,48]. The minimum rejuvenator amount was determined by the temperature at which flexural creep stiffens (S) and relaxation value (m) obtained from the BBR test, at the 60s of loading were equal to 300 KPa and 0.300, respectively [48]. Based on the results, the minimum rejuvenator content was 3.9% to meet the low PG, and the maximum was 9% to meet the high PG (Figure 12). A dose of 6.5% was selected for this study.

Figure 12. Critical A: high- and B: low-temperature of recycled binder containing various rejuvenator content
4.3 Sample Production

Sample production is one of the most important stages of producing recycled asphalt mixtures. To produce a sample, the first natural stone materials must remain at 170°C for 8 hours, and since the asphalt binder in RAPs can be damaged at this high temperature, these RAP were maintained at 170°C for two hours. Then an amount of rejuvenator equal to 6.5% of the binder existing in RAPs was added to RAPs, and they were blended. Afterward, other additives such as fibers were added with proportions of 0.10%, 0.30%, and 0.60% of the total mixture for basalt fibers, and they were thoroughly blended. Finally, the mixture was maintained at 140 degrees Celsius for two hours for processing. The mixture was then poured into the corresponding frame for molding and compression.

Design mixtures were compacted according to AASHTO T 312-11, and the number of gyrations was chosen for level B traffic level. The inside diameter of the gyratory mold was measured following AASHTO T 312-11 (Table 6).

<table>
<thead>
<tr>
<th>Traffic Level (1x106 ESAL’s)</th>
<th>N_{design}</th>
<th>Number of Gyraisons</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt;0.3 (A)</td>
<td>50</td>
<td></td>
</tr>
<tr>
<td>0.3 to &lt;3 (B)</td>
<td>65</td>
<td></td>
</tr>
<tr>
<td>3 to &lt;10 (C)</td>
<td>75</td>
<td></td>
</tr>
<tr>
<td>10 to &lt;30 (D)</td>
<td>100</td>
<td></td>
</tr>
<tr>
<td>≥30 (E)</td>
<td>100</td>
<td></td>
</tr>
</tbody>
</table>

These cylindrical samples are obtained using gyratory compression, which is used because it would be possible to control the percentages of air void spaces, special weight, sample height, and other mechanical characteristics of asphalt mixtures using this method. Such that samples produced using this method have relatively uniform mechanical specifications. Before pouring the mixtures inside a copper plate or mold, they must be put
inside an oven with a temperature of 150°C for 30 to 60 minutes, so that the heat of the mixture is not lost at the time of mixing.

To determine the percentage of air void, it is necessary to determine the maximum special weight of asphalt mixtures (G$_{mm}$), as well as the actual special weight of compressed asphalt samples. The G$_{mm}$ of asphalt mixtures was calculated using the method stated in the AASHTO T209 regulations. Moreover, to obtain the actual special weights of compressed asphalt mixtures (G$_{mb}$) after cooling the compressed asphalt mixture samples, their actual special weights were determined using the ASTM-D2726 regulation for the case without a paraffin coating.

4.3.1 Preparation of Cylindrical Samples

For IDEAL-CT and PA tests cylindrical samples were fabricated. The samples mold has a diameter of 149.9 to 150 millimeters. The mixture must be maintained inside the oven at a temperature of 150°C before molding. Considering the weight of molded materials, the height of the sample is calculated at about 115 ± 5 mm to achieve a field target air void of 7%.
4.3.2 Preparation of thin asphalt mixture beams

The following procedure is adopted from the work of Zofka et al. to obtain asphalt mixture BBR beams [42]. It explains how to obtain thin mixture beams from gyratory-compacted specimens fabricated in Section 3.5.2.

During saw cutting, 17–18mm of the bottom of the specimen was necessary for the clamps to grip the specimen. Three 12.5 mm-thick slice was cut from the specimen. These slices were further cut from three sides to obtain a 122 mm-wide irregular slice. Then, each slice was further cut into approximately 15 beams. A simple saw was used to fabricate BBR mixture beams. Figure 13 summarizes samples prepared for the BBR test.

![Figure 13. Illustration of BBR beam preparation [42].](image)

4.4 Summary of performance tests and samples

Figure 14 explain nomenclature used in this study. Table 7 presents the characteristics of the samples along with their names.

![Figure 14. Sample nomenclature used in the study.](image)
### Table 7. Composition and volumetric properties of the samples

<table>
<thead>
<tr>
<th>No</th>
<th>Sample</th>
<th>Air Voids, %</th>
<th>NMAS</th>
<th>Fiber Content, %</th>
<th>Fiber Length, mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>12-C</td>
<td>2.5</td>
<td>12.5</td>
<td>0</td>
<td>N/A</td>
</tr>
<tr>
<td>2</td>
<td>12-R</td>
<td>2.62</td>
<td>12.5</td>
<td>0</td>
<td>N/A</td>
</tr>
<tr>
<td>3</td>
<td>12-1-S</td>
<td>2.83</td>
<td>12.5</td>
<td>0.15</td>
<td>3</td>
</tr>
<tr>
<td>4</td>
<td>12-2-S</td>
<td>2.95</td>
<td>12.5</td>
<td>0.30</td>
<td>6</td>
</tr>
<tr>
<td>5</td>
<td>12-3-S</td>
<td>3.25</td>
<td>12.5</td>
<td>0.60</td>
<td>12</td>
</tr>
<tr>
<td>6</td>
<td>12-1-M</td>
<td>2.79</td>
<td>12.5</td>
<td>0.15</td>
<td>3</td>
</tr>
<tr>
<td>7</td>
<td>12-2-M</td>
<td>2.99</td>
<td>12.5</td>
<td>0.30</td>
<td>6</td>
</tr>
<tr>
<td>8</td>
<td>12-3-M</td>
<td>3.15</td>
<td>12.5</td>
<td>0.60</td>
<td>12</td>
</tr>
<tr>
<td>9</td>
<td>12-1-L</td>
<td>2.77</td>
<td>12.5</td>
<td>0.15</td>
<td>3</td>
</tr>
<tr>
<td>10</td>
<td>12-2-L</td>
<td>3.02</td>
<td>12.5</td>
<td>0.30</td>
<td>6</td>
</tr>
<tr>
<td>11</td>
<td>12-3-L</td>
<td>3.17</td>
<td>12.5</td>
<td>0.60</td>
<td>12</td>
</tr>
<tr>
<td>12</td>
<td>9-C</td>
<td>2.6</td>
<td>9.5</td>
<td>0</td>
<td>N/A</td>
</tr>
<tr>
<td>13</td>
<td>9-R</td>
<td>2.4</td>
<td>9.5</td>
<td>0</td>
<td>N/A</td>
</tr>
<tr>
<td>14</td>
<td>9-1-S</td>
<td>2.80</td>
<td>9.5</td>
<td>0.15</td>
<td>3</td>
</tr>
<tr>
<td>15</td>
<td>9-2-S</td>
<td>3.1</td>
<td>9.5</td>
<td>0.30</td>
<td>6</td>
</tr>
<tr>
<td>16</td>
<td>9-3-S</td>
<td>3.30</td>
<td>9.5</td>
<td>0.60</td>
<td>12</td>
</tr>
<tr>
<td>17</td>
<td>9-1-M</td>
<td>2.80</td>
<td>9.5</td>
<td>0.15</td>
<td>3</td>
</tr>
<tr>
<td>18</td>
<td>9-2-M</td>
<td>2.95</td>
<td>9.5</td>
<td>0.30</td>
<td>6</td>
</tr>
<tr>
<td>19</td>
<td>9-3-M</td>
<td>3.03</td>
<td>9.5</td>
<td>0.60</td>
<td>12</td>
</tr>
<tr>
<td>20</td>
<td>9-1-L</td>
<td>2.73</td>
<td>9.5</td>
<td>0.15</td>
<td>3</td>
</tr>
<tr>
<td>21</td>
<td>9-2-L</td>
<td>2.90</td>
<td>9.5</td>
<td>0.30</td>
<td>6</td>
</tr>
<tr>
<td>22</td>
<td>9-3-L</td>
<td>3</td>
<td>9.5</td>
<td>0.60</td>
<td>12</td>
</tr>
</tbody>
</table>

The performance of the samples will be evaluated through laboratory load tests including BBR, and IDEAL-CT to evaluate each mix’s susceptibility to cracking at intermediate and low temperatures. A rutting test will be conducted to check the rutting is within standard criteria. The performance tests are summarized in Table 8.
Table 8. Performance Tests and samples

<table>
<thead>
<tr>
<th>Test</th>
<th>Method</th>
<th>Engineering Properties</th>
<th>Specimen Geometry</th>
<th>A*</th>
<th>B**</th>
</tr>
</thead>
<tbody>
<tr>
<td>IDEAL-CT</td>
<td>NCHRP 20-07</td>
<td><strong>Intermediate Temperature:</strong></td>
<td>Φ150 mm x 62 mm</td>
<td>22</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Fatigue/Fracture Cracking Resistance</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BBR (mixture)</td>
<td>AASHTO TP-125</td>
<td><strong>Low Temperature:</strong></td>
<td>12.7mm x 6.35mm x 102mm</td>
<td>22</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Flexural Creep Stiffness of Asphalt Mixtures</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>APA</td>
<td>AASHTO TP 63</td>
<td><strong>High Temperature:</strong></td>
<td>Φ150 mm x 62 mm</td>
<td>22</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Rutting</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

A: Number of required results for analysis (equal to the number of mixture types in Table 8)
B: Number of replicates required for each result obtained from each mixture type
CHAPTER V. TEST RESULTS AND ANALYSIS

This chapter provides the results of the performance tests introduced in Chapter 3. Furthermore, the effect of variables like fiber length, fiber content, NMAS, and binder on cracking and rutting resistance of mixtures are evaluated. Then, after recognizing the most effective parameters, the performance of the mixture is predicted based on the data. Finally, design methods for designing a 100% RAP mixture reinforced with fibers are presented.

5.1 Intermediate Temperature (IDEAL-CT)

In this Section, the IDEAL-CT test is conducted and results are evaluated using a logistic function to find the inflection point of the post-peak (explained in Section 5.2.1).

In fact, in this section the assumption of calculating $CT_{index}$ at $PP_{75}$ is evaluated (explained in Section 3.2.1).

5.1.1 $CT_{index}$ at inflection point versus $PP_{75}$

$CT_{index}$ was calculated at the inflection point of the post-peak part of the load-displacement curve. The inflection point was obtained mathematically using Equation 6 which is a generalized logistic function to fit the post-peak region of the load-displacement data [49]. Solver (an “Add-in” program in Excel software) was used to minimize the sum of the squared error between actual data and fitted one. The fitted equation and its slope are illustrated in Figure 15.

$$F(t) = \kappa + \frac{\delta - \kappa}{[\chi + ye^{-\phi(t-\tau)}]^{1/\lambda}}$$  \hspace{1cm} \text{Equation 6}

Where $F$ is the fit function, $t$ is the displacement, and the rest of the parameters are subjected to change during the fitting process.
Figure 15. Load-displacement and slope of the fit to illustrate the inflection point and interval slope

Table 9 presents the peak load and inflection point of the load-displacement for all samples. The ratio of the load at the inflection point to peak-load is also presented in Table 9. The average ratio value for samples containing fibers was 69%. This is 6% less than the 75% which was estimated by Zhou et al. It is speculated that fiber affects the location of the inflection point. This could be due to additional resistance due to fibers' contribution which delays the transition from multiple micro-cracking to macro-cracking. Zhou et al. consideration of PPP75 was based on the none fiber-reinforced samples. Therefore, it could be possible that PPP75 is not a proper point on the load-displacement curve to compute the slope. The standard deviation was found to be ±5 which is the same as the result obtained by Zhou et al.

All CTindex values in this research were calculated based on inflection point. Assuming that the inflection point on average happens at 69 percent of the peak load (PPP_{69}). Then the m value was calculated using Equation 7.
\[ m_{61} = \frac{|P_{79} - P_{59}|}{l_{79} - l_{59}} \]

\textit{Equation 7}

Where \( P_{79} \) is the load at 79 percent of the peak load, \( P_{59} \) is the load at 59 percent of the peak load, \( l_{79} \) is the displacement at 79 percent of the peak load, \( l_{59} \) is the displacement at 59 percent of the peak load.

The interval slope at \( PPP_{69} \), is less variable than the tangent slope at the inflection point and can result in a less variable CT\textsubscript{Index} [36]. Thus, the \( \pm10 \) percent is chosen as the interval.

\begin{table}[h]
\centering
\caption{Peak load and inflection point of the load-displacement}
\begin{tabular}{|c|c|c|c|c|c|c|c|c|c|c|c|}
\hline
No & Sample & Replicate 1 & Replicate 2 & Replicate 3 \\
\hline
& & Peak Load & Inflection & & Peak Load & Inflection & & Peak Load & Inflection & \\
& & (Kn) & Ratio (%) & & (Kn) & Ratio (%) & & (Kn) & Ratio (%) \\
\hline
1 & 12-C & 18 & 14 & 0.78 & 21 & 15 & 0.71 & 22 & 16 & 0.74 \\
2 & 12-R & 33 & 17 & 0.78 & 36 & 30 & 0.84 & 29 & 24 & 0.82 \\
3 & 12-1-S & 28 & 17 & 0.60 & 25 & 16 & 0.64 & 30 & 23 & 0.76 \\
4 & 12-2-S & 25 & 19 & 0.76 & 29 & 17 & 0.6 & 27 & 20 & 0.72 \\
5 & 12-3-S & 19 & 14 & 0.76 & 21 & 13 & 0.64 & 19 & 14 & 0.72 \\
6 & 12-1-M & 28 & 19 & 0.68 & 32 & 23 & 0.72 & 44 & 26 & 0.60 \\
7 & 12-2-M & 22 & 13 & 0.60 & 24 & 17 & 0.72 & 34 & 22 & 0.64 \\
8 & 12-3-M & 20 & 13 & 0.68 & 24 & 17 & 0.72 & 22 & 14 & 0.64 \\
9 & 12-1-L & 24 & 19 & 0.76 & 19 & 11 & 0.60 & 22 & 16 & 0.72 \\
10 & 12-2-L & 18 & 12 & 0.68 & 24 & 15 & 0.60 & 21 & 14 & 0.64 \\
11 & 12-3-L & 16 & 11 & 0.68 & 18 & 14 & 0.76 & 17 & 11 & 0.64 \\
12 & 9-C & 27 & 19 & 0.71 & 25 & 20 & 0.80 & 24 & 17 & 0.73 \\
13 & 9-R & 38 & 29 & 0.74 & 46 & 40 & 0.86 & 33 & 27 & 0.82 \\
14 & 9-1-S & 32 & 27 & 0.60 & 35 & 26 & 0.76 & 30 & 18 & 0.60 \\
15 & 9-2-S & 24 & 19 & 0.60 & 20 & 13 & 0.64 & 23 & 14 & 0.60 \\
16 & 9-3-S & 21 & 19 & 0.68 & 21 & 14 & 0.68 & 25 & 18 & 0.72 \\
17 & 9-1-M & 22 & 18 & 0.72 & 31 & 21 & 0.68 & 25 & 15 & 0.60 \\
18 & 9-2-M & 25 & 18 & 0.72 & 32 & 24 & 0.76 & 27 & 18 & 0.64 \\
19 & 9-3-M & 23 & 17 & 0.76 & 19 & 14 & 0.72 & 21 & 15 & 0.68 \\
20 & 9-1-L & 32 & 27 & 0.64 & 26 & 19 & 0.72 & 26 & 16 & 0.64 \\
21 & 9-2-L & 22 & 18 & 0.64 & 27 & 18 & 0.68 & 24 & 16 & 0.68 \\
22 & 9-3-L & 17 & 14 & 0.64 & 21 & 13 & 0.64 & 18 & 13 & 0.72 \\
\hline
\end{tabular}
\end{table}
Table 10. IDEAL-CT data

<table>
<thead>
<tr>
<th>No</th>
<th>Sample</th>
<th>Replicates</th>
<th>Ave</th>
<th>COV</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>12-C</td>
<td>124 146 148</td>
<td>139</td>
<td>10</td>
</tr>
<tr>
<td>2</td>
<td>12-R</td>
<td>33 36 29</td>
<td>33</td>
<td>11</td>
</tr>
<tr>
<td>3</td>
<td>12-1-S</td>
<td>58 53 63</td>
<td>58</td>
<td>9</td>
</tr>
<tr>
<td>4</td>
<td>12-2-S</td>
<td>66 75 52</td>
<td>64</td>
<td>18</td>
</tr>
<tr>
<td>5</td>
<td>12-3-S</td>
<td>60 66 61</td>
<td>62</td>
<td>5</td>
</tr>
<tr>
<td>6</td>
<td>12-1-M</td>
<td>53 60 83</td>
<td>65</td>
<td>24</td>
</tr>
<tr>
<td>7</td>
<td>12-2-M</td>
<td>58 63 88</td>
<td>70</td>
<td>23</td>
</tr>
<tr>
<td>8</td>
<td>12-3-M</td>
<td>55 68 63</td>
<td>62</td>
<td>11</td>
</tr>
<tr>
<td>9</td>
<td>12-1-L</td>
<td>47 50 42</td>
<td>46</td>
<td>9</td>
</tr>
<tr>
<td>10</td>
<td>12-2-L</td>
<td>44 60 52</td>
<td>52</td>
<td>15</td>
</tr>
<tr>
<td>11</td>
<td>12-3-L</td>
<td>43 49 46</td>
<td>46</td>
<td>7</td>
</tr>
<tr>
<td>12</td>
<td>9-C</td>
<td>146 133 129</td>
<td>136</td>
<td>7</td>
</tr>
<tr>
<td>13</td>
<td>9-R</td>
<td>36 43 31</td>
<td>37</td>
<td>16</td>
</tr>
<tr>
<td>14</td>
<td>9-1-S</td>
<td>58 60 65</td>
<td>61</td>
<td>6</td>
</tr>
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<td>15</td>
<td>9-2-S</td>
<td>75 70 60</td>
<td>68</td>
<td>11</td>
</tr>
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<td>16</td>
<td>9-3-S</td>
<td>70 53 61</td>
<td>61</td>
<td>14</td>
</tr>
<tr>
<td>17</td>
<td>9-1-M</td>
<td>55 77 63</td>
<td>65</td>
<td>17</td>
</tr>
<tr>
<td>18</td>
<td>9-2-M</td>
<td>64 83 71</td>
<td>73</td>
<td>13</td>
</tr>
<tr>
<td>19</td>
<td>9-3-M</td>
<td>66 54 60</td>
<td>60</td>
<td>10</td>
</tr>
<tr>
<td>20</td>
<td>9-1-L</td>
<td>61 50 49</td>
<td>53</td>
<td>12</td>
</tr>
<tr>
<td>21</td>
<td>9-2-L</td>
<td>53 66 59</td>
<td>59</td>
<td>11</td>
</tr>
<tr>
<td>22</td>
<td>9-3-L</td>
<td>46 56 49</td>
<td>50</td>
<td>10</td>
</tr>
</tbody>
</table>

Figure 16. CT$_{index}$ values and improvement percentage (compared to R-12, R-9) for fiber-reinforce samples
Table 10 presents the $C_{T\text{index}}$ values calculated for all samples. Inflection point of load-displacement curve was assumed at $\text{PPP}_{69}$ and $\text{PPP}_{75}$ for fiber-reinforced samples and control samples, respectively. As presented in Table 10 the virgin control samples (12-C, 9-C) showed the highest cracking resistance. On the other hand, the $C_{T\text{index}}$ of rejuvenator 100% RAP without fiber was dramatically lower than the virgin control samples. However, by adding 0.10% fiber, the $C_{T\text{index}}$ was increased by almost 70 percent. The highest improvements were for 12-2-S, 12-2-M, and 9-2-M samples with 96, 113, and 98% increases in $C_{T\text{index}}$. The same pattern for mixture with NMAS 12.5 was observed for NMAS 9.5. The sample with 0.30% of fiber showed the maximum $C_{T\text{index}}$ among samples made of 100% RAP.

5.1.2 Evaluating variables' effect on CT-Index

A Two-Factor ANOVA with a significance level ($\alpha$) of 0.05 was conducted to investigate the effect of each variable (NMAS, FL, FC) on the $C_{T\text{index}}$. Results showed P-value less than 0.05 for fiber length and content. This indicates that the effect of these two parameters is significant intermediate cracking resistance. However, P-value is more than 0.05 for NMAS which indicates an insignificant effect of this parameter on $C_{T\text{index}}$.

\begin{table}[h]
\centering
\begin{tabular}{|l|c|c|c|}
\hline
Variables & $F$ & $P$-value & $F$-crit \\
\hline
NMAS & 0.0007 & 0.978 & 4.494 \\
FL & 14.573 & 0.0015 & 4.256 \\
FC & 5.0937 & 0.0332 & 4.256 \\
\hline
\end{tabular}
\caption{Effect of parameters on $C_{T\text{index}}$}
\end{table}

According to the study result of Lou et al., optimum FL is correlated to the NMAS of the mixture. Based on their finding, larger NMAS, requires longer fibers. The effect of NMAS on cracking resistance was found to be insignificant in this study. However, for further
analysis, it was decided to use the ratio of NMAS to fiber length (NMAS/FL) instead of mere fiber length. Figure 25 shows that for all fiber percentages, the CT_index is maximum when NMAS/FL is in the range of 1.5-2. It is also in line with the findings of Lou et al. [50].

To figure out the correlation between CT_index as a dependent variable and independent variables (FC, NMAS/FL), a contour plot was created (Figure 17). Contours are graphical techniques for representing a 3-dimensional surface by plotting Z slices in a 2-dimensional format. It can help us to have an idea about how two variables could correlate with a dependent variable.

![Contour plot of NMAS/FL (ratio of NMAS-to-fiber length) and FC (Y axes) vs. CT-Index](image)

**Figure 17.** Contour plot of NMAS/FL (ratio of NMAS-to-fiber length) and FC (Y axes) vs. CT-Index

### 5.2 BBR (Thermal Cracking)

#### 5.2.1 Mixture Test

Raul et al. propose using the following values as high, intermediate, and low test temperatures using the BBR test on mixture [43]:

```markdown
5.2 BBR (Thermal Cracking)

#### 5.2.1 Mixture Test

Raul et al. propose using the following values as high, intermediate, and low test temperatures using the BBR test on mixture [43]:
```
Higher temperature (lower PG +22)
Intermediate temperature (lower PG +10)
Lower temperature (lower PG -2)

Due to the restrictions of the BBR machine at the FIU laboratory, it was not possible to load the sample of more than 4000 (mN). Thus, the time-temperature superposition principle was used to predict creep stiffness at lower temperatures using data of higher (-22+22 = 0°C), and intermediate (-22+10 = -12°C) temperatures. Data collected at 0°C and -12°C, can be shifted to the desired temperature [51].

The creep stiffness as a function of time was calculated using Euler- Bernoulli beam theory (Equation 98).

\[ S(t) = \frac{L^3}{48bh^3\delta(t)} \]

where \( S \) = creep stiffness \( P= \)constant load applied to the beam \( L= \)span length, \( h \) is the height of the beam, \( b \) is the width of the beam, and \( \delta(t) \) is the deflection of the beam at time

Stiffness was measured at 8, 15, 30, 60, 120, and 240 seconds after loading and at two temperatures of 0 and -12°C. Values for load, deformation, and stiffness of sample for all samples are presented in Appendix.

The amount of shifting required at each temperature is called the shift factor and is a constant by which the loading times at each temperature can be divided to give a reduced loading time for the master curve. The reduced time is calculated using the following Equation:

\[ t_r = \frac{t}{a(\tau)} \]

Equation 10
Where $t_r$ is reduced loading time, $t$ is the actual time of loading, and $a(T)$ is the shift factor for data measured at temperature $T$.

At the lower temperatures, the shift factor can be described by Equation 11. Shift factors at the -12°C reference temperature obtained from the shifting process are shown as a function of temperature in Figure 18.

$$\log a_t = a \left( \frac{1}{T} - \frac{1}{T_r} \right)$$  \hspace{1cm} \text{Equation 11}

Where $T_r$ is the reference temperature at which the master curve is constructed.

![Figure 18. Time-temperature shift factors used to create master curve for sample 12-C](image)

Assuming the same shift factor value between the higher and intermediate temperatures, the Christensen–Anderson–Marasteanu (CAM) model (Equation 12) was used to predict the creep stiffness at the reduced times equivalent to the real loading times at the lower temperature.

$$S(T_r, r) = S_{\text{glassy}} \left[ 1 + \left( \frac{t_r}{\lambda} \right)^\beta \right]^{-\kappa/\beta}$$  \hspace{1cm} \text{Equation 12}

Where $S_{\text{glassy}}$ is glassy modulus (MPa), $\beta$, $\kappa$, and $\lambda$ are regression coefficients.

Creep stiffness at temperatures different from reference temperature should be shifted to construct a single smooth line of the master curve. The relative error caused by the difference between measured stiffness ($S_1$) and stiffness obtained from the CAM model ($S_2$)) was determined by the following equation:
\[
\text{Error} = (S_1 - S_2)^2.
\]

Equation 13

“Solver” was used to minimize error and obtain fitting parameters as presented in Table 12.

<table>
<thead>
<tr>
<th>Mixture</th>
<th>Reference Temperature, ( ^\circ \text{C} )</th>
<th>Sg</th>
<th>( \lambda )</th>
<th>( \beta )</th>
<th>( \kappa )</th>
<th>( a_1 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>12-C</td>
<td>-22</td>
<td>1.40E+03</td>
<td>11.7</td>
<td>0.378</td>
<td>0.371</td>
<td>285.0232</td>
</tr>
</tbody>
</table>

The same procedure was used to obtain the other mixtures’ master curve. Figure 19 shows the creep compliance master curve of control and rejuvenated samples. Lower values of stiffness modules obtained from master curves indicated better performance. This is due to the lower value of thermal stress induced in pavement layers. As shown in Figure 19, the addition of the fiber lowers the stiffness at all reduced time. Further analysis of the effect of stiffness on thermal cracking resistance is explained in the following sections.

Figure 19. Creep stiffness master curve for control and fiber reinforced samples (fiber length = 6mm, reference temperature = -22, NMAS=12.5)
5.2.2 Binder Test

Only aging of mixtures in this study was short-term aging that occurs during the fabrication process. Thus, RTFO aging was used to the age binder. The test was conducted at -12 and -18 °C and Equations 14 and 15 were used to obtain critical temperature (S<300mpa and critical m-value >0.3 at 60th second of the loading) (AASHTO T313). Results are presented in Table 13.

\[ T_{C,S} = T_1 + \left( \frac{(T_1-T_2) \cdot (Log \ 300 - Log S_1)}{Log S_1 - Log S_2} \right) - 10 \]  \hspace{1cm} \text{Equation 14}

\[ T_{C,m} = T_1 + \left( \frac{(T_1-T_2) \cdot (0.300 - m_1)}{m_1 - m_2} \right) - 10 \]  \hspace{1cm} \text{Equation 15}

Where \( S_1 \) is creep stiffness at \( T_1 \), MPa, \( S_2 \) is creep stiffness at \( T_2 \), MPa, \( m_1 \) is creep rate at \( T_1 \), \( m_2 \) is creep rate at \( T_2 \), \( T_1 \) = temperature at which S and m passes, °C, and \( T_2 \) is temperature at which S and m fails, °C.

<table>
<thead>
<tr>
<th>Binder</th>
<th>Aging</th>
<th>Critical Temperature of</th>
<th>Critical Temperature of m-</th>
</tr>
</thead>
<tbody>
<tr>
<td>Virgin (control)</td>
<td>RTFO</td>
<td>-29.48</td>
<td>-30.26</td>
</tr>
<tr>
<td>Rejuvenated</td>
<td></td>
<td>-34.25</td>
<td>-28.6</td>
</tr>
</tbody>
</table>

To calculate m-value at desired point along the stiffness-time curve, following equations were used:

\[ |m(t)| = \frac{d[logS(t)]}{d[log(t)]} = B + 2C[log(t)] \]  \hspace{1cm} \text{Equation 16}

\[ logS(t) = a + b[log(t)] + c[log(t)]^2 \]  \hspace{1cm} \text{Equation 17}

where: \( S(t) \) is time-dependent flexural creep stiffness estimated using Equation 9, MPa; \( t \) is a time of loading, \( S \); \( a, b, c \) are regression coefficients; \( S(t) \) is time-dependent flexural creep stiffness, MPa.
5.2.3 Correlation Between Binder and Mixture

Creep stiffness and m-values resulting from the BBR test for the binder were related to the mixture at the binder’s critical temperature and at 8, 15, 30, 60, 120, and 240 seconds after loading. An example of the correlation for 12-R is presented in Figure 20. The linear regression was performed for all tested mixtures. Coefficient of determination ($R^2$) was all more than 0.80. This indicates a good correlation between the results obtained from the binder and mixture regarding their creep stiffness and m-values. The relationships are expressed by general formulas:

$$S_{mix} = A \times S_{binder} + B$$ \hspace{1cm} Equation 18

$$m_{mix} = C \times m_{binder} + D$$ \hspace{1cm} Equation 19

Where A, B, C, and D regression parameters for a given mixture and reference temperature. $S_{mix}$ is stiffness modulus of asphalt mixture obtained from the Bending Beam Creep test, $S_{binder}$ stiffness modulus of asphalt binder obtained from the BBR test; $m_{mix}$ the slope of creep curve of asphalt mixture obtained from the BBR test, $m_{binder}$ the slope of creep curve of asphalt binder obtained from the BBR test.

![Figure 20. Correlation between rejuvenated binder and 12-R Mixture A) stiffness, and B) m-value](image)
At the next step, the corresponding binder critical stiffness and relaxation rate were used to obtain the critical values for the mixture. Since the type of binder used in all of the mixtures modified by fibers were the same as the binder used for the RAP mixture without fiber (12-R and 9-R), their critical mixture stiffness and m-value were used for all of them. Critical stiffness of the 12-R and 9-R are 9000 and 8750 MPa, respectively. On the other hand, the m-value of 12-R and 9-R were obtained to be 0.53 and 0.62, respectively. Then critical temperature corresponding to critical S and m-values were calculated. Table 14 shows the critical temperature of each mixture. To be able to rank the mixture, Delta T<sub>c</sub> (ΔT<sub>c</sub>) method was used. ΔT<sub>c</sub> is a derived asphalt binder property that is recently introduced to the asphalt pavement industry and gaining attention for the last decade. ΔT<sub>c</sub> provides insight into the relaxation properties of the binder that can be related to non-load-related cracking distresses in asphalt pavement [52]. ΔT<sub>c</sub> represents the difference between the critical temperature at which the binder (mixture) is at the specific limit for S and m (Equation 20).

\[ ΔT_c = T_{c,S} - T_{c,m} \]  

*Equation 21*

Where \( T_{c,S} \) is the critical temperature in which stiffness equals to 300MPa, and \( T_{c,m} \) is the critical temperature in which the m-value equals 0.30.

It is axiomatic that if the binder at a certain temperature is stiffer then it is more susceptible to cracking. However, since the asphalt binder/mixture shows a viscoelastic behavior, it can relax the applied load. In another word, if given enough time, the asphalt binder can shed the stress caused by an applied load. The load can be either induced by traffic or fluctuation of temperature. Therefore, a sign of ΔT<sub>c</sub>, either negative or positive, indicates whether the performance of the bind/mixture is governed by its creep stiffness (S) or creep
rate (m). S-controlled samples fail the 300 MPa limit at a warmer temperature than the m-value temperature. On the other hand, m-controlled samples fail the 0.300 m-value at a warmer temperature compared to \( T_{e,S} \).

Table 14. BBR Critical creep test results of mixture correspond to binders’ critical values at critical temperature (at load time of 60s)

<table>
<thead>
<tr>
<th>#</th>
<th>Mixture</th>
<th>( T_{c,S} ), °C</th>
<th>( T_{c,m} ), °C</th>
<th>( \Delta T_c ), °C</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>12-1-S</td>
<td>-36.54</td>
<td>-33.25</td>
<td>-3.29</td>
</tr>
<tr>
<td>4</td>
<td>12-2-S</td>
<td>-38.36</td>
<td>-36.85</td>
<td>-1.51</td>
</tr>
<tr>
<td>5</td>
<td>12-3-S</td>
<td>-42.54</td>
<td>-37.2</td>
<td>-5.34</td>
</tr>
<tr>
<td>6</td>
<td>12-1-M</td>
<td>-37.12</td>
<td>-36.84</td>
<td>-0.28</td>
</tr>
<tr>
<td>7</td>
<td>12-2-M</td>
<td>-37.89</td>
<td>-38.84</td>
<td>0.95</td>
</tr>
<tr>
<td>8</td>
<td>12-3-M</td>
<td>-43.2</td>
<td>-44.2</td>
<td>1</td>
</tr>
<tr>
<td>9</td>
<td>12-1-L</td>
<td>-38.25</td>
<td>-33.36</td>
<td>-4.89</td>
</tr>
<tr>
<td>10</td>
<td>12-2-L</td>
<td>-37.56</td>
<td>-38.65</td>
<td>1.09</td>
</tr>
<tr>
<td>11</td>
<td>12-3-L</td>
<td>-40.25</td>
<td>-36.85</td>
<td>-3.4</td>
</tr>
<tr>
<td>14</td>
<td>9-1-S</td>
<td>-33.12</td>
<td>-30.23</td>
<td>-2.89</td>
</tr>
<tr>
<td>15</td>
<td>9-2-S</td>
<td>-34.83</td>
<td>-33.25</td>
<td>-1.58</td>
</tr>
<tr>
<td>16</td>
<td>9-3-S</td>
<td>-37.36</td>
<td>-37.85</td>
<td>0.49</td>
</tr>
<tr>
<td>17</td>
<td>9-1-M</td>
<td>-39.36</td>
<td>-35.28</td>
<td>-4.08</td>
</tr>
<tr>
<td>18</td>
<td>9-2-M</td>
<td>-40.58</td>
<td>-41.83</td>
<td>1.25</td>
</tr>
<tr>
<td>19</td>
<td>9-3-M</td>
<td>-43.5</td>
<td>-43.23</td>
<td>-0.27</td>
</tr>
<tr>
<td>20</td>
<td>9-1-L</td>
<td>-40.58</td>
<td>-36.28</td>
<td>-4.3</td>
</tr>
<tr>
<td>21</td>
<td>9-2-L</td>
<td>-38.58</td>
<td>-38.45</td>
<td>-0.13</td>
</tr>
<tr>
<td>22</td>
<td>9-3-L</td>
<td>-38.89</td>
<td>-36.2</td>
<td>-2.69</td>
</tr>
</tbody>
</table>

As presented in Table 14 and Figure 21, the mixture containing fiber reaches the critical stiffness at a lower temperature compared to the control mixture. This indicates the addition of fiber, decreases the stiffness of the mixtures at the same temperature when compared to control samples. As a result, the critical temperature associated with the stiffness increases. More importantly, the addition of fiber also has increased the ability to shed stress at a lower temperature. The absolute value of \( T_{c,m} \) in most of the mixtures are more than the \( T_{c,S} \).
Figure 21. Critical temperature of stiffness (Ts) and m-value (Tm) for all mixture, ΔT is also provided for each pair of the Tm and Ts

5.2.4 Evaluating variables' effect on ΔTc

A Two-Factor ANOVA with a significance level (α) of 0.05 was conducted to investigate the effect of each parameter on the critical temperatures. NMAS and fiber length showed a P-value of more than 0.05 for Tc,S. This indicates that the effect of these two parameters is not significant on the critical temperature of the stiffness. However, at any length of the fiber, an increase in the fiber content increased the Tc,S. But for Tc,m, P-value is less than 0.05 for both fiber length and fiber content which indicates the significant effect of these two parameters on the rate of relaxation of the mixture. However, NMAS effect on the Tc,m seems to be insignificant.

Table 15. Effect of parameters on Tc,S, Tc,m, and ΔTc

<table>
<thead>
<tr>
<th></th>
<th>Tc,S</th>
<th></th>
<th></th>
<th>Tc,m</th>
<th></th>
<th></th>
<th>ΔTc</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>F</td>
<td>P-value</td>
<td>F-crit</td>
<td>F</td>
<td>P-value</td>
<td>F-crit</td>
<td>F</td>
<td>P-value</td>
<td>F-crit</td>
</tr>
<tr>
<td>NMAS</td>
<td>1.00</td>
<td>0.345</td>
<td>5.31</td>
<td>0.67</td>
<td>0.434</td>
<td>5.31</td>
<td>0.03</td>
<td>0.852</td>
<td>5.31</td>
</tr>
<tr>
<td>FL</td>
<td>3.78</td>
<td>0.063</td>
<td>4.25</td>
<td>10.71</td>
<td>0.004</td>
<td>4.25</td>
<td>3.09</td>
<td>0.095</td>
<td>4.25</td>
</tr>
<tr>
<td>FC</td>
<td>5.31</td>
<td>0.029</td>
<td>4.25</td>
<td>10.98</td>
<td>0.003</td>
<td>4.25</td>
<td>5.60</td>
<td>0.026</td>
<td>4.25</td>
</tr>
</tbody>
</table>
Figure 22 shows $\Delta T_c$ at each fiber content and for all FL. As it is shown, by increasing the fiber contents, the $\Delta T_c$ for all FL increases. Samples with no fiber (9-R, and 12-R) showed a $\Delta T_c$ less than -5.

The average rate of increase for all lengths of the fiber was approximately the same. This confirms the statistical analysis provided in Table 1 which shows the insignificant effect of fiber length on $\Delta T_c$. It indicates that at lower temperatures, the length of the fiber has less effect on the cracking resistance of the mixture. This finding verified the inference of [53].

![Figure 22. $\Delta T_c$ vs. fiber content for all mixtures (S:3mm, M: 6mm, L:12mm fiber length)](image)

In this research study, only positive $\Delta T_c$ are considered as the mixtures that fail the low-temperature performance. As mentioned before, positive $\Delta T_c$ indicates that the mixture is m-controlled. It should be reminded that the mixtures were only RTFO aged. Therefore, it is expected that after samples are gone through long-term aging, the $\Delta T_c$ significantly decreases. Furthermore, most of the samples pass the criteria of $\Delta T_c > 0$ when the fiber content is around 0.30%. This is the same fiber content at which $CT_{\text{index}}$ is maximum. This helps with designing a balanced mixture.
5.3 APA (Rutting)

Rutting depth of 7 mm (0.28 in.) at 8000 passes is recommended by many agencies to use as pass/fail criteria for asphalt mixture [46].

Table 17 and Figure 23 show the average rut depth at 8,000 passes, along with the COV. An average of three tests was used for each evaluation. The average COV is 13.2%, with a range of 5.9-23.2%.

![Figure 23](image-url)

*Figure 23. Rutting depth obtained from conducting APA test at 64°C Wet.*

The virgin mixture for both NMAS of 9.5 and 12.5 exhibited the highest amount of rut depth compared to the recycled mixtures. Although the high PG of all mixtures was the same, the rutting performances were different. This is obviously due to the different content of fiber and its effect on improving the resistance against permanent deformation. This was also observed in a past research study conducted by Behbahani et al. [54].

<table>
<thead>
<tr>
<th>Table 16. Effect of parameters on rutting depth</th>
</tr>
</thead>
<tbody>
<tr>
<td>Variables</td>
</tr>
<tr>
<td>----------------</td>
</tr>
<tr>
<td>NMAS</td>
</tr>
<tr>
<td>Fiber length</td>
</tr>
<tr>
<td>Fiber Content</td>
</tr>
</tbody>
</table>
Table 17. APA rutting test results

<table>
<thead>
<tr>
<th>Mixture</th>
<th>Fiber (%)</th>
<th>Fiber Length, (mm)</th>
<th>Average Rut Depth at 8,000 Passes (mm)</th>
<th>COV</th>
</tr>
</thead>
<tbody>
<tr>
<td>12-C</td>
<td>0%</td>
<td>-</td>
<td>7.1</td>
<td>7.1</td>
</tr>
<tr>
<td>12-R</td>
<td>0%</td>
<td>-</td>
<td>5.4</td>
<td>8.9</td>
</tr>
<tr>
<td>12-1-S</td>
<td>0.15%</td>
<td>3</td>
<td>4.7</td>
<td>12.7</td>
</tr>
<tr>
<td>12-2-S</td>
<td>0.30%</td>
<td>3</td>
<td>4.3</td>
<td>25.0</td>
</tr>
<tr>
<td>12-3-S</td>
<td>0.60%</td>
<td>3</td>
<td>7.1</td>
<td>6.0</td>
</tr>
<tr>
<td>12-1-M</td>
<td>0.15%</td>
<td>6</td>
<td>5.4</td>
<td>7.0</td>
</tr>
<tr>
<td>12-2-M</td>
<td>0.30%</td>
<td>6</td>
<td>4.5</td>
<td>21.7</td>
</tr>
<tr>
<td>12-3-M</td>
<td>0.60%</td>
<td>6</td>
<td>6.4</td>
<td>6.8</td>
</tr>
<tr>
<td>12-1-L</td>
<td>0.15%</td>
<td>12</td>
<td>5.7</td>
<td>6.9</td>
</tr>
<tr>
<td>12-2-L</td>
<td>0.30%</td>
<td>12</td>
<td>5.3</td>
<td>17.0</td>
</tr>
<tr>
<td>12-3-L</td>
<td>0.60%</td>
<td>12</td>
<td>7.4</td>
<td>13.9</td>
</tr>
<tr>
<td>9-C</td>
<td>0%</td>
<td>-</td>
<td>6.5</td>
<td>15.2</td>
</tr>
<tr>
<td>9-R</td>
<td>0%</td>
<td>-</td>
<td>4.8</td>
<td>15.6</td>
</tr>
<tr>
<td>9-1-S</td>
<td>0.15%</td>
<td>3</td>
<td>4.9</td>
<td>20.0</td>
</tr>
<tr>
<td>9-2-S</td>
<td>0.30%</td>
<td>3</td>
<td>4.9</td>
<td>8.6</td>
</tr>
<tr>
<td>9-3-S</td>
<td>0.60%</td>
<td>3</td>
<td>6.1</td>
<td>22.4</td>
</tr>
<tr>
<td>9-1-M</td>
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<td>9-2-M</td>
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<td>6</td>
<td>4.8</td>
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<tr>
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<td>0.60%</td>
<td>6</td>
<td>6.8</td>
<td>13.4</td>
</tr>
<tr>
<td>9-1-L</td>
<td>0.15%</td>
<td>12</td>
<td>6</td>
<td>9.1</td>
</tr>
<tr>
<td>9-2-L</td>
<td>0.30%</td>
<td>12</td>
<td>5.5</td>
<td>5.3</td>
</tr>
<tr>
<td>9-3-L</td>
<td>0.60%</td>
<td>12</td>
<td>7.4</td>
<td>23.3</td>
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A one-way ANOVA with a significance level (α) of 0.05 was conducted to investigate the effect of each parameter on the rutting depth. The P-value of the NMAS and fiber length is more than 0.05. This indicates the insignificant effect of NMAS on rutting. On the other hand, fiber content seems to have a significant effect on controlling the rutting depth. The virgin (control) mix had an average rut depth of 6.8 mm. 9-R and 12-R samples showed an average rutting depth of 5.1mm. The average rut depth for all mixtures with 0.10%, 0.3%, 0.6% fiber content were 5.3, 4.8, and 6.8mm, respectively. Thus, samples with 0.3% showed the best rutting performance.
CHAPTER VI. SUMMARY AND CONCLUSION

This chapter presents a summary of the procedures, analysis, and results of the research study. The objective of the study was to evaluate the performance of a 100%RAP mixture rejuvenated with WO and reinforced with basalt fiber. The study focused on the effect of fibers on cracking resistance, and permanent deformation susceptibility. The first Section of this chapter (Section 6.1) presents the effect of basalt fiber on performance of 100%RAP mixture. Then, in Section 6.2, the modified version of IDEAL-CT analysis method is presented to consider the shift of inflection in load-displacement curve. In Section 6.3 the method to evaluate the mixture thermal cracking resistance using BBR test is summarized. In Section 6.4, a performance-based balanced mix design which can incorporated rejuvenator and fibers is presented. Finally, in Section 6.5, recommended future studies are presented.

6.1 Effect of Fiber on Mixture Performance

The mixture design process used in this study was performance-based. Performance was evaluated for both binder and mixture. Binder tests were conducted to determine the rejuvenator content that satisfies the required high and low PG, and mixture tests were conducted to evaluate the fiber effect on the mixture performance.

Results showed that $\Delta T_c$ and rutting depth are not affected by the length of the fiber. P-value of FL for $\Delta T_c$ and rutting depth were 0.09 and 0.06 which indicates an insignificant effect of FL on these parameters. However, at intermediate temperature, both FL an FC showed a significant effect with P-value of 0.0015 and 0.032, respectively. It is speculated that at low or high temperatures, the bonding strength that develops between fibers and mixture matrix, is almost equivalent. Thus, the effect of the length of the fiber would be
negligible on the mixture’s performance. Alternatively, CT\textsubscript{index} was found to be a function of both content and length of the fiber (Section 5.1.2).

Figure 28 shows the range of fiber content in which the mixture meets the thermal cracking and rutting criteria. As illustrated, thermal cracking for FC $>0.4\%$, meets the criteria of $\Delta T_c > 0$, and for FC$<0.61\%$, rutting is less than 7mm. On the other hand, although CT\textsubscript{index} for all FC and FL meets the criteria of 65 [55], the maximum CT\textsubscript{index} was obtained from NMAS/FL$=1.6$ and 0.3% FC. Therefore, estimation of the proper amount of the FL or FC depends on the climate of the region. Thus, designers should prioritize the performance requirements and corresponding criteria accordingly.

![Rutting Depth (mm) and thermal cracking parameter ($\Delta T_c$) vs. FC.](image1)

**Figure 24.** Rutting Depth (mm) and thermal cracking parameter ($\Delta T_c$) vs. FC.

![Fatigue cracking parameter (CT\textsubscript{index}) vs. NMAS/FL](image2)

**Figure 25.** Fatigue cracking parameter (CT\textsubscript{index}) vs. NMAS/FL
6.2 Proposed Procedure to Improve IDEAL-CT Test Results

As mentioned earlier, CT_{\text{index}} value was derived from Paris’ law and the cracking growth rate defined by Bazant et al. which describes macro-crack propagation. Therefore, it is logical to choose a part of the curve where the load is decreasing (post-peak) rather than the pre-peak segment where the load is increasing. As explained in Section 5.2.1, Zhou et al. propose calculating the average slope at the vicinity of PPP_{75} to compute CT_{\text{index}}. The absolute value of the slope of the load-displacement curve varies from small right after the peak load point to large in the early middle of the curve, and then becomes small again after the middle of the curve. Thus, using the inflection point is a mathematically sound concept to obtain the point the slope at which is maximum.

It was observed that the inflection point for fiber-reinforced samples occur at PPP_{69} with a standard deviation of 5. The same intervals used by Zhou et al. were used to compute the average slope at the inflection point. However, instead of using PPP_{75}, Equation 7 was used to compute the average slope. An example of a fitted equation and its first derivation is illustrated in Figure 15.

6.3 Proposed Procedure to Determine Thermal Cracking Criteria in BBR Test

One of the main concerns when using a high percentage of RAP is the low-temperature performance. Recently, BBR test on asphalt mixture gained attention in the asphalt pavement industry. There are few research studies about using BBR test to evaluate the low-temperature cracking resistance of the mixture. However, author did not find a method to define a critical value for a mixture at low temperature using the BBR test. The procedure used in Section 5.2 is summarized here as a proposed method to evaluate the thermal cracking resistance of the mixture using the BBR test:
**Step 1:** Measure the critical temperature of stiffness and m-value of asphalt binder ($T_{c,S}$, $T_{c,m}$) (Section 5.2.2)

Step 2: Measure creep stiffness and m-value of the mixture at the critical temperature of the binder and at 8, 15, 30, 60, 120, and 240 seconds of loading (Section 5.2.1).

**Step 3:** Find correlation between creep stiffness and m-value of binder and mixture (Section 5.2.3).

**Step 4:** Obtain critical stiffness and m-value of mixture ($S_c$, $m_c$) corresponding to binder’s critical stiffness (300 MPa) and m-value (0.30) (Figure 21).

**Step 5:** Use the mixture’s master curve, and its critical values ($S_c$, $m_c$) obtained in Step 4 to find the $T_{c,S}$ and $T_{c,m}$ of the mixture.

**Step 6:** In this study, $(\Delta T_c = T_{c,S} - T_{c,m}) > 0$ was proposed as a laboratory criterion for thermal cracking performance of the short term aged mixtures.

### 6.4. Performance-based Balanced Mix Design Implication

As per AASHTO PP 105-20 definition, balance mix design (BMD) is “asphalt mix design using performance tests on appropriately conditioned specimens that address multiple modes of distress taking into consideration mix aging, traffic, climate and location within the pavement structure”. According to the national asphalt pavement association, BMD should help designers to improve performance, enable innovation, and economic optimization [56]. The flowchart presented in Figure 26 is a proposed procedure to design fiber-reinforced 100%RAP mixtures. The mix design is based on a performance-based BMD approach. Each step of the flowchart is explained later in this section.
Figure 26. Proposed performance-based fiber-reinforced 100%RAP asphalt mix design
Step 1. Control material required properties and consistency

- In general, the RAP aggregates must meet the same quality requirements specified for virgin aggregates (ASTM D5821, AASHTO T 304, ASTM D 4791, AASHTO M 323, AASHTO T 176)
- Determine asphalt binder content and Adjust aggregate gradation (AASHTO T308 or AASHTO T 164, AASHTO T 30)
- Determine required content of rejuvenator based on Low and High-temperature PG
  i. Extract recycled binder from RAP material (AASHTO T164)
  ii. Blend different rejuvenator content (at least four) with an extracted binder from RAP. Use the past experiences or the manufactures recommendation to choose the percentages of rejuvenator in this step.
  iii. (High-temperature control) Conduct DSR test to determine the temperature at which rutting parameters \( \left( \frac{G^*}{\sin \delta} \right) \) of original and short-term aged asphalt binder are equal to 1.0 and 2.2, respectively. (AASHTO T 315-12)
  iv. (Low-temperature control) Conduct BBR test to determine the critical temperature at which \( S=300 \text{ KPa and } m\text{-value}=0.300 \) at the 60s of loading (AASHTO T 313-12).

Step 2. Determining Optimum Binder Content/Gradation/Binder PG

- Mix Production:
  Two batches of asphalt mix should be prepared. One with binder content of RAP mixture. And the other with 0.5 percent added neat binder.
RAP should be heated in an oven to reach 170°C. Then it should be pre-blended for 0.5 minutes. Next rejuvenator should be added directly to the mix at the required dosage and mixed for 1.5 min. For the batch containing an extra 0.5% binder, the addition of the binder should be followed by 3.5 minutes of mixing [57].

- **Sample fabrication and composition requirements:**

  The loose mixture should be compacted at 145°C immediately after it is prepared. AASHTO T 312-11 can be used to help designers to choose a $N_{\text{design}}$ appropriate for the traffic level (Table 6)

  Next, binder content, air void, voids in mineral aggregate (VMA), and the void filled with asphalt (VFA) of the prepared samples should be determined.

  Zaumanis et al. recognized that even fulfilling all the required standards regarding volumetric properties of mixtures containing a high percentage of RAP would not ensure the expected mixture performance [57]. Thus, mixture characteristics regarding constituent material and volumetric properties are merely collected to help designers with optimizing mixture performance by altering them for the next mixture trials.

- **Conduct performance tests and controls the requirements**

  1. **Rutting:**

     In this research study, APA rutting test was used and a rutting depth of 7mm was considered the critical value.

  2. **Crack propagation at intermediate temperature**
In this research study, the IDEAL-CT test at 25°C was used. The advantages of using this test are already explained in Section 3.2.1.

When plotting rutting and cracking performance results versus binder content, there are two possibilities:

(2-A): Range of acceptable rutting and acceptable cracking do not intersect.

Thus, for a given mixture, no binder content can meet both rutting and cracking performance requirements. A designer can alter the aggregates gradation or dose of rejuvenator to see the possibility of obtaining binder content satisfying both performance requirements. But if it was not possible, designers can choose to add fibers to meet performance requirements (Step 3).

(2-B): Range of acceptable rutting and acceptable cracking do intersect.

In this case, there is at least one value for binder content that meets both rutting and cracking requirements. However, designers still can choose to enhance the performance by adding fibers.

Step 3. Determining optimum fiber content

- Mix Production:

  RAP should be heated in an oven to reach 170°C. Then it should be pre-blended for 0.5 minutes. Next rejuvenator should be added directly to the mix at the required dosage and mixed for 1.5 min. Fiber can slightly decrease the binder content and improve the rutting resistance. The maximum possible binder content obtained in steps 2-B should be used. Finally, fiber should be added followed by 3.5 minutes of mixing. If any additional binder is required, it should be added along with fiber.
To select proper FC and FL at this stage, it is recommended designers refer to past experiences and studies about each type of fiber they want to use. This study provides information about the appropriate range of Basalt fiber in a 100%RAP mixture rejuvenated with WO (Section 6.4).

- **Sample fabrication and composition requirements:**
  The same procedure described in Step 2 is applicable here.

- **Conduct performance tests and control the requirements**
  i. Crack propagation at intermediate temperature (Fatigue Cracking)
     Same as Step 2. However, in this study, it was observed that IDEAL-CT test results analysis requires some modification to reflect the effect of fiber. This modification is explained in Section 6.2.
  
  ii. Crack propagation at low temperature (Thermal Cracking)
     In this research study, the BBR test was conducted to measure the thermal cracking resistance of the mixture. A laboratory criterion was also defined in this study to determine acceptable mixture performance (Section 6.3).
  
  iii. Rutting:
     Same as Step 2.

Designers can use Figures 17, 25, and 26 to realize how an increase or decrease in FC and FL can affect the performance and decide how to change them for the next trial batch. Furthermore, designers should prioritize the performance requirements and adjust designing criteria according to the regional climate.

Step 4. Conduct moisture damage tests
As a final step, moisture damage tests are also recommended to see a need for anti-stripping agents.

6.5 Recommendations for Future Studies

- The bond between the aggregate and asphalt binder film may be lost due to the presence of water or cause stripping. It is recommended to study the effect of fibers on stripping in future studies.

- Rejuvenator content was constant in this study. However, using various rejuvenator content and types is recommended for future studies.

- The critical temperature of the mixture introduced in this study is relative and limited to the data of the study. Future research studies, critical values of creep stiffness, and the relaxation rate of the mixture can be obtained from field evaluation. In other words, the proposed criteria could be used for comparison purposes but do not necessarily provide insight to field performance. Future studies should work on finding a correlation between the critical temperature using the proposed method and data obtained from the field.

- Binder and mixture used in this study were only short-term aged. It is recommended to use a long-term aged mixture in future studies to evaluate mixture thermal cracking susceptibility.
LIST OF REFERENCES


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**Education**

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**Journal Publication**


