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Laboratory evaluation of eco-friendly additives for warm in- place recycling technology

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Laboratory Evaluation of Eco-Friendly Additives for Warm In-Place Recycling Technology

by

Ashkan Bozorgzad

A thesis submitted in partial fulfillment
of the requirements for the Doctor of Philosophy
degree in Civil and Environmental Engineering in the
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CERTIFICATE OF APPROVAL

PH.D. THESIS

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This dissertation is

dedicated to my beloved

family for their support

and encouragement

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ABSTRACT

Hot and cold in-place recycling are two of the economical and well-known pavement rehabilitation strategies that can be used to recycle existing pavements. Two main limitations of hot in-place recycling (HIR) are high mixing and compaction temperatures, and a low percentage of using recycling materials in new mixtures. For cold in-place recycling (CIR), damaging the aggregate gradation of milled mixtures and low quality of final mixtures are major restrictions.

In this study, eco-friendly additives to be used for warm in-place recycling technology (WIR) are proposed. To make this mix, two WIR additives were developed to decrease the mixing and compaction temperature while increasing increase the RAP percentage in Warm in-place recycling (WIR) process.

In order to evaluate the products, the mixing and compaction temperatures of mixture were measured and the performance of three different mixture types with three different percentages of RAP were evaluated. The Superpave test was conducted to identify the optimum dosage of two WIR additives and for additive 1 and 2, 3.99% and 7.77% were identified as optimum dosages respectively. To verify this optimum dosage, the second group of Superpave tests were conducted on modified asphalt binder sample and the results had acceptable correlation with first set of rheological tests. The Multiple Stress Creep Recovery (MSCR) and Fourier Transform Infrared (FTIR) Spectroscopy tests were done on modified asphalt binder with WIR additives to better understand the effects of additives on recoverability, rutting resistance and aging mechanism of modified asphalt binder.

The performance properties of mixtures containing 30, 70 and 100 percent RAP material which modified with optimum percentage of two different additives were

investigated. Hamburg Wheel Tracking test (HWT) was conducted to evaluate the moisture susceptibility and rutting performance of modified mixture. According to this test results, none of the modified mixtures with additives did not have the moisture susceptibility and rutting problems. Although, additive 2 made the mixture softer than additive 1, but it did not have a significant effect on rutting. Disc-Shaped Compacted Tension (DCT) test was considered to evaluate the fracture properties of mixture at low temperature. Test result showed that additive 2 was more effective in lowering the low-temperature cracking and modified mixtures containing 100% RAP material have acceptable fracture energy.

PUBLIC ABSTRACT

Pavement recycling is a growing technology for practical and economic rehabilitation of existing paved roadways. A conventional pavement rehabilitation method requires virgin aggregates and bituminous materials to replace the deteriorated asphalt with new asphalt mixtures after milling. Due to high costs of virgin materials, pavement recycling has become more preferred and practiced in the industry. Hot in-place recycling (HIR) and cold in-place recycling (CIR) are two economical pavement rehabilitation strategy that can be used to maintain the good condition of the existing pavement through recycling existing pavements.

Two major limitations of HIR are high mixing and compaction temperatures and low percentage of using recycling materials in new mixtures. For cold in-place recycling (CIR), damaging the aggregate gradation of milled mixtures and low quality of final mixtures are two major restrictions. In this study, warm in-place recycling technology (WIR) are developed by using eco-friendly additives and a new method of existing pavements scarification. In order to evaluate the products, the mixing and compaction temperatures of mixture were measured and the performance of three different mixture types with three different percentages of RAP were evaluated.

According to this test results, the mixing and compaction temperatures of modified asphalt mixture were decreased by about 20ºC for each of the additives by adding 4% of additive 1 and 8% of additive 2 which can save a huge amount of energy and preserve the enviroument. It should be mentioned that, the percentage of recycled material can be increased up to 70% in this new Warm In-place recycling method.

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CHAPTER 1: INTRODUCTION

Pavement recycling is a growing technology for practical and economic rehabilitation of existing paved roadways in the US. A conventional pavement rehabilitation method requires virgin aggregates and bituminous materials to replace the deteriorated asphalt with new asphalt mixtures after milling. Pavement recycling uses the recycled materials with minimal amounts of virgin materials. Due to high costs of virgin materials, pavement recycling has become more preferred and practiced in the industry. Also, recent advanced technologies improved the equipment and procedures from the past generations.

Hot in-place recycling (HIR) is an economical pavement rehabilitation strategy that can be used to maintain the good condition of the existing pavement through recycling existing pavements. The process of HIR is to 1) soften the pavement surface distress with heat, 2) remove the existing surface, 3) mix it with an asphalt binder and additional virgin aggregate and 4) replace the recycled material on the original pavement site. HIR can be performed in either single-pass operation or two/multiple–pass operation. Single-pass operation recombines the restored reclaimed asphalt pavement (RAP) with virgin material. On the other hand, two/multi-pass operation adds a new wearing surface immediately after compacting the RAP materials[1].

1.1 Problem Statement

Hot in-place recycling (HIR) process was developed to replace conventional milling and resurfacing to minimize costs and environmental impacts. According to the survey conducted by NCHRP, 34 out of 45 responding state DOTs have tried HIR but only 21 states continue to use it. The reasons for reluctance to use HIR were cost overruns and poor performance experiences. Also, past problematic projects showed that the root of the problem was often from poor project selection [2], [3].

Two major limitations of HIR are high mixing and compaction temperatures and low percentage of using recycling materials in new mixtures. For cold in-place recycling (CIR), damaging the aggregate gradation of milled mixtures and low quality of final mixtures are two major restrictions.

1.2 Research Motivation and Objectives

According to the previous investigations, the usage of warm mix additives and rejuvenators is desired to decrease the mixing and compaction temperatures while increasing the RAP percentage in Warm in-place recycling (WIR) process. In this study, eco-friendly additives to be used for warm in-place recycling technology (WIR) are proposed. These products can be used as a wearing course, a surface layer, an intermediate layer and a base layer of asphalt pavements. In order to evaluate the products, the mixing and compaction temperatures of mixture were measured and the performance of three different mixture types with three different percentages of RAP were evaluated.

The main objectives of this study were twofold. The first one is determination of the optimum dosage of two warm-rejuvenator additives by measuring the rheological properties of asphalt binders modified by them. The second objective of this study is Evaluation of the mechanical performances of modified mixtures with the optimum dosage of additives containing different percentages of RAP.

Rheological properties of a modified asphalt binder were evaluated to: 1) identify the optimum dosage of WIR additives and 2) evaluate the effects of these additives on rheological and mechanical properties of asphalt binders. Superpave binder test was conducted on virgin, aged and modified asphalt binder to comprehensively investigate the effects of additives on asphalt binders. These tests and are:

a) Rotational Viscosity (RV) test: one of the main goals of adding WIR additives to asphalt binder in this study is to decrease the compaction and mixing temperature of final modified mixtures. This test can determine these temperatures by measuring the viscosity using a rotation viscometer.

b) Rolling Thin Film Oven (RTFO) test: to simulate the short-term aging of asphalt binders during the construction period.

c) Pressure Aging Vessel (PAV) test: to simulate the long-term aging of asphalt binders during the serving life. To simulate the aging condition of RAP materials in mixtures, two cycles of PAV were run on some samples.

d) Dynamic Shear Rheometer (DSR) test: for virgin and RTFO aged asphalt binders, to evaluate the high-temperature performance of asphalt binders and for PAV aged asphalt binders, to evaluate the fatigue performances of asphalt binder during the service life. In addition, DSR test result can be used to determine the maximum percentage of additives that can be added to the asphalt binder.

e) Bending Beam Rheometer (BBR) test: to evaluate the low-temperature performance of asphalt binders. Additionally, BBR test result can be used to identify the minimum (optimum) dosage of additives.

The mechanical performance of mixtures modified by optimum dosage of WIR additives was evaluated in this study. Mixtures with the optimum dosage of additives containing three different percentages of RAP were compacted and tested through the following related performance tests:

a) Hamburg Wheel Tracking (HWT) test: to evaluate the rutting and moisture susceptibility of modified mixtures.
b) Disk-Shaped Compact Tension (DCT) test: to evaluate the thermal fracture properties of modified asphalt mixtures at low temperature.

1.3 Dissertation Outline

In **Chapter 1,** the significance of in-place recycling of mixtures is described and two main methods of in-place recycling are introduced. Furthermore, the limitations of each method are stated along with the research motivations and objectives. In **Chapter 2,** the background and procedure of HIR is presented. Next, the background, advantages, and disadvantages of using different HIR machines and facilities are described. Finally, the advantages and recommendations to implement Warm In-place recycling is presented in this chapter.

Chapter 3 and **Chapter4** focus on experimental tests on two WIR additives. **Chapter 3** presents the rheological test results on the modified asphalt binder with two additives. The optimum percentages of additives and rheological properties of the modified asphalt binders are presented in this chapter. **Chapter 4** discusses the performance evaluation of modified mixtures with WIR additives containing three different percentages of RAP. **Chapter 5** presents a summary of findings and conclusions derived through this research, as well as recommendations.

CHAPTER 2: BACKGROUND

The idea of pavement recycling system has been around for many years. In the United States, the first asphalt pavement was placed in 1870 [4], [5]. Reuse of asphalt pavement in 1915 demonstrated the importance of pavement rehabilitation[6], [7]. However, it was not until the 1930s or 1940s when the recycling was more widely accepted and practiced with the first heater planer machines [8]–[10]. Since then, recycling of asphalt pavements has been practiced in the US for more than 80 years. In the early years, recycling asphalt pavements was considered excessive and costly but such impression soon changed with the improved technology and productivity. Today, highway engineers consider recycling asphalt pavement using several methods such as both hot and cold in-place recycling methods. Among several methods of asphalt recycling, hot and cold in-place recycling (HIR) have been considered the most prevalent methods in North America in the past decades[11].

Prior to HIR construction, proper project selection is essential in order to achieve the desired results and satisfactory performance. To execute HIR construction, the pavement must be structurally stable for the expected traffic level and there shouldn't be high levels of distress below about two inches [12], [13]. Also, it is important to ensure that the existing pavement can withstand the weight of heavy HIR equipment, especially when the HIR construction is on low-traffic roads. The HIR process rejuvenates only upper surfaces (typically two inches) of asphalt. Therefore, this process is not for correcting any base or subgrade failures. Structural inadequacies cannot be addressed by HIR [12], [14].

HIR is usually performed to a depth of 25 mm (1 in) and possibly from 20 mm to 50 mm (2in). According to Asphalt Recycling and Reclamation Association, three types of hot in-place recycling techniques can be mentioned; surface recycling, repaving, and remixing. Recycling agents for rejuvenating the aged asphalt binder could be added in all three types. However, the virgin aggregate is used only in repaving and remixing process [1], [15].

2.1 Hot In-Place Recycling Equipment

Hot In-Place Recycling is a process of correcting asphalt pavement surface distress by softening the existing surface with heat, mechanically removing the pavement surface, mixing it with an asphalt binder. It may add virgin aggregate, which would replace the recycled materials. The equipment used for this process varies between the different methods of Hot In-Place Recycling; three main methods of this practice are heater-scarification, repaving, and remixing [16], [17].

2.1.1 Preheater

The start of the Hot In-Place Recycling train, the preheater, is towed behind a supply truck. Rubber tire wheels are used for transporting the equipment to and from the worksite, but steel tire wheels replace these during operation. The main purpose of this step is to remove excess surface moisture and begin the pavements heating process with horizontally fired, heaters. Propane burners are fired approximately one foot above the existing pavement and have the capability to heat the entire lane width [16], [18].

2.1.2 First Heater/Miller (Unit A)

This self-propelled piece of equipment utilizes propane fired heaters similar to the preheater, two five-foot-wide rotating milling heads, and a rejuvenating tank. First, the heaters heat the full lane to a depth of 1 to 1.25 inches. Once heated, Unit A’s rotating milling heads remove and windrow the upper inch of the softened asphalt from both sides into the center of the lane where rejuvenator is added [16], [19].

2.1.3 Second Heater/Miller (Unit B)

Unit B is pushed by a track paver. It utilizes a four-foot milling head, a drag-slat conveyor, heaters, a 12-foot milling head, a front mounted hopper, and a pugmill mixer. Its four-foot milling head removes material in the center of the lane. Then, the drag-slat conveyor picks up this material along with the windrowed material from Unit A and drops it directly in front of the pugmill mounted on the end of Unit B. The front mounted hopper meters the new mixture or coated aggregate into the drag-slat conveyer. Unit B's heaters heat the freshly exposed asphalt to a depth of 1 to 1.25 inches to be removed by the 12-foot milling head. Next, the pugmill picks up the material from Unit A, Unit B, and newly added material, mixes it, and transports it to the paver hopper. The paver then lays the asphalt on the road [16].

2.1.4 Paver and Rollers

Conventional pavers and rollers finish the HIR Train. A track paver spreads the mix into the place while pushing Unit B. Rollers then follow closely behind the paver and compact the asphalt [16].

2.2 Surface Recycling Method and Equipment

Surface recycling is a rehabilitation process that collects cracked, brittle and irregular pavement from a final thin wearing course [20], [21]. Pavements with stable and adequate base are suitable for this process. The most common scarification depth starts from 20mm or 25mm to 50mm that can be achieved [22]–[24]. If a hot mix asphalt is separately placed after the surface recycling, the process is considered to be a two-pass method. Otherwise, the process is considered a single-pass method.

Surface recycling process is shown in Figure 2-1 [9]. The equipment consists of three main parts, including a pre-heating unit, a heating and recycling unit, and a rubber roller. The preheating unit applies heats to the old HMA pavement surface first. Then, heating and recycling unit applies more heats and scarifies the HMA pavement with a set of non-rotating teeth. Recycling agent is also applied during this process. Then, the old pavement and recycling materials are mixed with an auger and flattened off with a screed. Finally, Recycled mix is compacted by a rubber-tired roller. No virgin aggregates are added during the surface recycling process.

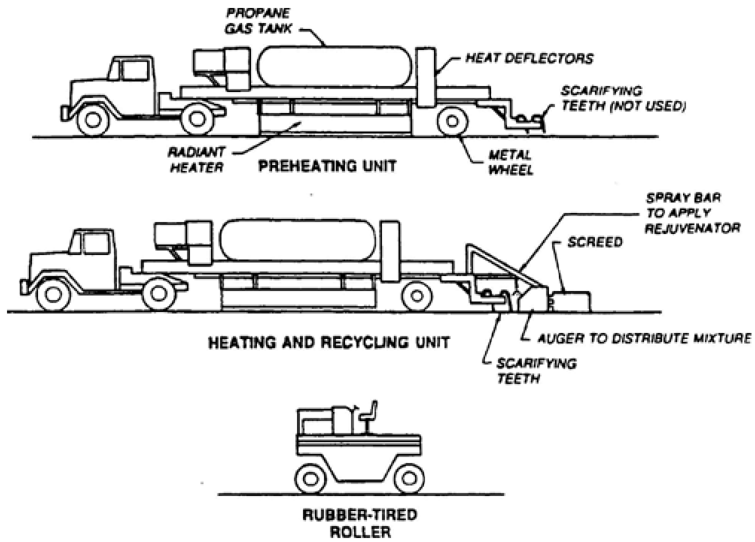


Figure 2-1 Surface Recycling method process [9]

Radiant or infrared heating, as opposed to direct flame, are commonly used to prevent damages to the asphalt cement binder and avoid unwanted emissions [4], [25], [26]. Propane is common fuel for this indirect heat [27], [28]. The heating process can be performed with one heating unit with two sets of heaters or two heating units that each unit has a single set of heater. At least, two sets of heaters are used for heating process. The temperature of the HMA pavement in this process is raised to 110 °C to 150 °C (230 °F to 302 °F) [29], [30]. The heated pavement is then scarified by multiple rows of spring loaded scarifiers. The spring loaded mounting is advantageous as it allows the scarifiers to pass over road obstacles including manhole cover and concrete patches. During the scarifying operation, added recycling agents soften the pavement that experienced oxidative hardening due to the long-term aging and heating during the recycling process [31], [32].

2.3 Repaving Method and Equipment

The repaving process includes correcting up to 25 mm depth of the pavement, adding a rejuvenator to improve asphalt properties, and simultaneously applying a thin overlay on top of the recycled layer. In other words, it is simply the surface recycling process followed by an overlay paving process. Pavements with minor problems such as minor rutting, shrinkage cracking, and raveling can be corrected by this method. The repaving process is necessary when the surface recycling process would not satisfy the pavement's surface requirements, or when a conventional HMA overlay is impractical. A thin overlay of about 12mm (0.5 in) may be applied to provide good skid-resistant pavements at a minimal cost. Comparing to a conventional HMA overlay, which would require more than 25 mm (1 in) think overlays, this process is cheaper [33], [34].

Figure 2-2 and Figure 2-3 [9] show the schematics of multiple and single-pass repaving process, respectively. The process consists of preheating, heating and scarifying, rotary milling, applying and mixing a rejuvenator, placing the recycled mix as a leveling course, and placing a new hot mix wearing course [35]. In the single-pass repaving process, one of two screeds is used to level the scarified existing pavement and the other screed is used to level off the new HMA layer [36].

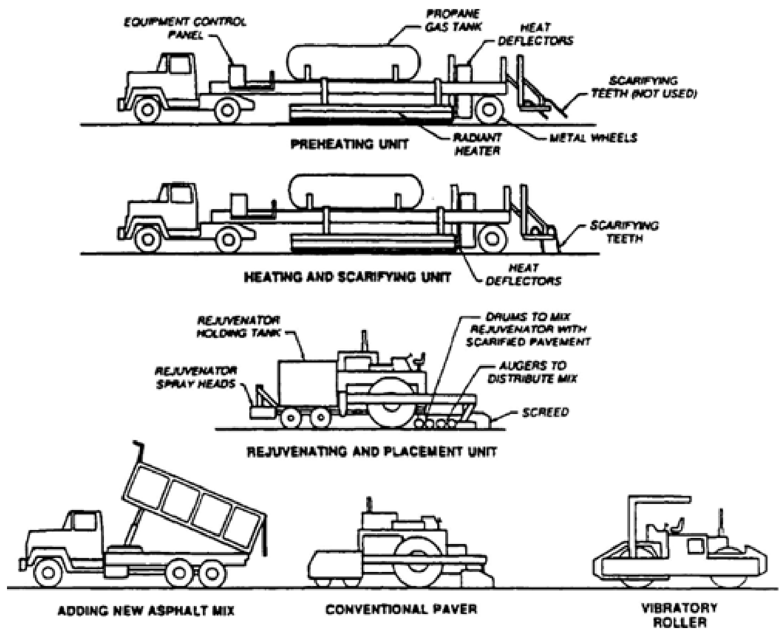


Figure 2-2 Multiple Pass repaving process [9]

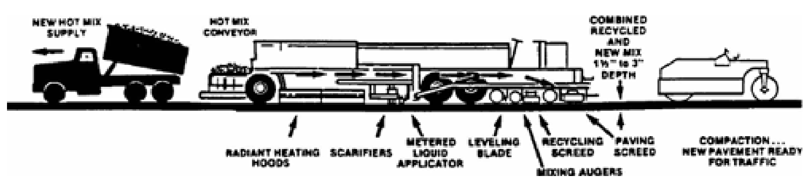


Figure 2-3 Single pass repaving process [9]

The pavement is heated up by the first and second heating units through the forced air or radiant heaters to a temperature close to 190 °C (374 °F) to a depth of 22mm to 30 mm (0.9 in to 1.2 in) [33]. A rejuvenator can be added to the scarified materials at a desired rate. The RAP materials mixed with a rejuvenator are collected and then moved transversely into a center windrow. The recycled mix is spread in front of the first screed with transverse augers and partially compacted as a leveling course. Lastly, new HMA from the hopper is placed on the recycled mix with a second screed at the temperature of 104 °C (219 °F) [33]. To ensure uniform bond between the new and the recycled mixture, compacting the new mix is performed immediately after the screed. Multiple lifts can be placed in a single-pass machine with two screeds, one trailing the other. Vibrators should be equipped in automatic screeds to perform initial compaction [37].

2.4 Remixing Method and Equipment

The remixing process consists of 1) heating of the pavement to a depth of 40 to 50 mm (1.5 in to 2 in), 2) scarification and collection of the softened material into a windrow, 3) mixing of the material with virgin aggregates and recycling agents in a pugmill, and 4) laying of the recycled mix as a homogenous mix [38]. This procedure can be used to replace 1 or 2 in of the pavement surface. This process is performed if repaving is not sufficient to restore the pavement to its performance requirements and needs greater strength and stability to the existing pavement [33].

Figure 2-4 and Figure 2-5 [9] display a schematic concept of the remixing method. Figure 2-4 illustrates the single pass methods and Figure 2-5 show the multiple passes process. First, by using preheating units, the existing pavement is heated and softened with a series of infrared heaters. The temperature of the asphalt pavement is raised to 85 °C to 104 °C (185 °F to 219 °F). The softened asphalt is collected in a windrow after being scarified or milled. The pavement is scarified to a depth of 25 to 40 mm (1 to 1.5 in) or possibly up to 50 mm (2 in). In Canada, a scarification depth of 75 mm (3 in) was performed since the softer grade of asphalt binder was used in the existing condition of pavement [38]. The recycled asphalt is moved from windrow to a pugmill to be mixed with a rejuvenator and measured amount of virgin aggregate or HMA. The mixed materials are then dumped from a truck at the front end of the remixing process and kept in a hopper. A rejuvenator can be added in the pugmill before the mixing process to provide a longer time for dispersion and mixing. The recycled mix is collected in a windrow by using a set of augers, then a vibrating/tamping screed is used to pace and compact the material. The typical temperature of exposed asphalt surface is approximately 66 °C (150 °F)[35], [39].

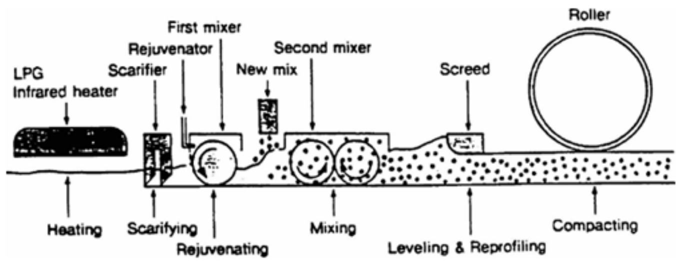


Figure 2-4 Schematic concept of the remixing method [9]

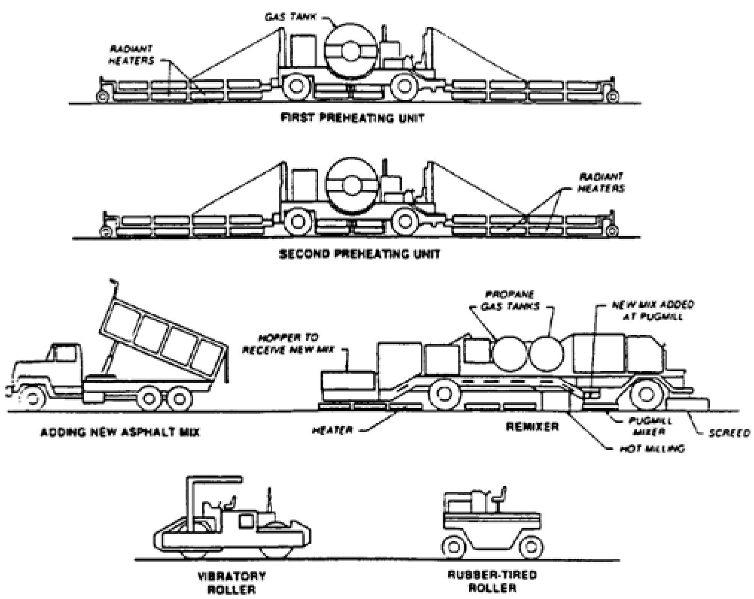


Figure 2-5 Multiple-pass remixing process showing equipment used [9]

When the pavement is heated by high intensity infrared heaters, smoke and other undesirable emissions can be generated. In addition, if a lower intensity infrared heat is applied, aggregate fracturing can occur during milling. As a solution to this problem, a preheating equipment that uses both low-level infrared heat and high velocity hot air can be used. This equipment can produce uniform heating of the pavement surface. A recycling train consists of six sets of equipment: two preheaters, one heater miller, one heater miller with paver, rubber tired roller and vibratory roller[36].

2.5 HIR Construction Project Consideration

Prior to HIR construction, proper project selection is essential to achieve the desired result and satisfactory performance. Limitations of HIR process should be acknowledged and each project should be thoroughly assessed with considerations.

To execute HIR construction, the pavement must be structurally stable for the expected traffic level and there shouldn't be a high level of distresses below about two inches of the pavement surface [12]. Also, it is important to ensure that the existing pavement can withstand the weight of heavy HIR equipment, especially when the HIR construction site is on the roads with low traffic level. The HIR process rejuvenates only upper surfaces (typically two inches) of asphalt. Therefore, this process is not for correcting any base or subgrade failures. Structural inadequacies cannot be addressed by HIR. Structural capacity of the pavement can only be corrected through a structural overlay over the recycled mixture [12].

Low quality of original mixture that may result in flushing, stripping and/or raveling can be improved by adding aggregates and asphalt during HIR. However, major improvements cannot be achieved by HIR. If these inadequacies are not recognized or identified, during the assessment of the project, the problem will soon appear again after rehabilitation [40]. In addition, a pavement with the stripping issues may not be improved by HIR although the original coating seems to be adequate. Even after HIR operation, preventing water damage is challenging. It is because the binder is typically softened, resulting more water-susceptible mixture. Adding anti-stripping liquid agent or lime additives are suggested in this situation [41]. Cracks that were reflected by joints in an underlying concrete pavement are not effectively delayed by HIR [42]. In addition, the existing pavement should not have severely oxidized asphalt cement as it is difficult to blend with rejuvenating materials [36].

Prior to HIR, it is recommended to remove any present patching materials which will affect the result of rehabilitation product. Particularly, cold patches should be eliminated because they negatively affect both production rates and mixture consistency. Also, crack sealing and seal coating materials can affect the final rehabilitation product. Crack sealing can produce excessive emissions when it is exposed to heats during the preheating process. Seal coat and chip seal materials limit the depth of cut and the equipment productivity as they insulate the underlying asphalt pavement. If excess amount of these materials are present, it is recommended to remove them prior to recycling [12].

To perform HIR, the existing pavement should be at least one inch thicker than the recycling depth. Also, the existing pavement should be wide enough to allow HIR equipment which has widths of 10 to 14.5 feet works on it. Areas with excessive amounts of obstructions such as manholes and/or gates are not suitable for HIR since such obstacles can reduce the productivity rate significantly. Due to the wide and heavy HIR equipment, the roadways with tight turns or steep grades are poor candidates as it is difficult to maneuver full-lane-width HIR [36].

Uneven surfaces causes uneven heating which might lead to several passes of the equipment to achieve the desired temperature [43], [44]. As compared to a conventional pavement, the final surface after HIR process is more variable. In general, the low speed ride results less smoother surface [45]. Obstacles in the road such as manholes and other stationary structures require handwork [46]. HIR pavement with imperfect handwork tends to ravel if the surface is not covered with a surface treatment or overlay. Potential compaction problems during HIR may cause segregation, open texture, low density and cracks again. Also, smooth deficiencies, inadequate depth of milling and insufficient mixing would be resulted [46].

Based on the past HIR projects, HIR equipment did not work well in windy or cool weather or when the pavement was damp [47], [48]. The ideal weather condition for HIR is hot and calm days. Having dry condition on the surface of pavement is necessary. Pavements with bleeding and flushing asphalt cements should be avoided [49]. Achieving desired level of warmth and dryness can increase the speed and quality of production. If the pavement is too cold during milling, excess aggregate fines and aggregate fracture may be generated [12], [50].

Selecting effective additives to rejuvenate the aged pavement and achieving right amount of heating without burning are the keys to successful HIR results. With developing advanced technology over the years, the problems were raised by HIR process have been minimized [51]–[53].

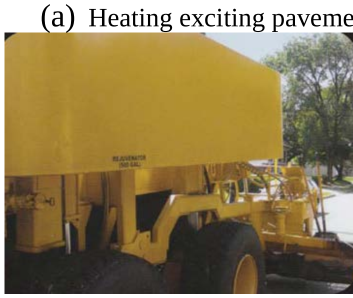
2.6 Comparing HIR Equipment by Different Manufactures/ Contractors

For the most effective HIR construction, it is imperative to use HIR equipment that is suitable to each project. HIR recycling methods vary by specialty contractors because each company uses special equipment and use their own equipment designs in Canada, Mexico, and the United States on a regional basis. Therefore, several companies that supply HIR equipment were researched and compared to each other for the cost and available depth of cut.

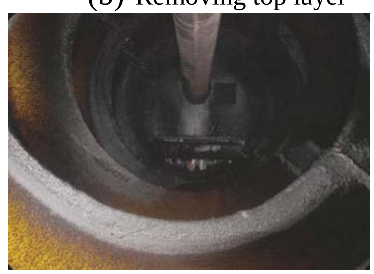
Angelo Benedetti, Inc. was a manufacturer of the patented Re-Heat 100% Asphalt Recycling equipment. Re-Heat process was an on-site, in-place pavement rehabilitation method that heated the old asphalt, then scooped it into a rotating chamber where the old asphalt was thoroughly mixed with a rejuvenating materials. Finally, placed it back on the road with a paving screed. Unlike other methods of in-place recycling, this method did not require a final wearing course with HMA paving [16]. Figure 2-6 shows the schematic of Re-Heat Process and the required sets of equipment.

(a) Heating exciting pavement

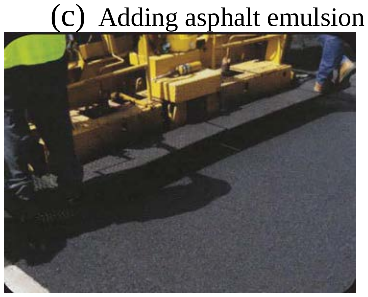
(b) Removing top layer



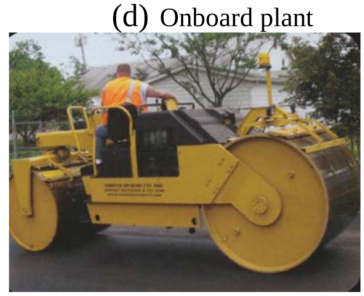
(c) Adding asphalt emulsion



(d) Onboard plant



(e) Relaying recycled material



(f) Compaction

Figure 2-6 How recycled hot emulsified asphalt treatment (Re-Heat) works [16]

Benedetti invented the Re-heat Asphalt process and demonstrated its performance through several state projects in the United States. Figure 2-7 shows the pavement conditions before and after Re-heat process.

Before re-heat

After re-heat

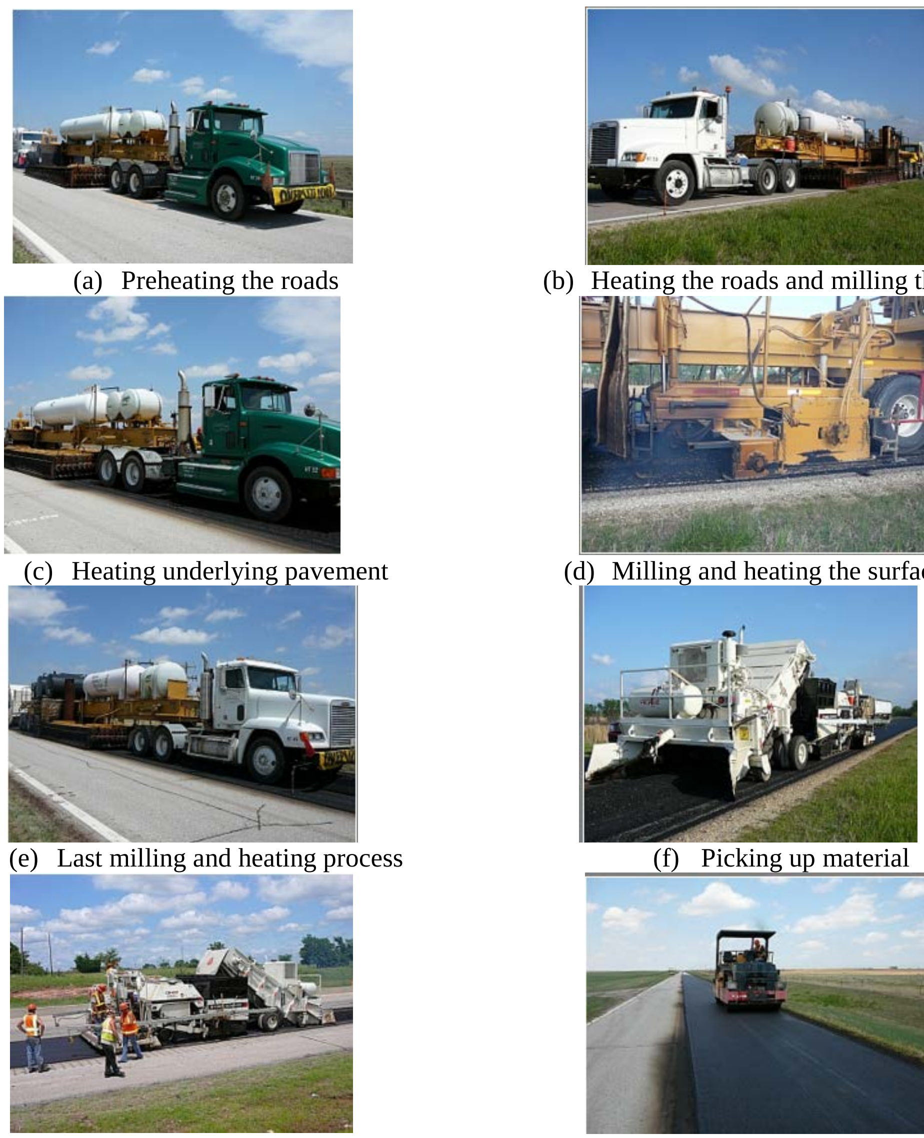
Figure 2-7 Condition before and After Re-Heat process [16]

The city of Greenville, Mississippi was the first city in Mississippi to implement Re-Heat hybrid type of hot in-place recycling work. 2-inch mill was chosen for this project and the cost saving was about 50% as compared to the traditional mill and surfacing method [54].

In 2011, the city of Milwaukee, Wisconsin, implemented Re-Heat hot in-place recycling process on over 67,000 square yards of pavement. Three roads with various backgrounds were selected including a road with heavy traffic for industrial uses, a minor arterial, and a four-lane split residential street. All roads were heavily oxidized and over 10 years old and contained curbs and gutters with varying levels of crack filler throughout the years. Despite the poor conditions, the construction was well performed by the Chicago area-based contractor called Gallagher Asphalt Corporation. Re-Heat process was performed by two pieces of equipment, a pre-heater, and a heater/recycler. The total length of the pair was approximately 80 feet long, far less than larger hot in-place equipment options. With smaller in size, the process was well suitable for municipal settings[54].

In August 2012, Re-Heat process was performed in Lancaster City. It was the first Re-Heat project in California. The additional benefit of Re-Heat process was the shorter turn-around time, compared to a conventional mill and fill project. Therefore, it was possible to perform construction quickly during none rush hours. The cost of Re-Heat was about $9.00 to $11.00 per square yard, less than those of the conventional resurfacing methods, which was about $12 to $15 per square yard. The cost savings allowed the city to stretch the amount of roadwork performed annually [16]. Also, Re-Heat demonstrated a 65% reduction in its carbon footprint, while increasing the serviceable life of most roadways by 7 to 10 years.

Another company invented innovative Hot in-Place equipment was called Dustrol, Inc. As a new method of Hot-In-Place Recycling, Dustrol invented MARS system (Mobile Asphalt Recycling System). The process of MARS system is described in Figure 2-8[55].



(a) Preheating the roads

(b) Heating the roads and milling the top 1”

(c) Heating underlying pavement

(d) Milling and heating the surface with

(e) Last milling and heating process

(f) Picking up material

(g) Paving

(h) Compaction

Figure 2-8 Mobile Asphalt Recycling System (MARS) System[55]

Dustrol'S MARS process addressed pavement distresses to a depth of 3 inches. Interstates, primaries and well-traveled secondary roads were common candidates for MARS system. Depending on the ambient and pavement temperature at the time of construction, the MARS process required up to seven to eight heating units. Then, the pavement was heated between 200 and 300 °F which could not degrade the existing asphalt. A double-drum vibratory roller compacted the mixture to the desired density. In one of the Kansas DOT projects, a surface smoothness and a typical in-place density was measured 0.7 inches per mile and 91 to 96 percent respectively. Dustrol's production rate was in between 20 and 25 feet per minute or 1200 to 1500 feet per hour. With the summer heat, Dustrol could process as much as 3.0 miles per 10 hour day [55]

Another contractor called Ecopave Systems began selling their equipment. Ecopaver 400, invented by Ecopave, was a two-stage, hot in-place asphalt recycling system which could heat, remove, rejuvenate and relay asphalt pavement to a depth of 2 inch at up to 4 lane kilometers (2.5 miles) per day. A rejuvenating agent was added to restore binder properties and new HMA could be added up to 30%. This patented system consisted of a preheater, two self-propelled heater/miller units, and a pugmill, then followed by a conventional paver and rollers. The Ecopaver 400 performed precisely to the exacting standards and incorporated a full emission control system that made the system almost smoke free. On average, it was 30-60% cheaper than conventional mill and fill methods. It also cut down significant amount of non-renewable aggregate and oil requirements up to 80%. The Ecopaver 400 was manufactured in British Columbia, Canada. Figure 2-9 shows the schematic of how the system worked [56]

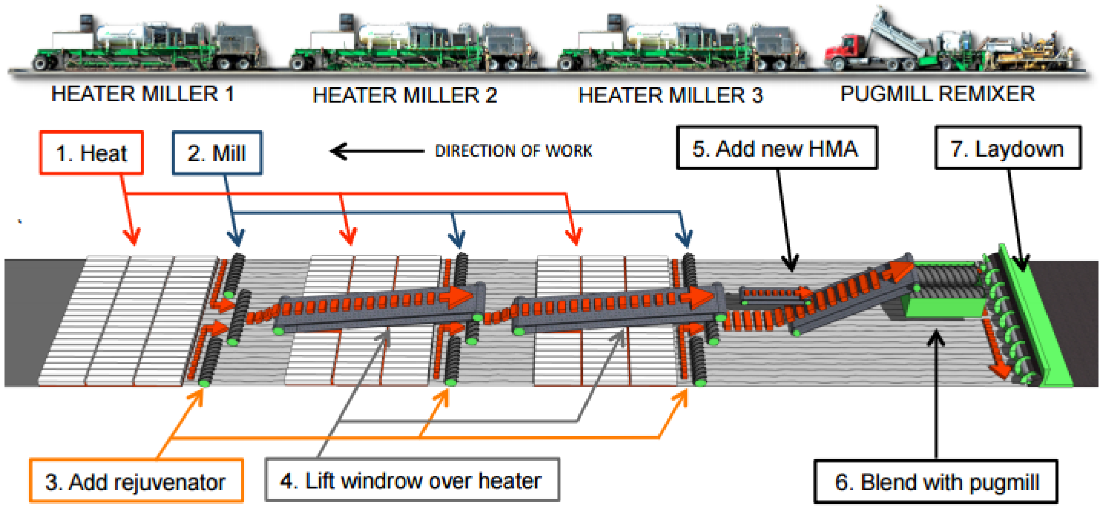


Figure 2-9 Ecopaver 400 Multi-stage recycler[56]

A new WIR equipment is being developed for recycling asphalt pavements with significantly reduced emission amounts. The purpose of this WIR technology is to recycle old asphalt pavements in the field while reducing the amount of carbon dioxide generated during the heating process. Figure 2-10 shows the indirect heating equipment for asphalt pavements before the recycling process. The equipment heats the existing asphalt pavement surface using an infrared heating burner with LPG fuel so than a flame does not hit the asphalt pavements directly. The equipment heats the top 2 in of an old asphalt pavement to about 121 ºC on the average. As a result, the warm surface milling can be performed without crushing the aggregates. As shown in Figure 2-11, an emission hood unit is installed on top of each heating plates where the collected emission is exhausted through the emission collector module (“off” condition is shown in Figure 2-11a. and “on” in Figure 2-11b). The emission controlled heating equipment reduces the emission by capturing the volatile organic compounds generated during the heating process, which would lead to the eco-friendly pavement recycling practices.

Figure 2-10 Emission controlled heating equipment for asphalt pavement recycling

(a) Emission control unit off (b) Emission control unit on

Figure 2-11 Heating equipment for asphalt pavement recycling

2.7 HIR Implementation

The potential saving of HIR is estimated to be 30% to 50% of the cost to mill and resurface. Also, emissions can be reduced to 70% and 100% of the existing pavement can be recycled [57]. Figure 2-12 contains the pavement conditions which is applicable to be corrected by HIR and the effectiveness of the work.

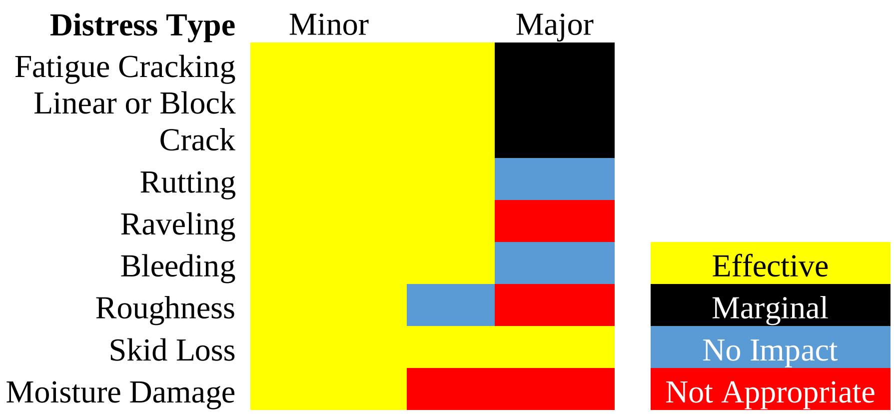
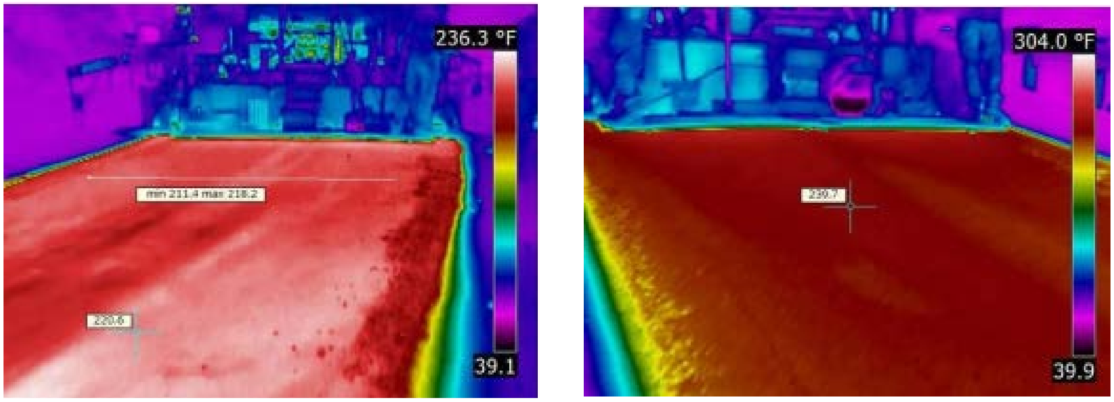


Figure 2-12 Allowable Pavement Conditions for HIR Use [29]

With numerous advantages of HIR technology, many Department of Transportations (DOT) have adopted HIR process as a rehabilitation of existing pavements. The productivity of HIR is highly controlled by the amount of applied heats to penetrate to the desired depth. Therefore, there are various requirements of ambient temperature, mix design, and properties of mixture for each state.

In order to begin HIR construction, Mississippi DOT (MSDOT) required an ambient temperature of 45 ºF [58]. Also, MSDOT noticed a minor uncoated aggregates when the temperature of the mixture in the paver hopper was below 265 ºF. In Florida, for a HIR project, the surface temperature of the existing pavement reached 530 ºF before remixing and paving. However, the mixture temperature right behind the paver screed was around 240 ºF [59]. The surface temperature changes according to the depth to be recycled, the number of heaters required, the speed of the paving train and the amount of oxidation of the existing pavement before recycling process.

Washington State Department of Transportation documented the construction of a section of SR 542 rehabilitated using the HIR process. A thermal image from a HIR project performed on SR 542 shows consistent temperatures across the mat. As shown in Figure 2-13, the temperature behind the paver screed is in the range from 195 ºF to 240 ºF [59].



(a) (b)

Figure 2-13 a) Thermal Image showing uniform temperature across the mat, b) Maximum mat temperature was approximately 240 ºF [59]

The mix design used in this project had an air void specification of 2.5% to 5.5% when compacted to 75 gyrations in a Superpave gyratory compactor. By adding the recycling agent, the mix became more compactable, allowing for adequate compaction at the lower temperature than with conventional HMA [59].

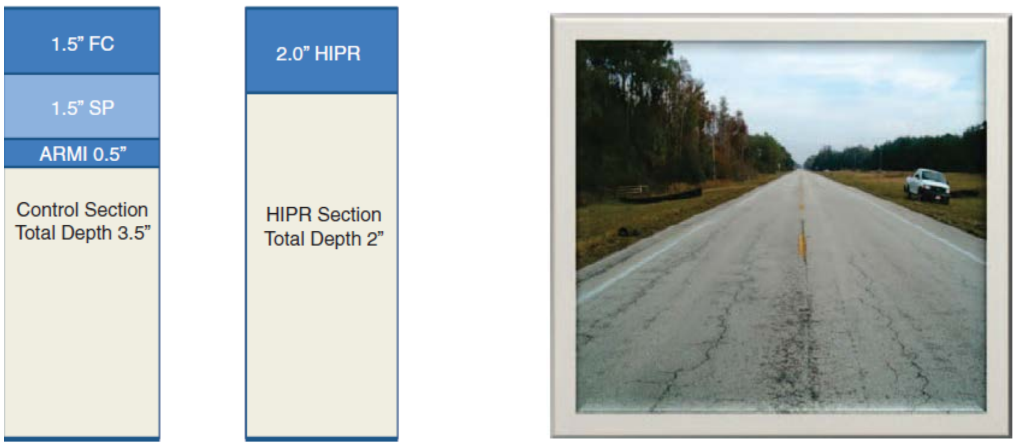
The total project cost including design, administration, safety, HMA paving, pavement marking and unexpected work required along the project was $5,670,000. However, the cost of HIR items only was $1,860,000, calculated to have a cost of $58,500 per lane mile. The total cost per lane mile for a conventional HMA rehabilitation on Washington State Highway was approximately, $200,000. In comparison, the initial cost of HIR was about 15 percent less than that of traditional HMA mill and fill. However, the long-term assessment of service maintenance cost was not addressed [59].

Florida Department of Transportation built a test section in 2002 and a control section in 2003. After 8 years, the aging of the sections were monitored to compare the performance and cost of HIR with conventional milling and resurfacing process. The results concluded that the test sections performed well with the cost less than half the cost of the conventional section. Projections predicted that HIR can reduce costs by 40% more than the conventional method of pavement rehabilitation.

As a test section, SR-471 was rehabilitated by H.I.P paving LLC of Safety Harbor, Florida. The test section location was in Florida DOT District 5, Sumter County, on SR-471 south of Tarrytown. The length of two-lane highway test section was 5.115 mile (10.23 lane miles and 96,026 yd^2). The north- and southbound lanes were 12 ft. long and shoulders were 4 ft. wide and they were recycled in place to a depth of 2 in. The average daily traffic was 2,800 vehicles. The last resurfacing was performed in 1991. The crack rating of original pavement was 4.5 out of a scale of 10.0. The low crack rating indicated that the pavement was deficient [57].

As an innovative construction process, the Florida DOT contracted the project with HIP for a lump sum prices of $615,000, or an average unit cost of $60,117 per lane mile ($6.40/yd^2). The cost of 3-year warranty was $80,000 and it was included in the $615,000 lump sum price for this test project [57]. If the maintenance bond cost was deducted from the lump sum price, the average unit cost was lowered as $52,287 per lane mile ($5.57/yd^2). The total construction duration was 22 calendar days (16 working days) from November 18, 2002, to December 9, 2002.

In March 2003, conventional milling and resurfacing techniques were performed on a control road section directly north of the HIR test section. The control section consisted of two 12 ft. wide lanes and two 4 ft. wide shoulders which was milled to a depth of 3 in. An asphalt rubber membrane interlayer (ARMI) was milled over the existing pavement surface to a depth of 0.5 in. Then, Type SP (Structural Superpave layer) and Type FC (Superpave Friction course layer) were covered to a depth of 1.5 in each. The total length of the control section was 9.81 mi (18.362 lane miles and 172,358 yd^2). D.A.B. Constructor, Inc. performed the work at a cost of $1,899,162, which is calculated to have an average unit cost of $103,429 per lane mile or $11.02/yd^2 [57]. Figure 2-14 displays the schematic of the test section with HIR and the control section with conventional milling and resurfacing.



(a) (b)

Figure 2-14 HIR and control sections a) schematics of both designs b) Highway Section before construction[57]

HIR process on SR-471 was considered as a mixed in-place process. The existing pavement was first heated and then milled and removed, finally it mixed with new paving materials where necessary. The rejuvenated material is applied back to the roadway [57].

In December 2010, the performance of test and control road sections were tested 8 years after the construction. Figure 2-15a and b demonstrate the average condition of conventional milling and HIR sections. Figure 2-15c and d show the distressed sections in HIR and control sections. Distress types in HIR are mainly rutting, cracking and depressions in the wheel path; approximately five 300-ft distressed areas were found and that is equivalent to 1,500 out of 53,000 lane feet in the project, about 3%. As shown in Figure 2-15c, in the control section, the distress consisted of longitudinal and fatigue cracking [57].

The HIR section satisfied the quality of control testing during construction. The levels of distress in cracking, ride, and frictional properties of the section were similar to those of conventional construction. Localized rutting of 0.2 in. occurred around areas of rutting before construction. The areas of rutting was limited and rutting performance was good enough to reach 9 out of 10 from 2005 to 2010. Localized distress on the HIR section was less than 5% of the paved area and was reworked. A life-cycle cost analysis, including 5% rework in year 8, proved that HIR process can save 40% over the cost of conventional milling and resurfacing process [57]

(a) (b) (c) (d)

Figure 2-15 a)Average condition of control section (8 years old) b) Average condition of HIR section (8 years old) c) Distressed areas in control section (8 years old) d) Distressed areas in HIR section (8 years old)[57]

2.8 Utilization of Various Recycling Methods by Contractors

To access the status of in-place recycling across the United States, state agency engineers, and recycling contractors participated in an online survey. Through NCHRP project, 45 states and District of Columbia documented their use of hot in-place recycling (HIR), cold in-place recycling (CIR), and full-depth reclamation (FDR). 33 contractors among fifty contractors identified as members of the Asphalt Recycling and Reclaiming Association (ARRA) responded to the survey as well [37].

The survey results were based on the “choose all that apply” questions. Different types of recycling methods are applicable to different distress severity and types [37]. HIR is the rehabilitation method to treat minor surface distresses that has no structural damages on the existing pavement. CIR is used for more severe distresses that may propagate further into the pavement. FDR is an in-place rehabilitation that treats the full range of pavement distresses and subgrade support. This method is usually used for lane widening, profile improving, and increased structural capability.

Agency and contractor responses were compared and sorted by experience with a specific in-place recycling process. The result is summarized in the Table 2-1.

Table 2-1 Experience of Agencies and Contractors [37]

In-Place Recycling Method	Number of Response (Percent of total response)	
	States	Contractors
HIR		
Resurfacing	13 (28)	4 (12)
Repaving	11(24)	3 (9)
Remixing	16 (35)	12 (36)
CIR	34 (74)	24 (73)
FDR	33 (72)	28 (85)

Not all respondents had experience with all three types of rehabilitation. It is assumed that low numbers of experienced contractors leads to low percentage of use in resurfacing and repaving HIR. Both agencies and contractors had more experience with CIR and FDR than those with HIR.

Figure 2-16 illustrates the distribution of usage of three types of recycling methods surveyed. While HIR and FDR are performed across the United States, CIR, on the other hand, was not reported in the southern and southeastern states. [60].

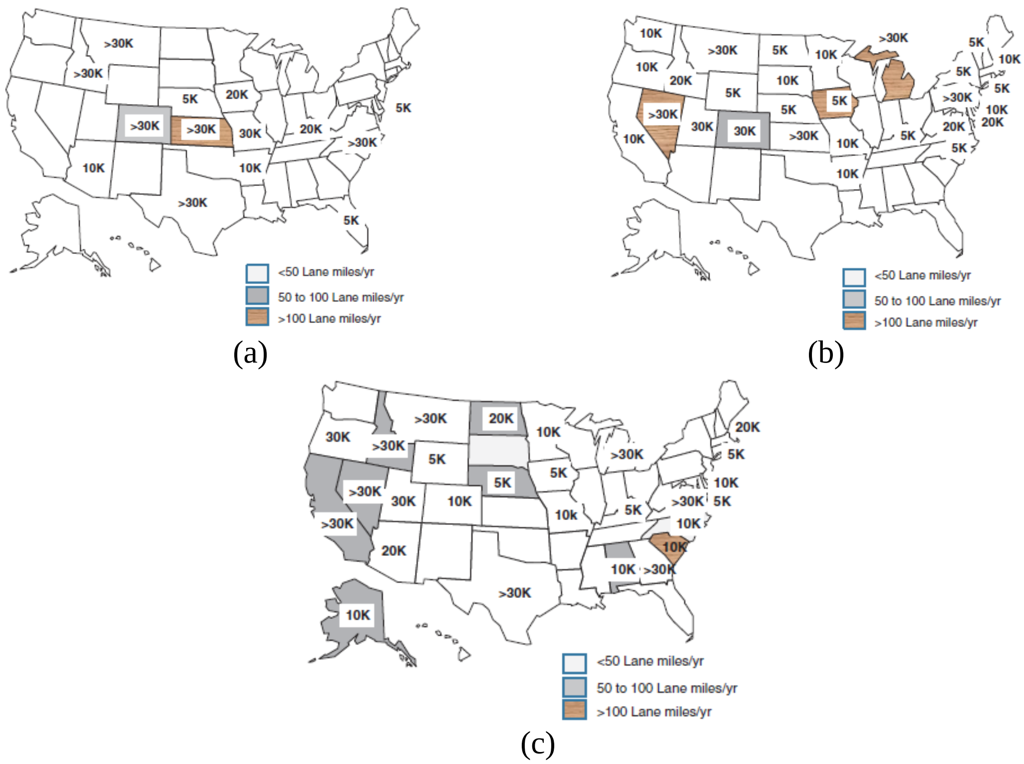


Figure 2-16 State use of processes: a. HIR, b. CIR, and c. FDR [61] (Numbers on map indicate AADR for recycled roadways)

2.9 Proposed Warm In-Place Recycling Technology

Hot In-Place Recycling (HIR) has been widely used as pavement maintenance methods in the world. With the heavy emphasis on sustainability in recent years, researchers have been searching for a new technology to improve the existing HIR process. Although many efforts such as using new blades, heaters and additives have been made to reduce emission from HIR process, the emission issues remains unresolved.

In the 'Flexible Pavement Preservation Guidelines' by Caltrans Division of Maintenance in 2008, it was suggested to apply warm mix additives and methodology into HIR process. This would further extend the HIR application as the additives can help HMA to be readily compacted at lower temperature than traditional compacting temperatures. Additives were expected to play a vital role in improving one of the limitations with HIR, reaching adequate compaction temperature in cool and windy weather [62].

In South Korea, there has been an invention of a modified, RAP recycled, and temperature-controlled asphalt mixture with 100% recycled asphalt mix composition. The product can be used as a wearing course, a surface layer, an intermediate layer and a base layer of asphalt pavements. A recycling modifier is mainly for improving the physical properties and a temperature-controlled agent is for controlling production and construction temperatures of the RAP mixture. The idea of temperature-controlled agent can easily be applied to HIR process where temperature-controlling agent can limit the amount of greenhouse gases and energy saving in HIR production.

Temperature-controlled agent or warm mixture additive consists of more than one agent among 0.5-100 % of a water blowing agent, 0.5-100 % of a wax and 0.5-100% a chemical blowing agent by weight. When they are added to the recycling process, a production and a construction temperature of the RAP can be lowered. When two or more agents are used, the combination is arbitrary but the total amount of temperature-controlling agent is 0.1-20 parts by weight on the basis of 100 parts by weight of RAP. They are mixed uniformly for 0.5 – 3 minutes at a temperature between 41 ºF and 356 ºF as follow: Cold mixing: 41 – 86 ºF (5 – 30 ºC) , Semi-warm mixing 86 – 212 ºF (30-100 ºC), Warm Mixing 212 – 302 ºF (100 – 150 ºC), and hot mixing 302 – 356 ºF (150 – 180 ºC) [62].

A water blowing agent in a temperature-controlling agent can act as a foaming agent that can melt old asphalt binder in RAP to lower its viscosity because water evaporates at 212 ºF. It can make the semi-warm mixing and the warm mixing be possible. A wax as a temperature -controlling agent drops the mix viscosity significantly once the mixing temperature reaches above the melting point of added wax. A chemical blowing agent as a temperature-controlling agent typically has its blowing temperature between 122 and 284 ºF. With the blowing temperature of added chemical agent(s) showing in the range of 122-212 ºF, semi-warming mixing can be used in warm mixing. Beyond the blowing temperature of chemical or water agents, micro-pores are produced in the mixture of old asphalt binders in RAP. These micro-pores can lower the viscosity of mixes, leading to the lower production and construction temperatures [62].

With lower temperature, HIR can be performed at reduced amount of harmful gas, benefiting not only workers but neighbors. Also, energy can be saved due to the decreased production and pavement construction. In addition, using HIR process can be completed under relatively cold climates.

CHAPTER 3: RHEOLOGICAL PROPERTIES EVALUATION OF MODIFIED ASPHALT BINDERS BY WARM-REJUVENATOR

As mentioned in previous chapter, adding two new WIR additives was developed to 1) improve the quality of WIR mixtures, 2) decrease the mixing and compaction temperatures of mixtures in WIR process, and 3) increase the percentage of recycled material in WIR process.

3.1 Experimental Plan

The major intent of this chapter is to characterize the rheological properties of asphalt binders modified by two different WIR additives to determine the optimum dosage of each of them. Superpave asphalt binder tests were conducted on the control asphalt binder, modified asphalt binders with 5% and 10% of additive 1 and with 7.5% and 15% of additive 2. The optimum dosage of an additive is the minimum percentage of additive that recover the rheological properties of aged asphalt binder to those of the virgin asphalt binder. After determination of the optimum dosage, a second round of rheological tests were designed and executed to verify the results.

3.1.1 Characterization of Asphalt Binders Modified by WIR Additives

The following asphalt binder tests have been performed: 1) Rotational Viscosity (RV) test to determine the mixing and compaction temperatures of asphalt mixtures; 2) Rolling Thin Film Oven (RTFO) test to simulate the short-term aging of asphalt binders during the construction period; 3) Pressure Aging Vessel (PAV) test to simulate the long-term aging of asphalt binders during the serving life; 4) Dynamic Shear Rheometer (DSR) test for virgin and RTFO aged asphalt binders to evaluate the high temperature performance of asphalt binders and mixtures; 5) DSR test for PAV aged asphalt binders to evaluate the fatigue (intermediate temperature) performance of asphalt binders and mixtures during the service life; 6) Bending Beam Rheometer (BBR) test to evaluate the low temperature performance of asphalt binders; MSCR test to evaluate the recoverability and rutting properties of modified asphalt binders.; 7) Fourier-Transform Infrared (FTIR) Spectroscopy test to measure the aging properties of asphalt binder quantitatively.

The asphalt binder used in this study includes petroleum-base asphalt binder PG 64-22. The compositions of additive 1 and 2 used in this study are shown in Table 3-1. To develop new WIR additives, soybean oil was blended with various amounts of three amide type anti-stripping additives: 1) ethylenediamine (ED), 2) 2-hydroxyethyl ethylenediamine (HEED) and 3) Tetraethylenepentamine (TEPA). The formation of amide bonds was successfully achieved in the laboratory, which has been confirmed using the FT-IR and 1H-NMR equipment. The adhesive properties of antistripping agents were measured using Bituminous Bond Strength (BBS) test. Based on the BBS test results, PG 64-22 binder with antistripping additive synthesized from TEPA exhibited the higher moisture resistance value than those synthesized from ED and HEED. To enhance the stiffness of the WIR additive, the SBS polymer was also added to TEPA/Soybean-based WIR additive. A recycling modifier is mainly for improving the physical properties, and a temperature-controlling agent is for controlling production and construction temperatures of the mixture. Therefore, the most significant advantages of using these products are that they can rejuvenate the aged asphalt binder and decrease the mixing and compaction temperatures of mixture, simultaneously.

Table 3-1 Compositions of additives used in this study

Additive	Composition
Additive 1	Tepta + Soybean oil
Additive 2	Tepta+ SBS + Soybean oil

3.1.2 Rotational Viscometer (RV) Test

The RV test is to determine the viscosity at the high temperature and furthermore the workability of asphalt binders. The RV test for virgin asphalt binder can determine the mixing and compaction temperatures of asphalt mixtures during the construction as well. The RV test follows the standard AASHTO T 316. The test temperatures are 85°C to 135°C in this study.

3.1.3 Dynamic Shear Rheometer (DSR) Test

DSR test is to determine the visco-elastic property of asphalt binders in a high temperature and frequency range. The standard procedure of DSR test follows AASHTO T 315. In this study, the DSR test was conducted for virgin, RTFO- and PAV- aged asphalt binders. For virgin and RTFO aged asphalt binders, the test temperatures were 64 70, 76 and 82°C, while the frequency was 1.59 Hz. For the PAV aged asphalt binders, the test temperatures were 13, 19, 25, 31 and 37°C, while the frequency was 1.59 Hz.

3.1.4 Rolling Thin Film Oven (RTFO) Test

The RTFO test is to simulate the short term aging of asphalt binders during the construction. The standard RTFO test follows the AASHTO T 240. The asphalt binders are conditioned in the oven at 163°C for 85minutes.

3.1.5 Pressure Aging Vessel (PAV) Test

The PAV test is to simulate the long term aging of asphalt binders during the service life. The standard PAV test follows AASHTO R 28. In the test, the RTFO aged asphalt binders were exposed in the aging condition with 2.1MPa and 100°C for 20 hours.

3.1.6 Bending Beam Rheometer (BBR) Test

The BBR test is to investigate the low temperature performance (thermal cracking) of asphalt binders. The test procedure follows AASHTO T 313. The materials used in the BBR test are the PAV aged asphalt binders. The Superpave binder test recommends -12°C as the test temperature for PG 64-22.

3.1.7 Multiple Stress Creep Recovery (MSCR) Test

The limitation of old performance grad system, especially regarding modified asphalt binder, created the multiple stress creep recovery (MSCR) test method. The new standard specifications have been adopted by AASHTO. The test procedure follows AASHTO T 350-14 and the material used in this test is RTFO aged asphalt binder. This new specification uses multiple stress creep recovery test to characterize asphalt behavior at high temperature. The ultimate purpose of this test is quantification of the asphalt binder contribution to rutting resistance more accurately than current standards, especially with modified binder. The new standard specifications were according to the environmental factors such as temperature as well as design traffic such as standard, heavy very heavy or extreme. The main intent is to replace current asphalt binder grading methods with more technical methods which reflect traffic and environment simultaneously.

This test method covers the determination of percent recovery and nonrecoverable creep compliance of asphalt binders by means of the Multiple Stress Creep Recovery

(MSCR) test. The MSCR test is conducted using the Dynamic Shear Rheometer (DSR) at a specified temperature.

The MSCR test is the improved method from Repeated Creep and Recovery (RCR) test. The MSCR test procedure uses 1 s creep loading followed by 9 s recovery at no load for various stress levels; originally stresses of 25, 50, 100, 200, 400, 800, 1600, 3200, 6400, 12800 and 25600 Pa are used, applying ten cycles at each stress level [62]. The test is started at the low stress level and increased to the next stress level at the end of every 10 cycles, with no rest periods between cycles. The average non recoverable strain for the 10 creep and recovery cycles is then divided by the applied stress for those cycles. The obtained value is the nonrecoverable creep compliance J_{nr} which is proposed to replace the current $G^*/\sin(\delta)$.

Based upon correlation between binder permanent deformation and mixture rutting, D'Angelo [62], [63] selected two stress levels of 100 Pa and 3200 Pa at ten cycles for each stress level instead of the original 11 stress levels. A shear stress of 100 Pa was proposed to study the behavior of asphalt binder in the linear region whereas 3200 Pa was in the nonlinear viscoelastic region for most modified and unmodified binders.

Since wide range of stress level is used in MSCR, this test is able to reflect both linear and nonlinear viscoelastic properties of asphalt binders. The main application of the MSCR test is for modified asphalt binder. D'Angelo and Dongre [64]. showed the MSCR result (J_{nr} and percentage recovery) can detect the dispersion of styrene-butadiene-styrene (SBS) in modified asphalt binder.

Wasage et al. [65] studied the rheological properties of unmodified and modified asphalt binders using the MSCR test. They reported that for unmodified asphalt binders, the accumulated compliance is a function of time for stress levels up to 10 kPa and for modified asphalt binder, the accumulated compliance is a function of applied stress and time, except for very low stress levels [64]. In addition, they studied the correlation between J_{nr} with laboratory wheel tracking test results [64]. The best result was obtained when J_{nr} and rut depth were correlated at the high stress levels of the MSCR test.

3.1.8 Fourier Transform Infrared (FTIR) Spectroscopy

Fourier-Transform Infrared (FTIR) Spectroscopy is commonly used to identify certain molecules or functional groups and the concentration of those molecules within a sample [64]. The FTIR measures amounts of Infrared light that was absorbed by asphalt at each wavelength over a range of, e.g. 4,000 to 400 cm-1. The asphalt would absorb different wavelengths and create a unique interferogram of reflected lights, which should be then processed using Fourier transform algorithm to derive the transmittance level for each wavelength [66]. FTIR spectrometers have been found cheaper than conventional spectrometers since producing an interferometer is easier than the fabrication of a monochromatic. In addition, measurement of a single spectrum is much faster for the FTIR technique as the information at all frequencies can be collected simultaneously [64].

To analyze the aging process of asphalt quantitatively, the peak area of oxygenated functional groups (S=O and C=O), which represents the degree of asphalt aging, can be examined (1032cm-1 for sulfoxide and 1699cm-1 for carbonyl) by measuring a coherence of electromagnetic radiation. It can be postulated that rejuvenators would decrease these sulfoxide and carbonyl peaks [67].

3.1.9 Test Plan and Test Numbers to Identify Optimum Dosage of Additives

Table 3-2 summarizes the sample ID and content of each group. There were six main groups of asphalt binders in this study. Four groups with different types and dosages of additives and two control groups. The major difference of two control groups was the aging condition of them. The C-V was completely virgin binder and C-100 experienced one RTFO and PAV tests before the normal rheological tests. In this table, 100 means 100% of sample (all sample material) experienced one cycle of Superpave aging by running through RTFO and PAV tests and then mixed with additives.

Table 3-2 Sample ID and contents of each sample

Sample ID	Content of Sample
C-V	Virgin binder (PG 64-22)
C-100	100% Aged* (RTFO + PAV)
100-A1-5	100% Aged (RTFO + PAV) + 5% Additive One
100-A1-10	100% Aged (RTFO + PAV) + 10% Additive One
100-A2-7.5	100% Aged (RTFO + PAV) + 7.5% Additive Two
100-A2-15	100% Aged (RTFO + PAV) + 15% Additive Two

Aged*: Virgin binder(PG64-22) aged by running RTFO and PAV tests on it once

Table 3-3 gives the test plan and number of each Superpave test that was needed for this study. As mentioned in the beginning of this chapter, the major goal of conducting these tests was to identify the optimum dosage of each additive. The RV tests identified the mixing and compaction temperatures of mixtures. The DSR tests for virgin, RTFO, and PAV aged asphalt binder were conducted to determine the high temperature rheological properties of samples. These DSR test results were used to identify the maximum dosage of each additive. The BBR test was executed to determine the low temperature of asphalt binders in PG grade system and the results of this test were used to determine the minimum (optimum) dosage of each additive. The MSCR test was used to identify the effects of each additive on the rutting and recoverability of modified asphalt binders.

Table 3-3 Rheological test plan and number of tests

	Tests						
Sample ID	**RV**	**DSR (Virgin)**	**DSR (RTFO)**	**DSR (PAV)**	**MSCR**	**BBR(T1)**	**BBR(T2)**
C-V	XXX	XXX	XXX	XXX	XXX	XXX	**XXX**
C-100	XXX	XXX	XXX	XXX	XXX	XXX	**XXX**
100-A1-5	XXX	XXX	XXX	XXX	XXX	XXX	**XXX**
100-A1-10	XXX	XXX	XXX	XXX	XXX	XXX	**XXX**
100-A2-7.5	XXX	XXX	XXX	XXX	XXX	XXX	**XXX**
100-A2-15	XXX	XXX	XXX	XXX	XXX	XXX	**XXX**
Total	18	18	18	18	18	**36**	

*note: X means one replicate

3.2 Sample Preparation to Identify Optimum Dosage of Additives

One of the main intents of using these two additives was to rejuvenate the aged asphalt binders. It is also desired to investigate how much they were able to rejuvenate asphalt binders. Therefore, to simulate the aging condition of real pavement, first the virgin asphalt binder was aged by running one RTFO and PAV test. Next, it was mixed with each of the additives. The modified aged sample was treated as a base sample (Virgin Sample) and all Suprpave tests were performed. For instance, the DSR for RTFO aged asphalt binders means the modified sample that already experienced one cycle of RTFO and PAV tests, was aged on RTFO again and then DSR test were performed run on it.

Before mixing the additives and aged asphalt binders, the virgin asphalt binder was heated to about 120°C. Since the additives were stored in plastic bottles and it was not proper to heat them in the oven, a water bath was used to take out the additives from the bottles. Thus, the temperature to heat the additives was about 90 to 100°C. After heating, the additives and aged asphalt binders were mixed with the designed fractions. The mixing temperature was around 120°C. One of the significant advantages of using these two additives was a simple mixing condition due to the fact that a high speed mechanical shearing equipment was not needed. The additives and aged asphalt binders were mixed with a spoon.

3.3 Results and Discussions to Identify the Optimum Dosage of Additives

3.3.1 Rotational Viscometer (RV) Tests

The viscosity was measured using a rotation viscometer at 85, 95, 105, 115, 125 and 135°C in this study. This wide range of temperatures was considered to better understand the viscosity properties of samples. The spindle of 25mm in diameter and the rotation speed of 0.3 Hz were selected. Figure 3-1 illustrates the viscosity of PG 64-22 and modified samples with 5, 10 and 15 percentages of additives 1 and 2 at different temperatures.

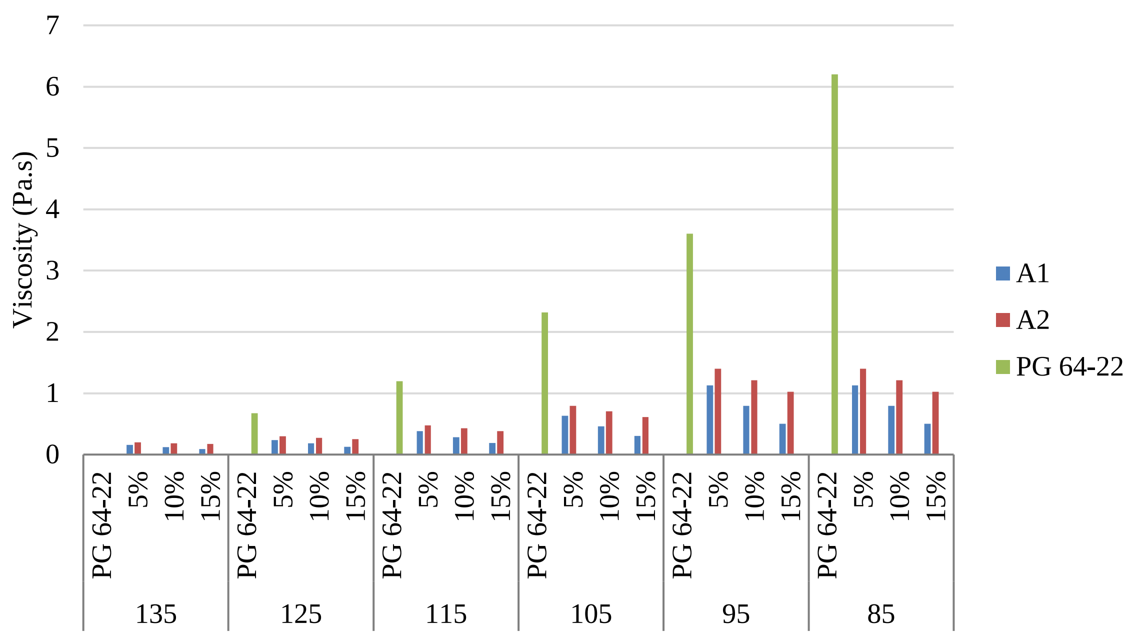


Figure 3-1 Viscosity of samples with the additives of A1 and A2

As can be seen in Figure 3-1, since the virgin asphalt binder already experienced one RTFO and PAV tests cycle, the viscosity of aged PG 64-22 was higher than virgin asphalt binders. It should be mentioned that both additives were able to significantly decrease the viscosity of aged asphalt binders at all temperatures. This fact resulted in lowering mixing and compaction temperatures for both additives. However, in general, additive 1 could decrease the viscosity more than additive 2.

Figure 3-2 and Figure 3-3 summarize the viscosity of modified asphalt binder with different percentages of additives at different temperatures.

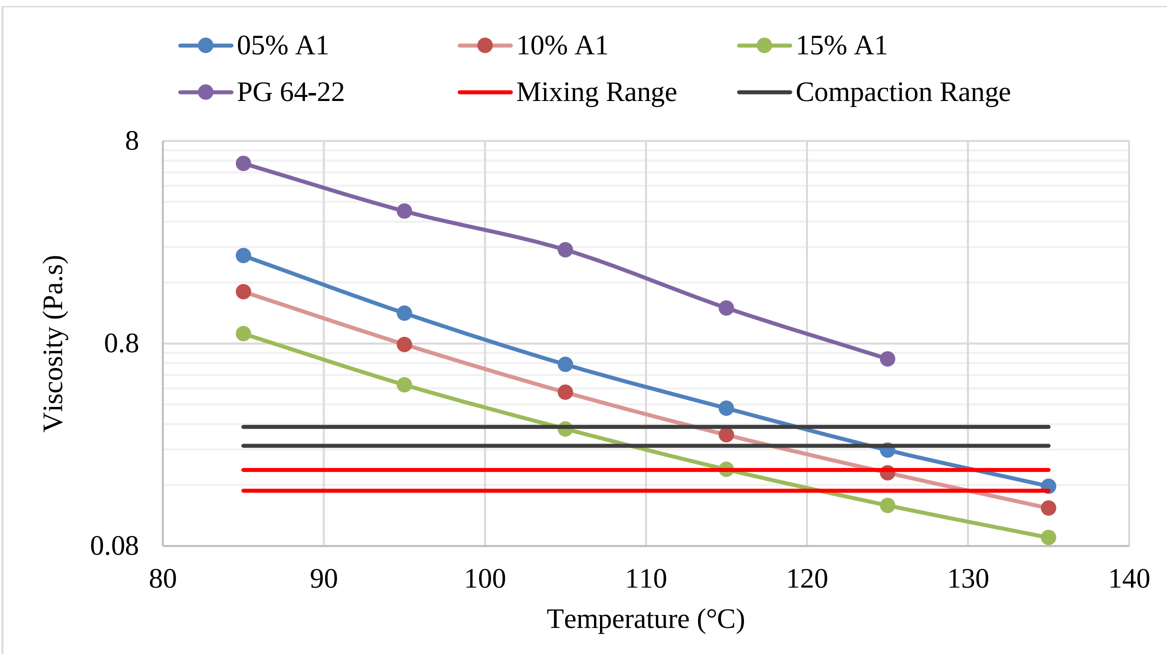


Figure 3-2 Viscosity of modified sample with additive A1

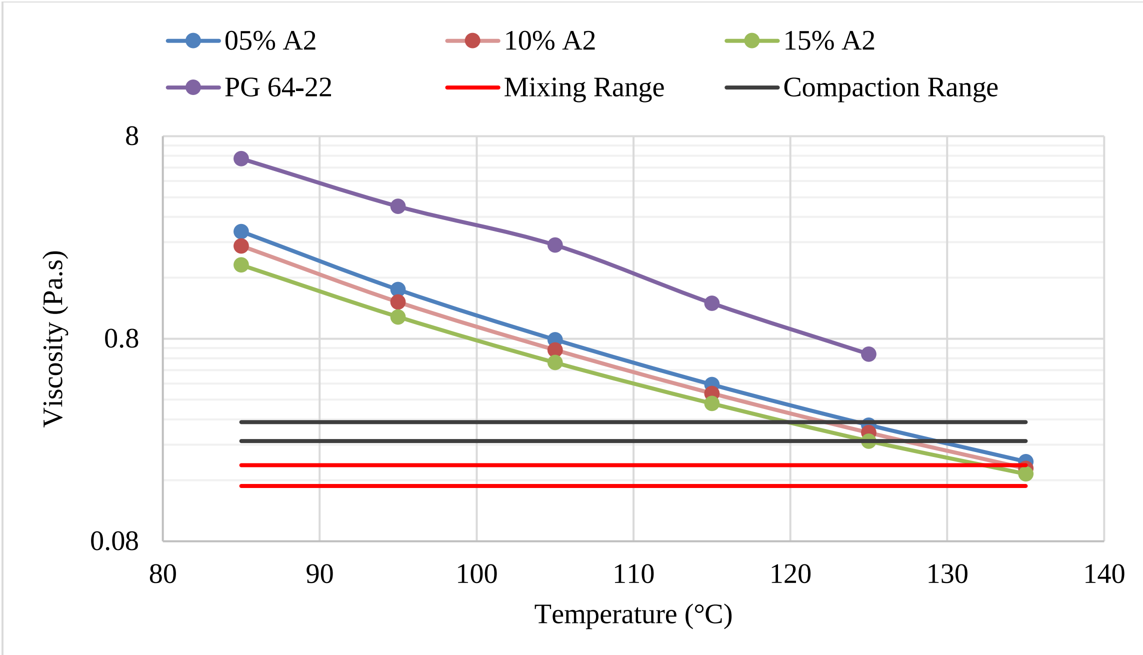


Figure 3-3 Viscosity of modified sample with additive A2

The optimum dosage of additives for additive 1 and 2 were previously identified as 3.99% and 7.77% respectively in the section 3.3.3 of this chapter. Therefore, by interpolation and extrapolation, the mixing and compaction temperatures of modified asphalt mixtures with additive 1 and 2 were identified as shown in Table 3-4.

Table 3-4 Mixing and compaction temperature for the optimum percentage of additives

		Mixing T °C		Compaction T °C	
	% of additive	Low	High	Low	High
Additive 1	3.99%	132	136	120	125
	5%	131	135	119	124
	10%	124	130	113	118
Additive 2	10%	134	139	123	127
	7.70%	133	138	121	126
	15%	132	138	119	125

3.3.2 Dynamic Shear Rheometer (DSR) Test

DSR test was conducted for base, RTFO and PAV aged modified asphalt binders in this study. The major intent of DSR test is measuring the rheological properties of asphalt binder at high and intermediate temperatures. The Superpave specification is its reliance on testing the asphalt binder in condition that simulate critical stages during the asphalt binder life. $G^*/\sin(\delta)$ is the Superpave specification to prevent rutting. $G^*/\sin(\delta)$ should be greater than 1.0 kPa before aging to control rutting during transport, storage, and handling and greater than 2.2 kPa after RTFO to prevent rutting during mix production and construction of asphalt mixtures. Table 3-5 and Table 3-6 present the results of DSR test for virgin and RTFO samples, respectively at different temperatures.

Table 3-5 DSR test result for base (unaged) asphalt binder

Sample ID	**Parameter**	**T (°C)**			
		64	**70**	**76**	**82**
C-V	G* (kPa)	1.72			
	δ (°)	86.7			
	G*/Sin(δ)(kPa)	1.72			
C-100	G* (kPa)	17.5	8.2	3.85	1.74
	δ (°)	72.8	78	80.9	83.6
	G*/Sin(δ)(kPa)	18.3	8.38	3.9	1.75
100-A1-5	G* (kPa)	4.55	2.07	1.05	0.51
	δ (°)	81.1	83.5	85.6	87.1
	G*/Sin(δ)(kPa)	4.6	2.09	1.05	0.51
100-A1-10	G* (kPa)	1.38	0.678		
	δ (°)	85	86.4		
	G*/Sin(δ)(kPa)	1.39	0.68		
100-A2-7.5	G* (kPa)		4.26	2.02	1.03
	δ (°)		79.6	82.6	84.8
	G*/Sin(δ)(kPa)		4.33	2.04	1.03
100-A2-15	G* (kPa)		2.64	1.33	
	δ (°)		80.7	82.9	
	G*/Sin(δ)(kPa)		2.67	1.34	

Table 3-6 DSR test result for RTFO aged asphalt binder

Sample ID	Parameter	T (°C)				
		58	64	70	76	82
C-V	G* (kPa)		4.22			
	δ (°)		83.3			
	G*/Sin(Δ)(kPa)		4.25			
C-100	G* (kPa)				6.79	3.16
	δ (°)				76.7	79.9
	G*/Sin(δ)(kPa)				6.98	3.21
100-A1-5	G* (kPa)		7.78	3.51	1.72	
	δ (°)		78.2	81.1	83.5	
	G*/Sin(δ)(kPa)		7.95	3.56	1.74	
100-A1-10	G* (kPa)	4.6	2.19			
	δ (°)	80.4	82.9			
	G*/Sin(δ)(kPa)	4.66	2.21			
100-A2-7.5	G* (kPa)				3.51	1.72
	δ (°)				78.5	81.3
	G*/Sin(δ)(kPa)				3.58	1.74
100-A2-15	G* (kPa)				2.31	1.26
	δ (°)				78.8	81.3
	G*/Sin(δ)(kPa)				2.35	1.28

All RTFO aged samples, except the C-V samples, had experienced one cycle of RTFO and PAV tests before they experienced the second RTFO test. It should be mentioned that in PG grade system, the interval temperature is 6 °C and the high temperature starts from 46 °C. The test temperatures for each group were selected according to the highest temperature that the sample passed the Superpave criteria.

As can be seen in Table 3-5 the high temperature of PG grade for aged asphalt binder (C-100) was 82 °C. Adding 5% and 10% of additive 1 decreased it to 76 °C and 64 °C respectively. This means that adding 5% of additive 1 changed the high temperature of PG grade by one grade and adding 10% changed it by two grades. For additive 2, adding

7.5% did not change the high temperature of PG grade, but adding 15% decreased it by one level from 82 °C to 76 °C. This fact shows that, in general, additive 1 was more effective than additive 2. According to the Table 3-6, the high temperature of PG grade for the aged asphalt binder (C-100) was 82 °C. Adding 5% and 10 % of additive 1 changed the PG high temperature from 82 °C to 70 °C and 64 °C respectively. For the second additive, adding 7.5 % did not change the grade, but adding 15% changed the grade from 82 °C to 76. °C. It can be concluded that additive 1 was able to make the aged asphalt binders softer than additive 2. Therefore, lower percentage of additive 1 than additive 2 was expected to modify asphalt binders.

The high temperature PG grade of unmodified (aged) asphalt binders was increased in comparison with virgin asphalt binders since aging of asphalt binders made them stiffer. Adding additives to aged asphalt binders made them softer. The maximum percentage of additive is the percentage that soften the modified asphalt binder, but not softer than the virgin asphalt binder.

Since the high temperature PG grade of virgin asphalt binder (C-V) was 64 °C, the Superpave criteria ($G^*/\sin(\delta)$) of all samples was measured at 64 °C and can be seen in Table 3-7.

Table 3-7 $G^*/\sin(\delta)$ value of all groups for base (unaged) and RTFO aged asphalt binder at 64 °C

Sample ID	Base asphalt (kPa)	RTFO aged (kPa)
C-V	1.72	4.25
100-A1-5	4.6	7.95
100-A1-10	1.39	2.21
100-A2-7.5	8.6	14.32
100-A2-15	4.8	9.4

The maximum percentage of additive is the percentage that decreases the G*/sin(δ) of each group to the G*/sin(δ) value of virgin binder which is highlighted in Table 3-7. By interpolation, the maximum percentage of additive for each additive was identified and shown in Table 3-8. The accepted value of maximum percentage of additives for each additive was highlighted in this Table 3-8.

Table 3-8 maximum percentage of additives according to the DSR test for base and RTFO aged modified asphalt

Sample ID	Base asphalt (kPa)	RTFO aged (kPa)
Additive1	9.48 %	8.22 %
Additive 2	21.5 %	22.85%

To evaluate the rheological properties of asphalt binders at the intermediate temperature, DSR test was run for PAV aged asphalt binders. G*×sin(δ) is the Superpave specification to control the fatigue cracking after long period in a pavement. G*×sin(δ) at the intermediate temperature should be less than 5000 kPa for PAV aged asphalt binders. The PAV residue represents binder that has been exposed to all the aging conditions to which binders are subjected during production and in-service. Table 3-9 represents the results of DSR test for PAV aged for each group at different temperatures.

Table 3-9 DSR test result for PAV aged asphalt binder

Sample ID	**Parameter**	**T (° C)**						
		13	**16**	**19**	**22**	25	**28**	**31**
C-V	G* (kPa)					6.50E +03		
	δ (°)					48.6		
	G*Sin(δ)(kPa)					4.87E +03		
C-100	G* (kPa)						9.65E +03	6.60E+ 03
	δ (°)						40.3	43
	G*Sin(δ)(kPa)						6.23E +03	4.50E+ 03
100-A1-5	G* (kPa)			1.00E +04	6.48E +03	4.28E +03		
	δ (°)			42	44.4	47.3		
	G*Sin(δ)(kPa)			6.70E +03	4.54E +03	3.14E +03		
100-A1-10	G* (kPa)	6.54E +03	4.77E +03	3.07E +03				
	δ (°)	45.2	47.4	50.1				
	G*Sin(δ)(kPa)	4.64E +03	3.51E +03	2.36E +03				
100-A2-7.5	G* (kPa)			1.08E +04	7.36E +03			
	δ (°)			38.3	41.1			
	G*Sin(δ)(kPa)			6.70E +03	4.83E +03			
100-A2-15	G* (kPa)	7.26E +03	4.83E +03					
	δ (°)	38.4	40.8					
	G*Sin(δ)(kPa)	4.51E +03	3.15E +03					

As can be seen in this table, the intermediate temperature PG grade of virgin asphalts was 25 °C and adding 5% of additive 1 and 7.5% of additive 2 changed it to 19 °C. Adding 10% of additive 1 and 15% of additive 2 decreased it to 16 °C. Again, it can be concluded that additive 1 was more efficient than additive 2 since by adding less amount of material softer sample was produced. Converting the intermediate temperature to the high temperature of PG grade for all groups, it could be seen that for none of the groups, this test was critical. In other word, this test leaded to the higher PG grade for samples than DSR test on base and RTFO aged asphalt binders.

3.3.3 Bending Beam Rheometer (BBR) Test

The low temperature performances of asphalt binders were evaluated through the bending beam rheometer (BBR) test. Superpave recommends a test temperature -12 °C for PG 64-22. In this study, -6, -12, -18, -24 and -30 °C were selected as test temperatures to determine the low temperature of the PG grade. Two important parameters of BBR test are the stiffness and m-value. The stiffness represents the hardness of asphalt binders at low temperature and m-value represents the ability of asphalt binders to release tensions result from the temperature and loading changes. If the stiffness of a sample is less than 300 MPa and the m-value is greater than 0.3 at the specific temperature the sample meets Superpave criteria at that temperature and can be tested for a lower temperature.

Table 3-10 shows the results of BBR test for PAV aged asphalt binders at different temperatures. It should be mentioned that since this test was very sensitive and significant for determination of the minimum (optimum) dosage of additives, for each case three samples were tested.

Table 3-10 BBR result for PAV aged samples for different temperatures

Sample ID	T1 (°C)	T2 (°C)	Parameter	T1 (°C)				T2 (°C)			
				Sample 1	Sample 2	Sample 3	Ave	Sample 1	Sample 2	Sample 3	Ave
C-V	-12	-18	m-value	0.331	0.331	0.324	0.329	0.263	0.268	0.262	0.264
			Stiffness (MPa)	232	233	231	232.0	479	460	469	469.3
C-100	-6	-12	m-value	0.318	0.319	0.315	0.317	0.271	--	0.279	0.275
			Stiffness (MPa)	163	163	163	163.0	317	--	315	316.0
100-A1-5	-12	-18	m-value	0.355	0.341	0.359	0.352	0.298	0.301	0.301	0.300
			Stiffness (MPa)	125	121	129	125.0	264	262	272	266.0
100-A1-10	-24	-30	m-value	0.309	0.322	0.328	0.320	0.248	0.248	0.245	0.247
			Stiffness (MPa)	238	243	243	241.3	490	479	497	488.7
100-A2-7.5	-12	-18	m-value	0.325	0.329	--	0.327	0.275	0.272	--	0.274
			Stiffness (MPa)	165	153	--	159.0	304	296	--	300.0
100-A2-15	-18	-24	m-value	0.328	0.323	--	0.326	0.268	0.268	0.277	0.271
			Stiffness (MPa)	130	140	--	135.0	268	272	272	270.7

As can be seen in this table, the low temperature PG grade of the virgin sample was -12 °C as it was expected. For C-100 sample which was experienced RTFO and PAV tests two times, the low temperature PG was -6 °C. Adding 5% and 10% of additive 1 decreased the lower PG grade to -18 °C and -24 °C respectively. For additive 2, adding 7.5% did not change the low PG grade, but adding 15% changed it to -18 °C. Again, it can be concluded that additive 1 was more impressive since with less amount of material than additive 2, softer asphalt binders at the low temperature was produced.

The low temperature PG grade of unmodified (aged) asphalt binders was decreased in comparison with virgin asphalt binders since aging of asphalt binders made them stiffer. Adding additives to the aged asphalt binder made it softer. The minimum (optimum) percentage of additive is the percentage that soft the modified asphalt binders as soft as the base (unaged) asphalt binders.

Since the low temperature PG grade of the virgin binder (C-V) was -12 °C, the Superpave specifications of all groups (stiffness and m-value) were measured at -12 °C and can be seen in Table 3-11.

Table 3-11 BBR Superpave criteria of all sample groups for PAV aged asphalt binder at -12 °C

Sample ID	Parameter at T = -12 °C	
	m-value	**Stiffness (MPA)**
C-V	0.329	232.0
100-A1-5	0.352	125.0
100-A1-10	0.466	50.1
100-A2-7.5	0.327	159.0
100-A2-15	0.381	78.4

The optimum percentage of the additive is the percentage that decrease the stiffness and m-value of each group to the stiffness and m-value of the virgin binder which was highlighted in Table 3-11. By interpolation and extrapolation, the optimum percentage of additives were identified 3.99% and 7.77% for additive 1 and 2 respectively. Since the m-value was more critical, the optimum percentage of additive based on this parameter was higher than the optimum percentage of additive based on stiffness. The final optimum percentage for each additive was identified according to this parameter.

3.4 Results and Discussions to Verify the Optimum Dosage of Additives

After determination of the optimum dosage for additives, the second set of rheological tests was proposed to verify the optimum dosage. For this set of tests, it was needed to mix 60% of aged binders (an asphalt binder which already was aged by running RTFO and PAV tests on it once) with 40% of the virgin binder (PG 64-22). The major reason of selecting this proportion (60% aged asphalt binder plus 40% virgin asphalt binder) was the percentage of RAP material in the mix design in **Chapter 4**. Three mix designs with 30%, 70% and 100% of RAP will be mentioned in **Chapter 4**. The optimum percentage of additives for 100% aged asphalt binder was used for mixture with 100% RAP material. The result of the optimum percentage of modified asphalt binder with 60% aged asphalt binder and 40% virgin binder was used both the mix design with 70% RAP (60% by binder replecement). Additionally, this result can be used to verify the optimum percentage of additives in the previous section. 5% and 10% of modified asphalt binder with additive 1 and 7.5% and 15% of asphalt modified binder with additive 2 were prepared. It should be mentioned that since the additives are supposed to rejuvenate the aged asphalt binder, 5, 10, 7.5 and 15 percentage of aged asphalt binder additives were added to the samples.

3.4.1 Test Plan and Test Numbers to Verify the Optimum Dosage of Additives

Table 3-12 demonstrates the sample ID and content of each group of samples. Table 3-13 represents the test plan and number of tests that were needed to verify the optimum percentage of additives.

Table 3-12 Sample ID and contents of each sample group for verification

Sample ID	Content of Sample
C-60	60% Aged (RTFO + PAV) + 40 % Virgin (PG 64-22)
60-A1-5	60% Aged (RTFO + PAV) + 40 % Virgin (PG 64-22) + 5% Additive One
60-A1-10	60% Aged (RTFO + PAV) + 40 % Virgin (PG 64-22) + 10% Additive One
60-A2-7.5	60% Aged (RTFO + PAV) + 40 % Virgin (PG 64-22) + 7.5% Additive Two
60-A2-15	60% Aged (RTFO + PAV) + 40 % Virgin (PG 64-22) + 15% Additive Two

Aged*: Original binder(PG64-22) aged by running RTFO and PAV tests on it once

Table 3-13 Rheological test plan and number of tests for verification

	Tests						
Sample ID	**RV**	**DSR (Virgin)**	**DSR (RTFO)**	**DSR (PAV)**	**MSCR**	**BBR (T1)**	**BBR (T2)**
C-60	XXX	XXX	XXX	XXX	XXX	XXX	XXX
60-A1-5	XXX	XXX	XXX	XXX	XXX	XXX	XXX
60-A1-10	XXX	XXX	XXX	XXX	XXX	XXX	XXX
60-A2-7.5	XXX	XXX	XXX	XXX	XXX	XXX	XXX
60-A2-15	XXX	XXX	XXX	XXX	XXX	XXX	XXX
Total	15	15	15	15	15	30	

*note: X means one replicate

3.4.2 Sample Preparation to Verify the Optimum Dosage of Additives

To verify the optimum dosage of additives, first the virgin asphalt binder was aged by running one RTFO and PAV tests on it. Then, the aged and virgin asphalt binder were heated to about 120°C and mixed together with the proportion of 60% aged asphalt binder and 40% PG 64-22. Finally, the mix of virgin and aged asphalt binders was mixed with each of the additives. The modified asphalt binder was treated as a base sample (virgin) and all Suprpave tests run on it. For instance, the DSR for RTFO aged asphalt binder means the modified sample that 60% of it had already experienced one cycle of RTFO and PAV tests and 40% of it was virgin asphalt binder, was aged on RTFO and then DSR test run on it.

3.4.3 Dynamic Shear Rheometer (DSR) Test

Table 3-14 and

Table 3-15 provide the results of DSR test for virgin and RTFO aged asphalt binders at different temperatures respectively. G*/sin(δ) should be greater than 1.0 kPa before aging to control rutting during transport, storage, and handling of the asphalt mixtures and greater than 2.2 kPa after RTFO to prevent rutting during mix production and construction of asphalt mixtures.

Table 3-14 DSR test result for base (unaged) asphalt binder for verification of the optimum dosage

Sample ID	**Parameter**	**T (°C)**			
		64	**70**	**76**	**82**
C-60	G* (kPa)			1.68	0.804
	δ (°)			84.5	86.2
	G*/Sin(δ)(kPa)			1.69	0.806
60-A1-5	G* (kPa)		1.39	0.709	
	δ (°)		85.5	87.1	
	G*/Sin(δ)(kPa)		1.39	0.71	
60-A1-10	G* (kPa)	1.28	0.607		
	δ (°)	85.9	87.3		
	G*/Sin(δ)(kPa)	1.28	0.607		
60-A2-7.5	G* (kPa)			1.04	0.533
	δ (°)			85.6	87.1
	G*/Sin(δ)(kPa)			1.05	0.534
60-A2-15	G* (kPa)		1.61	0.856	
	δ (°)		83.1	84.9	
	G*/Sin(δ)(kPa)		1.63	0.859	

Table 3-15 DSR test result for RTFO aged asphalt binder for verification of the optimum dosage

Sample ID	Parameter	T (° C)				
		58	**64**	**70**	**76**	**82**
C-60	G* (kPa)			7.3	3.6	1.64
	δ (°)			78	80.8	83.3
	G*/Sin(δ)(kPa)			7.47	3.64	1.65
60-A1-5	G* (kPa)			2.72	1.29	
	δ (°)			82.7	85	
	G*/Sin(δ)(kPa)			2.74	1.3	
60-A1-10	G* (kPa)		2.37	1.15		
	δ (°)		83.5	85.5		
	G*/Sin(δ)(kPa)		2.39	1.15		
60-A2-7.5	G* (kPa)			4.17	2.01	
	δ (°)			79.2	82.3	
	G*/Sin(δ)(kPa)			4.25	2.03	
60-A2-15	G* (kPa)			3.03	1.5	
	δ (°)			79	82.3	
	G*/Sin(δ)(kPa)			3.09	1.51	

As can be seen in Table 3-14, the high temperature PG grade of aged asphalt binder (C-60) was 76 °C. This temperature was 82 °C for C-100 since all of the sample experienced one cycle of PAV and RTFO tests, but for C-60, 60% of sample experienced one cycle of RTFO and PAV test. Adding 5% and 10% of additive 1 decreased it to 70 °C and 64 °C respectively. This means that 5% of additive 1 was able to change the high temperature of PG grade of aged asphalt binders one grade. Therefore, adding 10% of additive 1 decreased the PG grade two levels same as the previous section. For additive 2, adding 7.5% did not change the high temperature of PG grade, but adding 15% of additive 2 decreased it one level from 76 °C to 70 °C same as previous section. As indicated in

Table 3-15, again the high temperature of PG grade for the aged asphalt binder (C-60) was 76 °C. Adding 5% and 10 % of additive 1 changed the PG high temperature of aged binder from 76 °C to 70 °C and 64 °C respectively. For the second additive, adding 7.5 % did not change the grade, but adding 15% changed the grade from 76 °C to 70. °C.

Since the high temperature of PG grade for the unaged binder (C-V) was 64 °C, the Superpave criteria of all groups (G*/sin(δ)) was measured at 64 °C and can be seen in Table 3-16.

Table 3-16 G*/sin(δ) value of all sample groups for base (unaged) and RTFO aged asphalt binder at 64 °C to verify the optimum dosage

Sample ID	Base asphalt (kPa)	RTFO aged (kPa)
C-V	1.72	4.25
60-A1-5	2.07	5.48
60-A1-10	1.28	2.39
60-A2-7.5	4.2	8.5
60-A2-15	3.1	6.18

The maximum percentage of additive is the percentage that decreases the G*/sin(δ) of each group to the G*/sin(δ) of the virgin binder which was highlighted in Table 3-14. By interpolation, the maximum percentage of additive for each additive was identified and shows in Table 3-17. The accepted value of the maximum percentage for each additive was highlighted in this table. This maximum percentage was 8.22% and 21.5% for additive 1 and 2 respectively.

Table 3-17 G*/sin(δ) value of all sample groups for base (unaged) and RTFO aged asphalt binder at 64 °C to verify the optimum dosage

Sample ID	Base asphalt (kPa)	RTFO aged (kPa)
Additive1	7.2 %	6.99 %
Additive 2	24.4 %	21.23%

To evaluate the rheological properties of asphalt binders at intermediate temperature, DSR test was conducted for PAV aged asphalt binders. G*×sin(δ) is the Superpave specification to control the fatigue cracking after long periods in a pavement. G*×sin(δ) at the intermediate pavement temperature should be less than 5000 kPa for PAV aged asphalt binders. Table 3-18 reports the results of DSR test for PAV aged asphalt binders for each group at different temperatures.

Table 3-18 DSR test result for PAV aged asphalt binder to verify the optimum dosage of additives

Sample ID	Parameter	T (° C)						
		13	**16**	**19**	**22**	**25**	**28**	**31**
C-60	G* (kPa)						7.31E +03	4.88E +03
	δ (°)						44	46.7
	G*Sin(δ)(kPa)						5.08E +03	3.55E +03
60-A1-5	G* (kPa)				7.85E +03	5.25E +03		
	δ (°)				45.8	48.3		
	G*Sin(δ)(kPa)				5.63E +03	3.92E +03		
60-A1-10	G* (kPa)		7.67E +03	5.17E +03				
	δ (°)		45.8	48.7				
	G*Sin(Δ)(kPa)		5.50E +03	3.89E +03				
60-A2-7.5	G* (kPa)			1.18E +04	8.49E +03	5.60E +03		
	δ (°)			40.8	43.3	46.3		
	G*Sin(δ)(kPa)			7.71E +03	5.82E +03	4.05E +03		
60-A2-15	G* (kPa)	1.13E +04	7.24E +03					
	δ (°)	39.2	42.5					
	G*Sin(δ)(kPa)	7.16E +03	4.89E +03					

As can be seen in this table, the intermediate temperature of virgin asphalts was 28 °C. Adding 5% and 10% of the additive 1, changed it to 22 °C and 16 °C. For additive 2, adding 7.5% and 15% decreased it to 22 °C and 13 °C respectively. Again, it can be concluded that additive 1 was more efficient than additive 2. It should be mentioned that for none of the groups this test was critical same as the previous section.

3.4.4 Bending Beam Rheometer (BBR) Test

Superpave recommends a test temperature -12 °C for PG 64-22. For this section, -12, -18 and -24 °C (6 °C interval) were selected to better understand the rheological properties of modified asphalt at low temperatures and determine the low temperature of the PG grade.

Table 3 10 reveals the results of BBR test for PAV aged asphalt binders at different temperatures. It should be mentioned that for each case three samples were tested.

Table 3-19 BBR result for PAV aged samples for different temperatures to verify the optimum dosage

Sample ID	T1 (° C)	T2 (° C)	Parameter	T1 (° C)				T2 (° C)			
				S1	S2	S3	Ave	S1	S2	S3	Ave
C-60	-12	-18	m-value	0.294	0.287	--	0.291	0.239	0.239	--	0.239
			Stiffness (MPa)	290	267	--	278.5	529	513	--	521.0
60-A1-5	-12	-18	m-value	0.354	0.346	0.352	0.351	0.29	0.291	0.291	0.291
			Stiffness (MPa)	150	151	154	151.7	323	315	317	318.3
60-A1-10	-24	-30	m-value	0.293	0.297	0.296	0.295	0.217	0.227	0.231	0.225
			Stiffness (MPa)	358	369	352	359.7	687	713	695	698.3
60-A2-7.5	-12	-18	m-value	0.32	0.33	0.325	0.325	0.278	0.277	0.275	0.277
			Stiffness (MPa)	166	172	172	170.0	338	339	339	338.7
60-A2-15	-18	-24	m-value	0.324	0.308	0.318	0.317	0.264	0.264	0.255	0.261
			Stiffness (MPa)	201	212	200	204.3	394	390	404	396.0

As can be identified from this table, the low temperature PG grade of C-60 was -6 °C. Adding 5% and 10% of additive 1, decreased it to -12 °C and -18 °C respectively. For additive 2, adding 7.5% changed the low PG grade to -12 °C, and adding 15% changed it to -18 °C. Again, it can be concluded that additive 1 was more efficient than additive 2 since with less material than additive 2, softer asphalt binders at low temperature were produced.

Since the low temperature grade of the unaged (virgin) asphalt binder (C-V) was -12 °C, as shown in Table 3-10, the stiffness and m-value of all groups were measured at -12 °C and can be seen in Table 3-20.

Table 3-20 BBR Superpave criteria of all sample groups for PAV aged asphalt binder at -12 °C

Sample ID	Parameter at T = -12 °C	
	m-value	**Stiffness (MPA)**
C-V	0.329	232.0
60-A1-5	0.351	151.7
60-A1-10	0.44	150.4
60-A2-7.5	0.325	170.0
60-A2-15	0.373	140.5

The optimum percentage of additive is the percentage that decreases the stiffness and m-value of each group to the stiffness and m-value of virgin binders which was highlighted in Table 3-20. By interpolation and extrapolation, the optimum percentage of additive 1 was 3.76% and for additive 2 was 8.12%. These value was 3.99% and 7.77% in the previous section. The differences between the optimum dosage of 100% aged samples and 60% aged samples were acceptable and small.

3.5 Results and Discussions to Verify the Linear Relationship Between Rheological Properties of Modified Asphalt Binder and Additive Dosage

One of the fundamental assumptions to identify the optimum dosage of additives was linear relationship between the rheological properties of modified asphalt binders and the additives dosage. To check and verify this assumption, the Superpave tests were done on the modified asphalt binders with the optimum dosage of additives. Comparing the results of these tests with rheological test results of the virgin asphalt binder (PG 64-22), it can be concluded whether the assumption of linear relationship was true or not. It should be mentioned that the ultimate goal of this chapter was determination of the optimum dosage of additive, not the type of relationship between rheological properties and dosage of additives. Therefore, with these tests, not only the linear relationship between rheological properties and dosage of additives but also the accuracy of the optimum dosage can be evaluated. Table 3-21 lists the sample ID and content of the samples

Table 3-21 Sample ID and contents of each sample to verify linear relationship between the dosage of additives and asphalt binder rheological properties

Sample ID	Content of Sample
100-A1-3.99	100% Aged (RTFO + PAV) + 3.99% Additive 1
100-A2-7.77	100% Aged (RTFO + PAV) + 7.77% Additive 2

DSR on virgin, RTFO, and PAV aged modified asphalt binders along with BBR on PAV aged asphalt binders were conducted. DSR test results and BBR test results can be found in Table 3-22 and Table 3-23 respectively.

Table 3-22 DSR test result for virgin, RTFO, PAV modified asphalt binder and PG64-22

Sample ID	Parameter	unaged	RTFO	PAV
		T= 64 °C	T= 64 °C	T= 25 °C
C-V	G* (kPa)	1.72	4.22	6.50E+03
	δ (°)	86.7	83.3	48.6
	G*/Sin(δ)(kPa) Or G*/Sin(δ)(kPa)	1.72	4.25	4.87E+03
100-A1-3.99	G* (kPa)	5.2	8.26	4.45E+03
	δ (°)	80.2	79.1	46.8
	G*/Sin(δ)(kPa) Or G*/Sin(δ)(kPa)	5.28	8.41	3.21E+03
100-A2-7.77	G* (kPa)	8.36	13.53	3.92E+03
	δ (°)	77.4	74.3	43.6
	G*/Sin(δ)(kPa) Or G*/Sin(δ)(kPa)	8.56	14.05	2.70E+03

Table 3-23 BBR test result for PAV modified asphalt binder and PG64-22

Sample ID	Parameter	T = -12 °C			
		Sample 1	Sample 2	Sample 3	Ave
C-V	m-value	0.331	0.331	0.324	0.329
	Stiffness (MPa)	232	233	231	232.0
100-A1-3.99	m-value	0.328	0.331	0.329	0.331
	Stiffness (MPa)	102.3	100.4	103.1	101.9
100-A2-7.77	m-value	0.322	0.322	0.331	0.325
	Stiffness (MPa)	157.2	159.7	157.7	158.2

Since the optimum dosage of additives was identified according to the m-value, the error percentage for this parameter was calculated and presented in Table 3-24

Table 3-24 Error percentage for m-value

Sample ID	Difference	% Error
100-A1-3.99	0.002	% 0.6
100-A2-7.77	0.004	% 1.2

As can be seen in these tables, the rheological test results of the modified asphalt binders with optimum dosage of additives were close to the results of the rheological test of the virgin asphalt binder. However, in general, additive 1 made the modified asphalt binder softer than virgin binder and additive 2 made it harder. Since the acceptable results for the modified asphalt binders were observed, the assumption of the linear relationship between rheological properties of asphalt binders and the dosage of additives was true.

3.6 Multiple Stress Creep Recovery (MSCR) Test

The MSCR test method (AASHTO T 350) uses the well-established creep and recovery test concept. In the MSCR test method, one second shearing creep load is applied to the RTFO-aged asphalt binder by using a DSR. After the one second load is removed, the test sample is allowed to release the creep load for nine seconds. The test is started with the application of a low stress 0.1 kPa for 10 creep/recovery cycles, and then the stress is increased to 3.2 kPa, which is repeated for an additional 10 cycles. Figure 3-4 and Figure 3-4 represent how the loads are applied in the MSCR test method. [68]

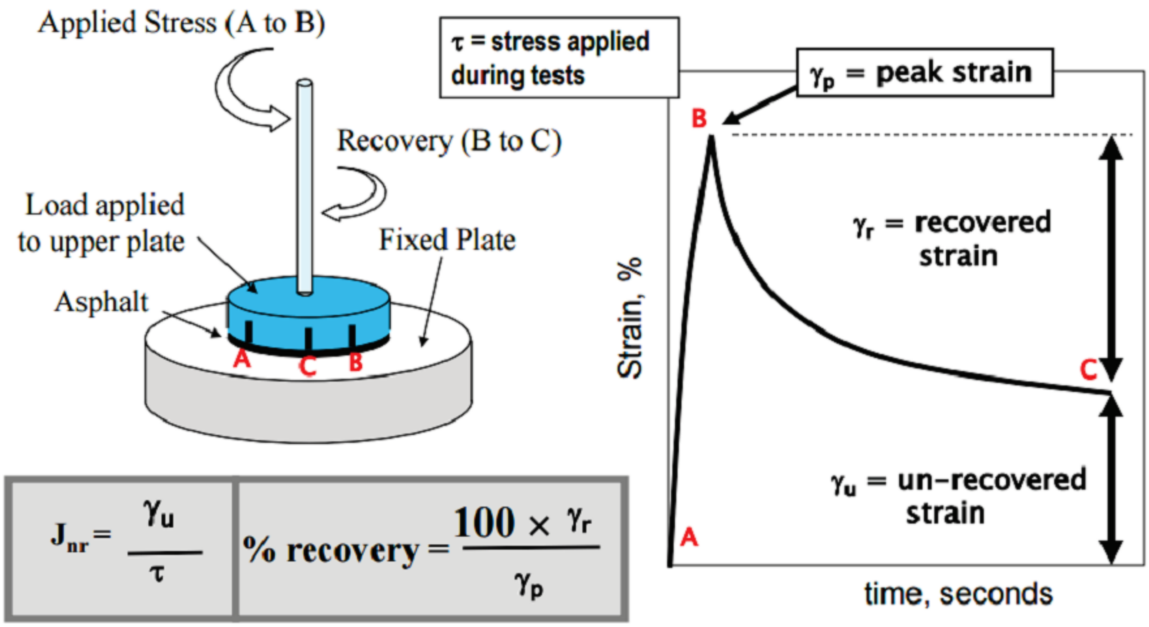


Figure 3-4 The concept of the Percent Recovery and J_{nr} Value [68]

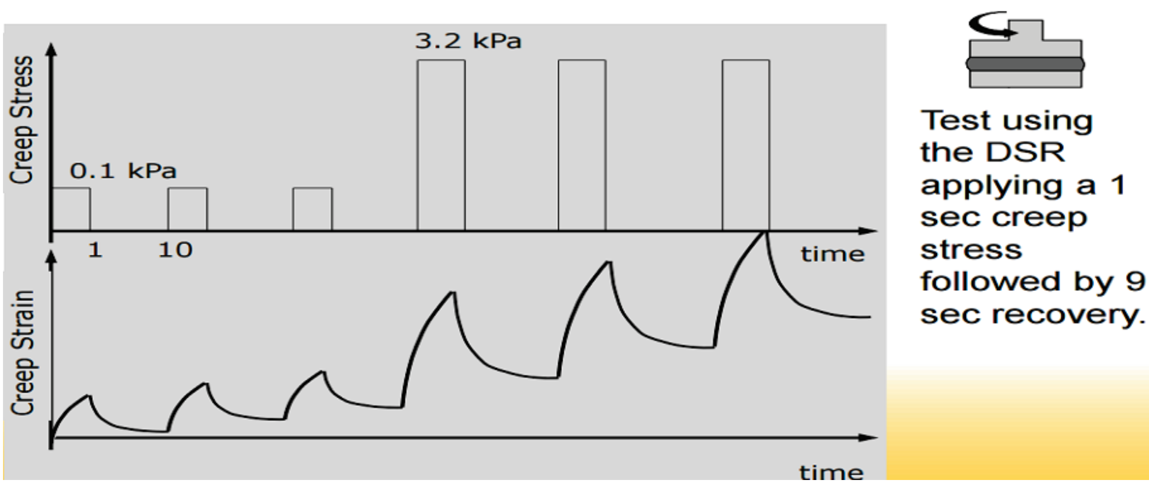


Figure 3-5 Loading scheme of MSCR test method [68]

The MSCR test gives two major output parameters, namely, J_{nr} and percent creep recovery (%R), as shown in Figure 3-4. The J_{nr} value indicates the amount of residual strain left in the binder within the linear and nonlinear viscoelastic range at high temperatures and high stress levels. The percent creep recovery measures how much the asphalt specimen returns to its original position after the load is released. Table 3-25 summarises the results of MSCR tests for all samples.

Table 3-25 The result of MSCR test for samples

		100 Pa		3200 Pa			
Sample ID	**Temperature (°C)**	**Percent recovery**	**J_{nr} (kPa^{-1})**	**Percent recovery**	**J_{nr} (kPa^{-1})**	**Difference in Percent Recovery**	**Percent Difference in J_{nr}**
C-V	64	4.88%	2.123	1.76%	2.282	63.94%	7.52%
C-100	82	8.10%	2.475	2.33%	2.869	71.20%	15.93%
100-A1-5	70	7.63%	2.367	2.14%	2.704	71.89%	14.21%
100-A1-10	64	4.06%	4.054	1.00%	4.507	75.31%	11.18%
100-A2-7.5	76	11.34%	2.241	3.15%	2.721	72.22%	21.45%
100-A2-15	76	16.13%	3.010	2.59%	4.179	83.96%	38.87%
C-60	76	7.35%	2.390	2.10%	2.725	71.39%	14.02%
60-A1-5	70	4.23%	3.371	1.29%	3.720	69.62%	10.36%
60-A1-10	64	3.78%	3.877	0.98%	4.249	74.01%	9.60%
60-A2-7.5	70	8.31%	1.972	3.12%	2.226	62.40%	12.87%
60-A2-15	70	12.74%	2.462	4.02%	3.002	68.45%	21.94%

It is more desired to have a higher percentage of recovery and a lower value of J_{nr}. In general, the percentage of recovery for all samples was low since the additives were not polymer-based additives or had less amount of polymers. In all groups, the percentage of recovery for 0.1 kPa was higher than the percentage of recovery for 3.2 kPa. This proved that the modified asphalt binders performed better at low stress level than high stress level. Also, it should be mentioned, unlike the previous sections, modified asphalt binders with additive 2 had more recoverability than modified asphalt binder with additive 1 for both levels of stress. It is because additive 2 has SBS which is a polymer chemical material. Additionally, adding more additive 2 increased the percentage of recovery, but adding more additive 1 decreased the percentage of recovery. Comparing 100% aged samples with 60% aged samples, it can be concluded that 100% modified samples had more percentage of recovery because these samples had more additive materials than 60% aged samples.

Overall, adding additives, increased J_{nr} which was not desire. J_{nr} was almost same for all groups and for both stress levels. This fact showed that by increasing the stress level, the hardness of modified asphalt sample was not changed. Again, the smaller value of J_{nr} was observed for additive 2 than additive 1. This fact proved that additive 2 had better performance than additive 1 according to the MSCR test. However, adding more additive 1 decreased J_{nr} which was more favorable, but adding more additive 2 increased it. 100% aged modified samples had smaller value of J_{nr} in comparison with 60% aged modified samples because of more additives materials that 100% aged samples had.

3.7 Fourier Transform Infrared (FTIR) Spectroscopy

FTIR is a technique that can determine the functional characteristics of a material. The quantitative and qualities analysis can be conducted using FTIR by observing the absorbance spectra. FITR is also utilized to analyze the oxidative aging of asphalt binders by observing the amount change of carbonyl (C=O) and sulfoxide (S=O) [68]. The oxidative aging index is determined by the bond ratio changes before and after the aging. The bond ratio is determined by the equations below [68]:

$$I_{S=O} = \frac{\text{Area of the sulfoxide band centered around 1030 } cm^{-1}}{\sum Area\ of\ the\ spectra\ bands\ between\ 600\ cm^{-1} and\ 2000\ cm^{-1}}$$

$$I_{C=O} = \frac{\text{Area of the sulfoxide band centered around 1700 } cm^{-1}}{\sum Area\ of\ the\ spectra\ bands\ between\ 600\ cm^{-1} and\ 2000\ cm^{-1}}$$

where, $I_{S=O}$ is sulfoxide index, $I_{C=O}$ is carbonyl index,

FTIR analysis was performed on the aged asphalt with and without the rejuvenators. Figure 3-6 and Figure 3-7 show FTIR spectra for the base (C-100) asphalt, aged asphalt, as well as the aged asphalt containing rejuvenators 1 and 2 respectively. The sulfoxide (S=O) peak occurring at 1032 cm-1 corresponds to the oxidation of compounds containing sulfur and the carbonyl (C=O) peak at 1699 cm-1 corresponds to the oxidation of carbonyl compounds. The saturated C-H peak occurs at 1459 cm-1. To determine if the use of rejuvenators would minimize the oxidation, the spectra of all samples were analyzed using above equations. It should be mentioned that for each sample group three samples were tested and analyzed.

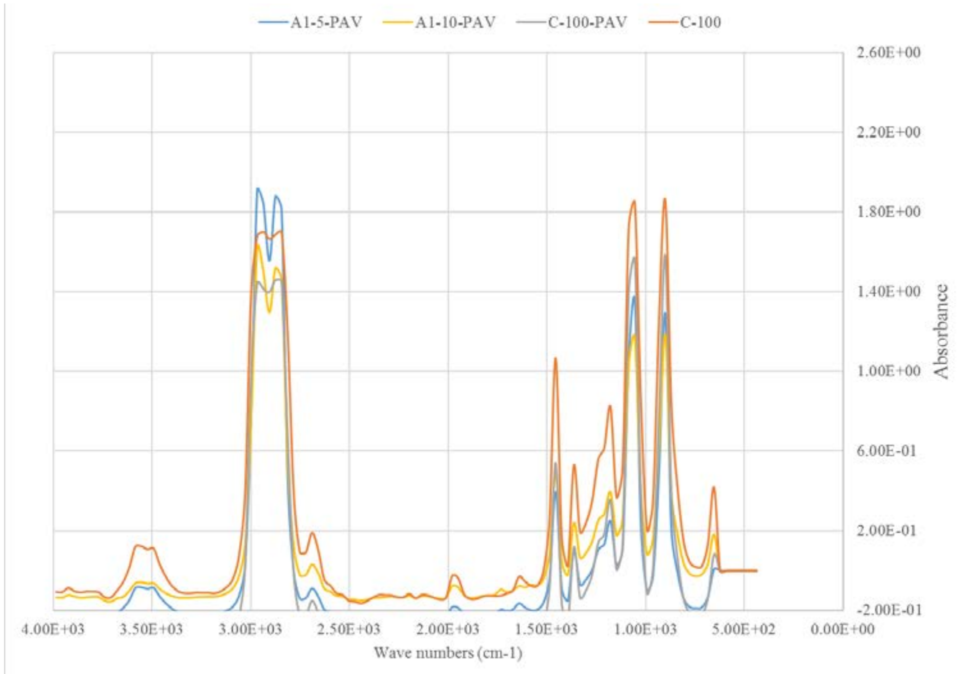


Figure 3-6 FTIR test results for aged sample and rejuvenated samples with additive 1

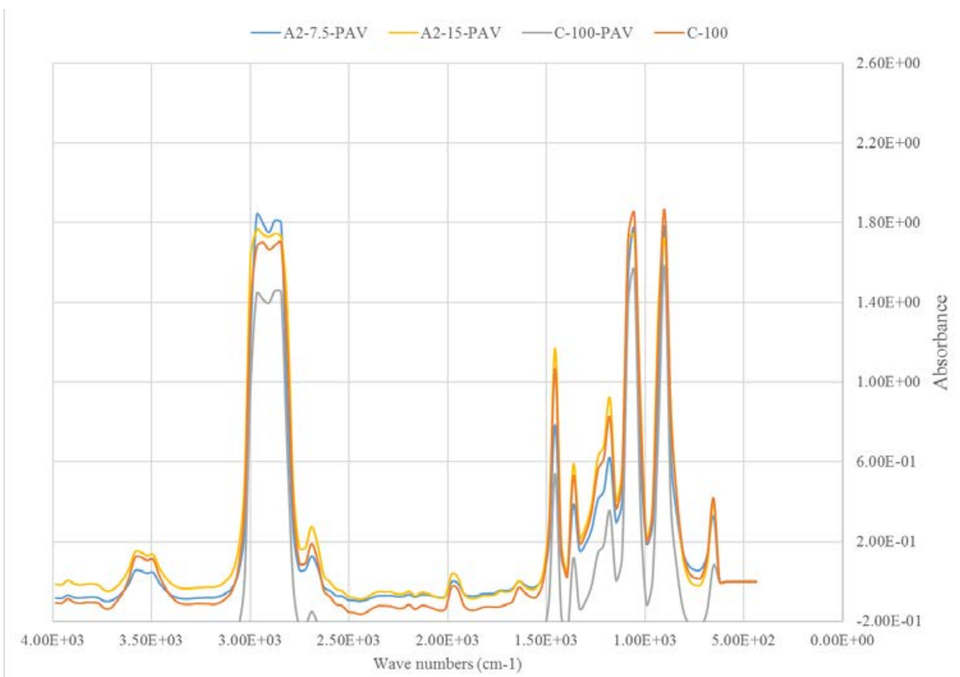


Figure 3-7 FTIR test results for aged sample and rejuvenated samples with additive 2

Figure 3-8 displays how the areas under FTIR spectra for Sulfoxide and Carbonyl and C-H functional groups by manually identifying a baseline was calculated. The sulfoxide index and carbonyl index values are summarized in Table 3-26.

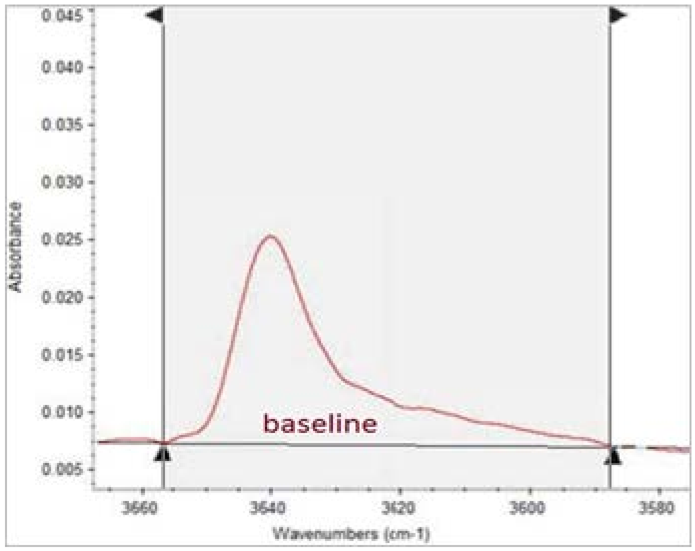


Figure 3-8 FTIR spectrum area measurement

Table 3-26 Sulfoxide and Carbonyl index values for each binder type

Sample ID	C-H Area	S=O Area	C=O Area	Sulfoxide Index	Carbonyl Index
C-100	20.41	1.61	1.72	7.89	8.43
C-100- PAV	25.7	2.8	2.43	10.89	9.46
A1-5-PAV	19.6	1.72	1.84	8.78	9.39
A1-10-PAV	21.56	1.87	2.03	8.67	9.42
A2-7.5-PAV	21.3	1.84	1.81	8.64	8.50
A2-15-PAV	22.5	1.8	1.7	8.00	7.56

The PAV-aged samples represented an increase in the SI value. The rejuvenated asphalt samples had lower SI values than the PAV-aged asphalt binders, but they had higher SI value than virgin binders. All rejuvenators reduced the oxidation of sulfur and carbon. Further, additive 2 reduced the oxidation of carbon more than additive 1.

3.8 Summary of the Chapter

In this chapter, the Superpave tests were conducted to identify the optimum dosage of two new eco-friendly WIR additives. DSR test results on virgin and RTFO sample were used to identify the maximum dosage of additives and BBR test results on PAV aged sample were used to identify the minimum (optimum) dosage of additives. For additive 1 and 2, 3.99% and 7.77% were measured as the optimum dosage of additives. The second set of Superpave tests were done on 60% aged plus 40% virgin asphalt binders with additives. The major intent of these tests was verifying the optimum dosage of additives. 3.76% for additive 1 and 8.12% for additive 2 were the optimum dosage of additives according to the second set of tests which correlated with the first set of tests. Then, the MSCR and FTIR test were executed on modified asphalt binder with WIR additives to better understand the effects of additives on recoverability, rutting resistance and aging mechanism of modified asphalt binders. In Chapter 4, asphalt mixture tests will be done to evaluate the performance of modified asphalt mixture with additives. Figure 3-9 illustrates the summary of this chapter.

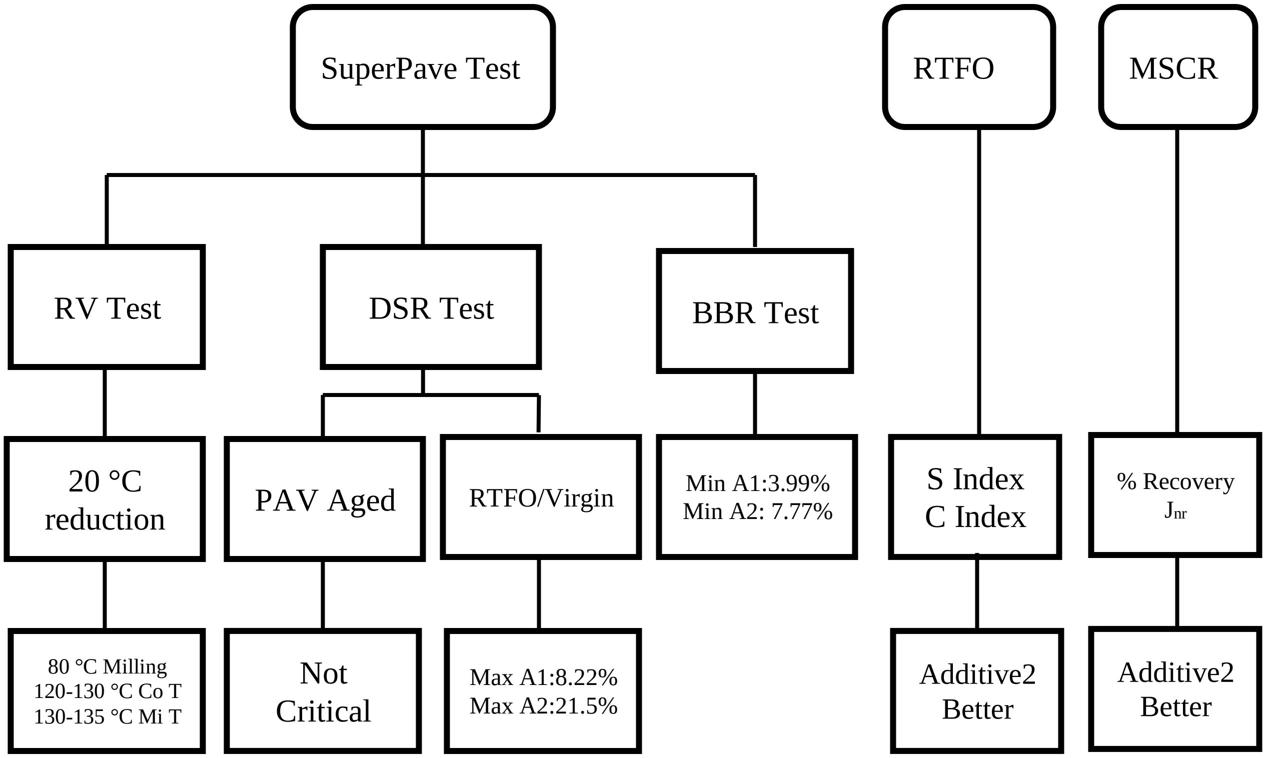


Figure 3-9 Chart. Summary of chapter 3

CHAPTER 4: MECHANICAL PERFORMANCE EVALUATION FOR ASPHALT MIXTURES MODIFIED BY WIR ADDITIVES

In this chapter, mechanical performances of asphalt mixtures modified by eco-friendly WRI additives were investigated. The asphalt binder was blended with the optimum dosage of each of the additives, measured in **chapter 3,** by weight. Then, it was mixed with the aggregate to make the asphalt mixtures. The mechanical performances of asphalt mixtures including rutting performance, moisture susceptibility, and low temperature cracking were evaluated in this chapter. The corresponding test methods are Hamburg Wheel Tracking (HWT) Test for rutting and moisture susceptibility and Disk-Shaped Compact Tension (DCT) Test for thermal fracture properties of asphalt mixtures. Since one of the objects of this research is increasing the percentage of RAP in recycling process, three different mix designs with 30%, 70%, and 100% RAP by weight were proposed. For each of the three mixtures, all the performance tests run and the maximum percentage of RAP that can be used in mixtures was determined.

4.1 Introduction

This section of the research addressed the design, laboratory preparation and performance characterization of WIR modified asphalt mixtures. Essentially, this chapter addressed the challenge of whether WIR modified asphalt mixtures prepared with typical Iowa aggregate materials can work satisfactorily when used for road pavements in the State. Additionally, if the performance of modified mixtures is acceptable, how much recycled materials can be used.

Three characterization approaches were selected to achieve the principal objective of evaluating the mechanical performance properties of WIR modified asphalt mixtures in relation to its resistance against permanent deformation, moisture susceptibility, and low temperature cracking. The characterization methods used were: 1) Permanent deformation or rutting and moisture susceptibility using the Hamburg Wheel Tracking (HWT) Test; 2) Disk-Shaped Compact Tension (DCT) Test for low temperature cracking evaluation.

4.2 Material Preparation

4.2.1 Asphalt Binders

In this study, first, the WIR additives mixed with virgin asphalt binder (PG 64-22) according to the percentage of RAP in mixture. Higher percentage of RAP in mixtures needed more WIR additives. The weight of additives was equal to the optimum dosage of each additive times the weight of asphalt binders in RAP materials. This is because each additive is supposed to rejuvenate only the aged asphalt binder, not the virgin asphalt binder. The asphalt mixture was compacted by mixing the aggregate, RAP and the WIR modified asphalt binders (For the100% RAP mixture type, no virgin aggregate was added). The petroleum base asphalt binder used in this study was PG 64-22, which was also the control asphalt binder in this study. The preparation procedure of modified asphalt binders including preheating the control asphalt binder and the WIR additives to120˚C and 100˚C, and mixing them for 5 minutes at 120˚C using a normal spoon was done before mixture preparation.

4.2.2 RAP Material Properties

RAP materials used for this study were provided by LL Pelling company, which were collected from a stockpile of the milled pavement surface (top 1 inch) of Highway 1 in Iowa City. Volumetric properties of extracted aggregate gradations of RAP material were measured. The extraction test was performed on a random sample of RAP materials following the ASTM D2172/D2172M. The apparent and bulk specific gravities of the extracted aggregates were then measured as 2.678 and 2.619, respectively, following the AASHTO T 84 and 85.

4.2.2.1 Gradation of extracted aggregates from RAP materials

The gradation test was performed on extracted aggregates from RAP materials. Each of the stockpiles was divided in four parts and, after mixed very well, all four parts were added together. Five RAP samples were then selected from each of the RAP stockpiles, which would represent each RAP stockpile. The aggregate gradations of these five samples are summarized in Table 4-1 and plotted in Figure 4-1. As can be seen in this figure, overall, gradation curves were above the maximum density line which indicates large amounts of fine materials.

Table 4-1 Aggregate gradation of 5 samples extracted material

Sieve Size		Samples of Extracted Aggregate Gradation (% passing)				
in	**mm**	**1**	**2**	**3**	**4**	**5**
3/4 in	19	100%	100%	100%	100%	100%
1/2 in	12.5	93.5%	95.3%	91.5%	96.3%	92.2%
3/8 in	9.5	86.7%	88.7%	84.6%	90.9%	86.1%
#4	4.8	68.2%	75.8%	69.4%	77.7%	69.2%
#8	2.4	49.8%	60.8%	52.0%	63.3%	51.4%
#16	1.2	36.3%	48.9%	38.8%	51.1%	39.6%
#30	0.6	27.1%	39.6%	29.0%	42.8%	31.8%
#50	0.3	12.7%	28.1%	16.8%	31.5%	20.8%
#100	0.15	6.0%	23.3%	11.7%	26.1%	17.0%
#200	0.075	3.4%	19.9%	8.7%	23.4%	13.5%

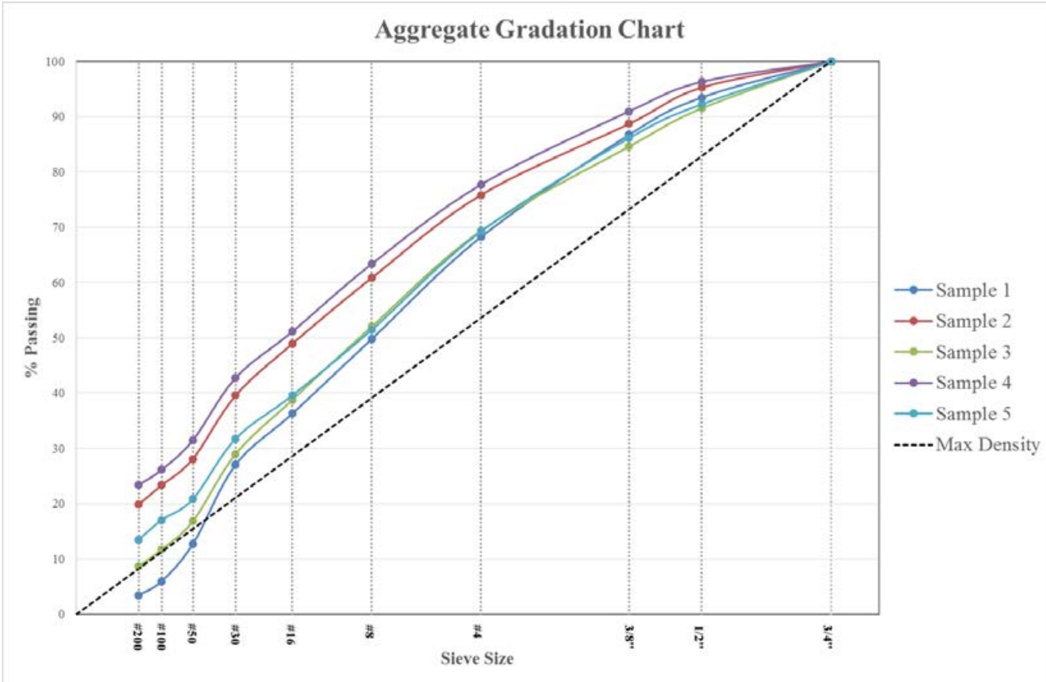


Figure 4-1 Aggregate gradation of 5 extracted samples

The average gradation of five samples are summarized in Table 4-2 and plotted in Figure 4-2. As can be seen from Figure 4-2, the gradation did not meet the Superpave graduation requirements especially in the dust content. For the purpose of evaluating the potential of using 100% RAP materials, RAP materials were used as is without replacing RAP materials with virgin aggregates.

Table 4-2 Average aggregate gradation of the RAP stockpile

Sieve Size		Average aggregate gradation (% passing)	Superpave limitations	
ID	mm		Min	Max
3/4 in	19	100.0%	90%	100%
1/2 in	12.5	93.7%	--	90%
3/8 in	9.5	87.4%	--	--
#4	4.8	72.1%	--	--
#8	2.4	55.5%	23%	49%
#16	1.2	42.9%	--	28%
#30	0.6	34.0%	--	24%
#50	0.3	22.0%	--	--
#100	0.15	16.8%	--	--
#200	0.075	13.8%	2%	8%

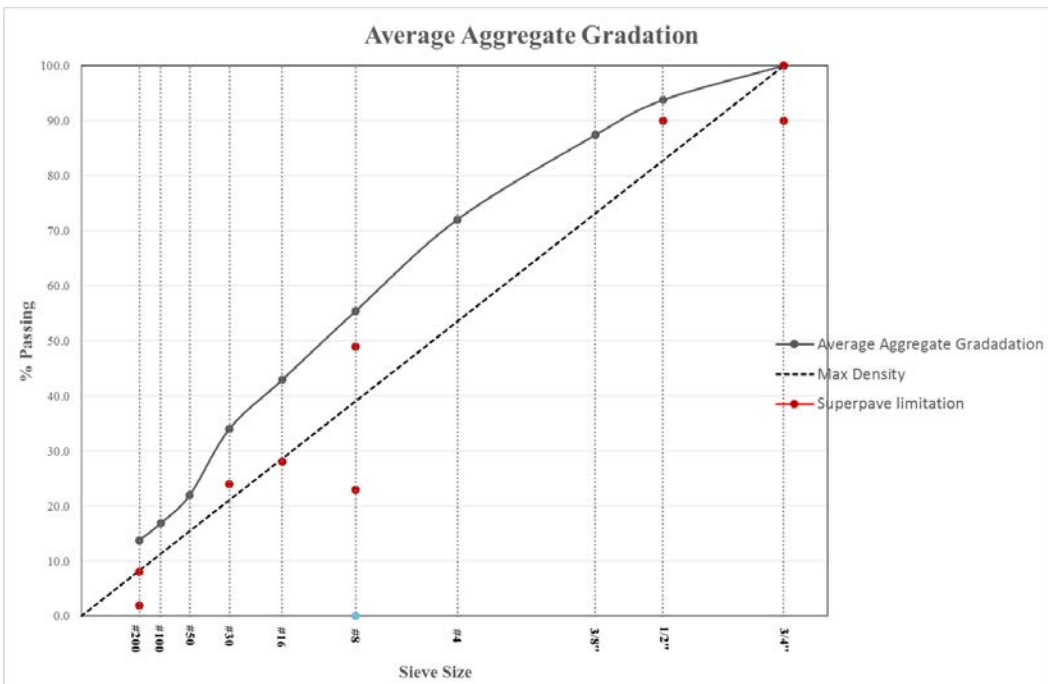


Figure 4-2 Aggregate gradation of the RAP stockpile

4.2.2.2 The Asphalt Content of RAP Stockpile

It is critical to know the asphalt content of the RAP material since, for this study, 100% RAP materials were used and the asphalt content of the RAP material would play an important role in determining the optimum asphalt content. Since the RAP material were stockpiled outside, they contained high moisture contents. First, to determine the moisture content, they were dried in the oven at 120 ˚C. To determine the asphalt content, the weights of samples before and after the burn-off test were measured. Table 6 shows the asphalt and moisture contents of five RAP samples.

Table 4-3 The asphalt content of RAP stockpile

No. sample	% of asphalt	% of moisture
1	4.82%	4.75%
2	4.91%	4.8%
3	4.70%	4.83%
4	4.98%	4.89%
5	4.88%	4.68%
Average	4.86%	4.79%

4.2.3 Aggregates and Gradation

The aggregates used in this study were a mix of natural sand, washed manufactured sand, 1/2 minus screen and 3/8 minus screen collected from Ames mine, Iowa, US. The nominal aggregate size was 12.5 mm. The detailed gradation of the asphalt mixtures in this study is shown in Table 4-4. The mixture designs for the asphalt mixture followed the Iowa Department of Transportation (Iowa DOT) specification.

Table 4-4 Aggregate gradation of virgin aggregate sources used in this study

Sieve Size		Sand	Man. Sand	1/2" to Dust	3/8 "
ID	mm				
3/4 in	19.0	100.0	100.0	100.0	100.0
1/2 in	12.5	100.0	100.0	83.0	100.0
3/8 in	9.5	100.0	100.0	64.0	97.0
#4	4.75	97.0	100.0	32.0	42.0
#8	2.36	86.0	73.0	20.0	10.0
#16	1.18	68.0	39.0	15.0	9.0
#30	0.6	40.0	19.0	12.0	8.0
#50	0.3	20.0	7.5	10.0	7.5
#100	0.15	0.6	4.3	8.0	7.0
#200	0.075	0.3	3.5	5.8	5.0

4.3 HMA Mix Design and Preparation

As mentioned before, since one of the objects of this research is increasing the percentage of RAP in WIR, three different mix designs with 30%, 70% and 100% RAP were proposed. Figure 4-3 and Table 4-5 represents the final aggregate gradation of 100% RAP, virgin aggregate, mixture with 30% RAP and mixture with 70% RAP.

Table 4-5 Final aggregate gradation of all mixtures with different percentage of RAP

Sieve Size		100% RAP	70% RAP	30 % RAP
ID	mm			
3/4 in	19.0	100	100	100
1/2 in	12.5	90	90	90
3/8 in	9.5	83	83	83
#4	4.75	61	62	61
#8	2.36	38	41	38
#16	1.18	23	27	23
#30	0.6	15	20	15
#50	0.3	8.5	13	8.5
#100	0.15	5.8	10	5.8
#200	0.075	4.6	8	4.6

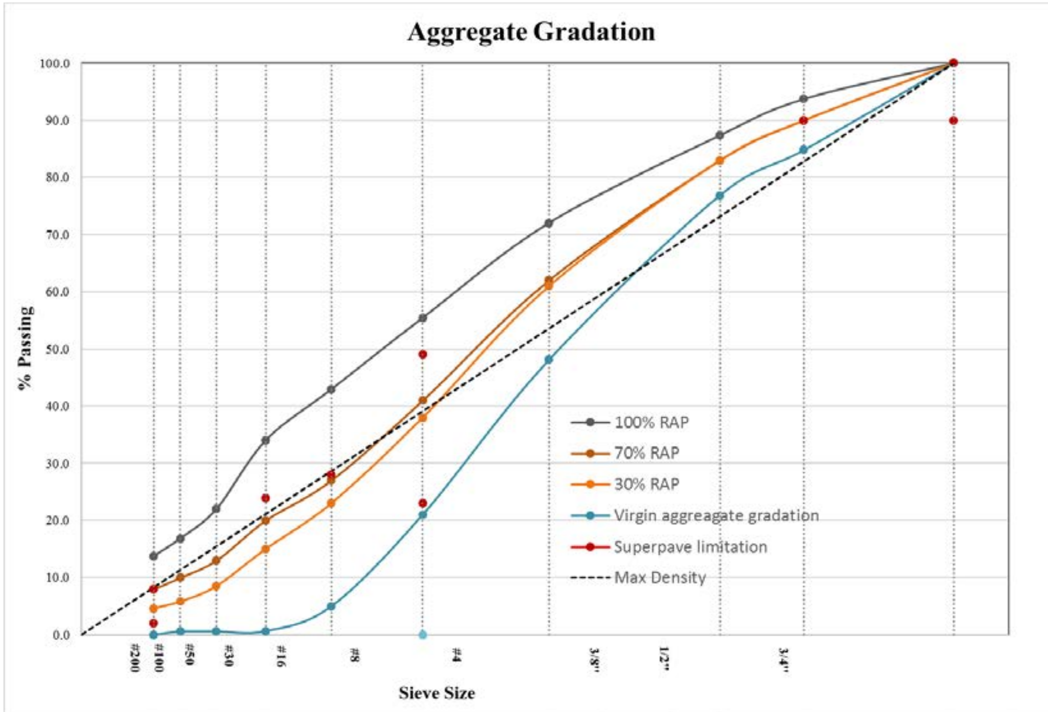


Figure 4-3 Aggregate gradation of 100, 70 and 30 % RAP mixtures

It was attempt to produce same aggregate gradation for all three mixture types. However, as can be seen in Table 4-5, for 70% and 30% mixture, the aggregate gradation were not exactly same, especially for fine aggregates. This difference was unavoidable because of the existing virgin aggregates gradation. However, it should be mentioned that the solver function of Excel was used and the percentage of each virgin aggregate stockpile was determined to minimize this difference. In other word, with the exciting virgin aggregate sources, these two gradations were the best that could be generated. For the 100% RAP mixture, it is obvious that the aggregate gradation was not only close to the two other mixtures gradation but also did not meet the Superpave criteria. However, to evaluate the potential of using 100% RAP materials in WIR, the aggregate gradation of RAP materials was changed. The aggregate gradation of one HIR mixture was considered as a target aggregate gradation and the 100% RAP mixture with same aggregate gradation was produced.

First, asphalt mix samples were fabricated with the initial trial binder content. 135 ºC and 125 ºC were used as the mixing and compaction temperatures as they were measured in **Chapter 3**. The guidance presented in appendix of AASHTO R35 was used to calculate an initial trial binder content for the extracted aggregate gradation. The air void and other volumetric measures were measured for samples made with initial asphalt binder content percent. The 68 gyration number for a low level of traffic level was adopted for compaction. According to the volumetric results of samples with initial asphalt binder content, this initial asphalt content was corrected. For each mixture type, a total of five specimens were prepared, which include one sample with corrected initial asphalt binder, two samples with initial asphalt binder ±0.5%, and two samples with initial asphalt binder ±1%. The optimum percentage of asphalt binder were calculated and the volumetric properties of the mixture with optimum asphalt content is presented in Table 4-6 to for all three mixtures. As can be seen in this Table, it was increasingly difficult to meet all the Superpave mix design requirements as more RAP materials are added in the asphalt mixtures

Table 4-6 Volumetric Mix Design Results

Volumetric properties	27.6% RAP Mixture	70.0% RAP Mixture	100% RAP Mixture	Requirements
Gsb RAP	2.619	2.619		
Gsb Virgin Aggregate	2.625	2.625	--	
Gsb Total	2.623	2.622		
Optimum binder content (%)	5.56	3.93	4.86	
Gmm @ opt. binder	2.430	2.471	2.412	
Gmb @ opt. binder	2.333	2.372	2.4576	
VMA	16.00	13.12	12.32	Min. 14
VFA	75	70	84.87	70-80
Film Thickness	12.63	6.39	14.31	8-13
Air Void (%)	4.00	4.00	1.76	4.00

4.4 Analysis of the Compactability of Field WIR Mixtures

In order to analyze the compactability of RAP mixture, RAP mixture were compacted using a gyratory compactor. Three random samples of RAP materials each weighing 4,800 grams were compacted up to 150 gyrations at a typical warm mix temperature of 125 ºC. The heights versus the number of gyrations of three specimens are summarized in Table 4-7 to Table 4-9 and plotted in Figure 4-4.

Table 4-7 Number of gyrations vs height of sample one

N	H (mm)	N	H (mm)	N	H (mm)	N	H (mm)	N	H (mm)
1	130.9	31	116.9	61	114.9	91	114.3	121	114.0
2	129.0	32	116.8	62	114.9	92	114.2	122	113.9
3	127.2	33	116.7	63	114.9	93	114.2	123	113.9
4	126.0	34	116.5	64	114.8	94	114.2	124	113.9
5	124.9	35	116.5	65	114.8	95	114.2	125	113.9
6	124.0	36	116.4	66	114.8	96	114.2	126	113.9
7	123.3	37	116.3	67	114.7	97	114.2	127	113.9
8	122.6	38	116.2	68	114.7	98	114.2	128	113.9
9	122.1	39	116.1	69	114.7	99	114.2	129	113.9
10	121.6	40	116.0	70	114.7	100	114.1	130	113.9
11	121.2	41	116.0	71	114.6	101	114.1	131	113.9
12	120.8	42	115.9	72	114.6	102	114.1	132	113.9
13	120.4	43	115.8	73	114.6	103	114.1	133	113.9
14	120.1	44	115.7	74	114.6	104	114.1	134	113.9
15	119.8	45	115.7	75	114.6	105	114.1	135	113.9
16	119.5	46	115.6	76	114.5	106	114.1	136	113.9
17	119.2	47	115.6	77	114.5	107	114.1	137	113.8
18	119.0	48	115.5	78	114.5	108	114.1	138	113.8
19	118.8	49	115.4	79	114.5	109	114.0	139	113.8
20	118.6	50	115.4	80	114.4	110	114.0	140	113.8
21	118.4	51	115.4	81	114.4	111	114.0	141	113.8
22	118.2	52	115.3	82	114.4	112	114.0	142	113.8
23	118.0	53	115.3	83	114.4	113	114.0	143	113.8
24	117.8	54	115.2	84	114.4	114	114.0	144	113.8
25	117.7	55	115.2	85	114.4	115	114.0	145	113.8
26	117.5	56	115.1	86	114.3	116	114.0	146	113.8
27	117.4	57	115.1	87	114.3	117	114.0	147	113.8
28	117.2	58	115.0	88	114.3	118	114.0	148	113.8
29	117.1	59	115.0	89	114.3	119	114.0	149	113.8
30	117.0	60	115.0	90	114.3	120	114.0	150	113.8

Table 4-8 Number of gyrations vs height of sample two

N	H (mm)	N	H (mm)	N	H (mm)	N	H (mm)	N	H (mm)
1	131.1	31	117.0	61	114.9	91	114.3	121	113.9
2	129.2	32	116.9	62	114.9	92	114.2	122	113.8
3	127.5	33	116.8	63	114.9	93	114.2	123	113.8
4	126.2	34	116.6	64	114.8	94	114.2	124	113.8
5	125.1	35	116.6	65	114.8	95	114.2	125	113.8
6	124.2	36	116.5	66	114.8	96	114.2	126	113.8
7	123.5	37	116.4	67	114.7	97	114.2	127	113.8
8	122.8	38	116.3	68	114.7	98	114.1	128	113.8
9	122.3	39	116.2	69	114.7	99	114.1	129	113.8
10	121.8	40	116.1	70	114.7	100	114.1	130	113.8
11	121.3	41	116.0	71	114.6	101	114.1	131	113.8
12	121.0	42	116.0	72	114.6	102	114.1	132	113.8
13	120.6	43	115.9	73	114.6	103	114.1	133	113.8
14	120.2	44	115.8	74	114.6	104	114.1	134	113.8
15	119.9	45	115.7	75	114.6	105	114.0	135	113.8
16	119.6	46	115.7	76	114.5	106	114.0	136	113.8
17	119.4	47	115.6	77	114.5	107	114.0	137	113.7
18	119.1	48	115.5	78	114.5	108	114.0	138	113.7
19	118.9	49	115.5	79	114.5	109	114.0	139	113.7
20	118.7	50	115.4	80	114.4	110	114.0	140	113.7
21	118.5	51	115.4	81	114.4	111	114.0	141	113.7
22	118.3	52	115.3	82	114.4	112	114.0	142	113.7
23	118.1	53	115.3	83	114.4	113	113.9	143	113.7
24	117.9	54	115.2	84	114.4	114	113.9	144	113.7
25	117.8	55	115.2	85	114.4	115	113.9	145	113.7
26	117.6	56	115.1	86	114.3	116	113.9	146	113.7
27	117.5	57	115.1	87	114.3	117	113.9	147	113.7
28	117.3	58	115.0	88	114.3	118	113.9	148	113.7
29	117.2	59	115.0	89	114.3	119	113.9	149	113.7
30	117.1	60	115.0	90	114.3	120	113.9	150	113.7

Table 4-9 Number of gyrations vs height of sample three

N	H (mm)	N	H (mm)	N	H (mm)	N	H (mm)	N	H (mm)
1	131.2	31	117.0	61	114.9	91	114.2	121	113.8
2	129.3	32	116.9	62	114.9	92	114.1	122	113.7
3	127.7	33	116.8	63	114.9	93	114.1	123	113.7
4	126.3	34	116.7	64	114.8	94	114.1	124	113.7
5	125.3	35	116.6	65	114.8	95	114.1	125	113.7
6	124.3	36	116.5	66	114.7	96	114.1	126	113.7
7	123.6	37	116.4	67	114.7	97	114.1	127	113.7
8	123.0	38	116.3	68	114.7	98	114.0	128	113.7
9	122.4	39	116.2	69	114.7	99	114.0	129	113.7
10	121.9	40	116.1	70	114.6	100	114.0	130	113.7
11	121.4	41	116.0	71	114.6	101	114.0	131	113.7
12	121.1	42	116.0	72	114.6	102	114.0	132	113.7
13	120.7	43	115.9	73	114.5	103	114.0	133	113.6
14	120.3	44	115.8	74	114.5	104	114.0	134	113.6
15	120.0	45	115.7	75	114.5	105	113.9	135	113.6
16	119.7	46	115.7	76	114.5	106	113.9	136	113.6
17	119.5	47	115.6	77	114.4	107	113.9	137	113.6
18	119.2	48	115.5	78	114.4	108	113.9	138	113.6
19	119.0	49	115.5	79	114.4	109	113.9	139	113.6
20	118.8	50	115.4	80	114.4	110	113.9	140	113.6
21	118.5	51	115.4	81	114.3	111	113.9	141	113.6
22	118.4	52	115.3	82	114.3	112	113.9	142	113.6
23	118.2	53	115.3	83	114.3	113	113.8	143	113.6
24	118.0	54	115.2	84	114.3	114	113.8	144	113.6
25	117.8	55	115.2	85	114.3	115	113.8	145	113.6
26	117.7	56	115.1	86	114.2	116	113.8	146	113.6
27	117.5	57	115.1	87	114.2	117	113.8	147	113.5
28	117.4	58	115.0	88	114.2	118	113.8	148	113.5
29	117.3	59	115.0	89	114.2	119	113.8	149	113.5
30	117.1	60	115.0	90	114.2	120	113.8	150	113.5

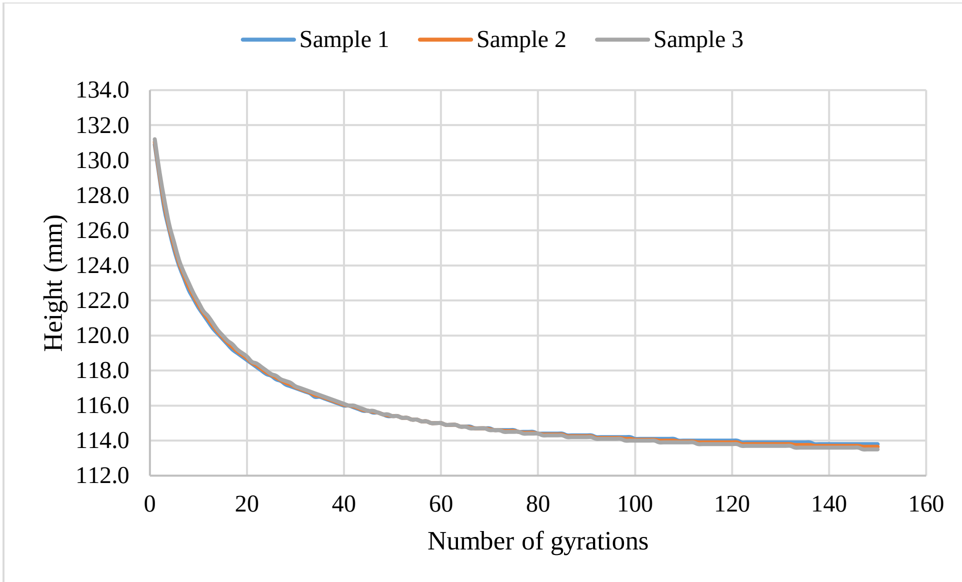


Figure 4-4 Number of gyrations vs height for all samples

As can be seen in Figure 4-4, the compatibilities of RAP materials were similar among all three samples. A significant densification occurred up to the first twenty gyrations and the densification gradually decreased up to eighty gyrations. From 80 to 150 gyrations, very little densification occurred.

Theoretical maximum specific gravity (G_{mm}) and bulk specific gravity (G_{mb}) of samples were calculated according to the AASHTO T 209 and AASHTO T 166, respectively. Table 4-10 summarizes G_{mm} and G_{mb} for three RAP samples.

Table 4-10 Bulk and maximum specific gravity of samples

Sample	**1**	**2**	**3**
G_{mb}	2.419	2.415	2.424
G_{mm}	2.46	2.46	2.46

The % G_{mm} at any number of gyrations (N_x) is calculated by:

$$\%\, G_{mm}@\, N_x = \%\, G_{mm}@\, N_{max} \frac{H_{final}}{H_x}$$

Which H_{final} is the final height of sample and H_x is the height at the x number of gyrations. Figure 4-5 shows the compaction level or percent of G_{mm} for all three samples.

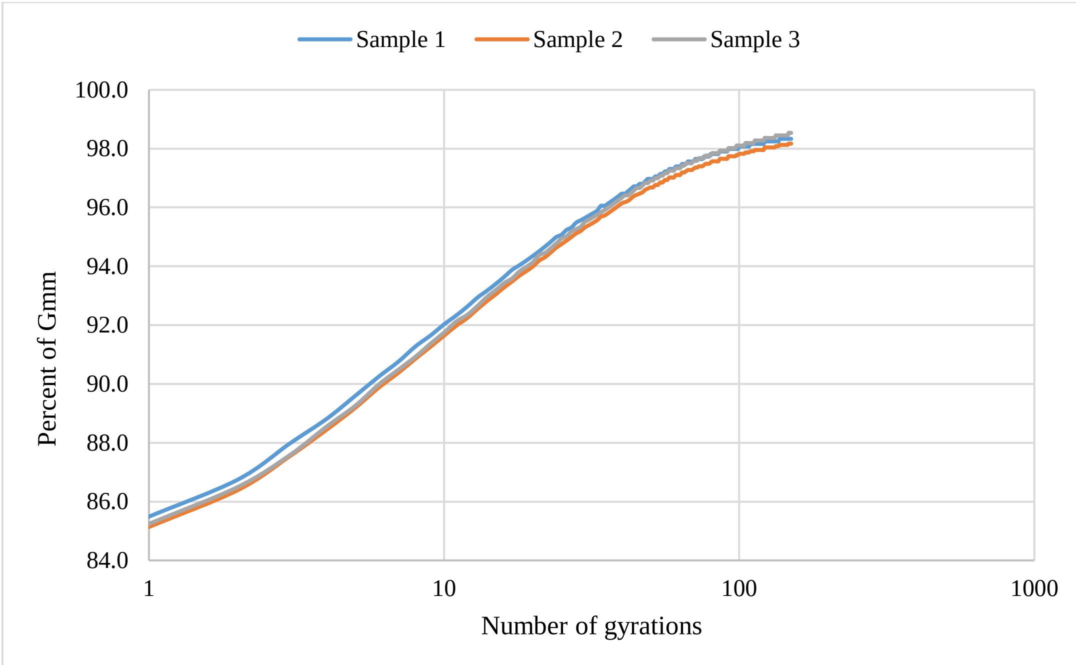


Figure 4-5 Percent of G_{mm} for each number of gyrations for all samples

As can be seen in this figure, after 150 gyrations, around 98% of maximum specific gravity was achieved in all three samples. It should be noted that according to the previous version of Superpave specifications, for high level of traffic, the percent of Gmm at 110 gyrations should be less than 98%. Since the compaction temperature for our sample was 125 ºC (20 ºC less than normal sample), it can be concluded that the compactability of RAP material is acceptable. The percent of air void in sample is calculate by:

$$\%\,A\ @\ N_x = 100 - \ \%\,G_{mm} @\ N_x$$

Figure 4-6 shows the percent of air void in samples. As expected, the air voids decreased as the number of gyrations increased. The lowest percent of air voids is around 2% for all samples that is acceptable according to the Superpave design requirements.

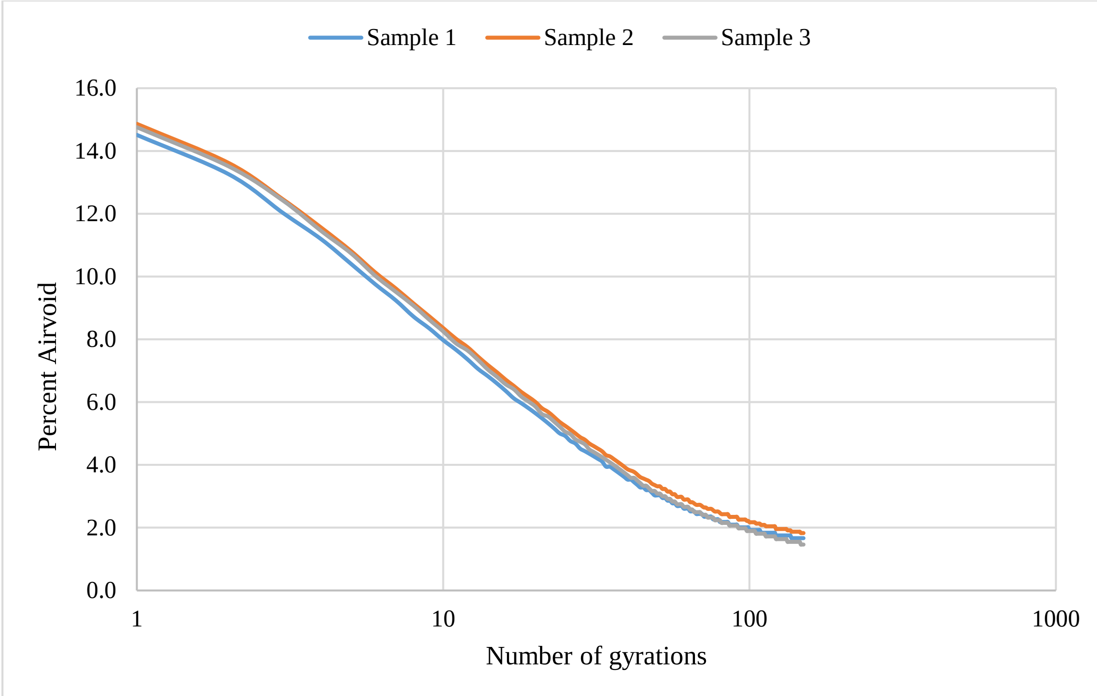


Figure 4-6 Percent of Air void in samples

4.5 Test Plan for Performance Evaluation of Modified WIR Mixtures

As mentioned earlier, the mechanical performances of HMA mixture modified by WIR additives were evaluated by related performance tests. The related performance tests were: Hamburg Wheel Tracking (HWT) Test for rutting and moisture susceptibility evaluation; Disk-Shaped Compact Tension (DCT) Test for evaluation of thermal fracture properties of asphalt mixtures. Modified mixtures were prepared with the optimum dosage of additives, which was calculated with rheological tests in **Chapter 3**. For each of the three mixtures, all the performance tests were run and evaluated the maximum percentage of RAP that can be added to mixtures. The detailed test plan is listed as Table 4-11.

Table 4-11 Mechanical performance test plan for modified WMA mixtures

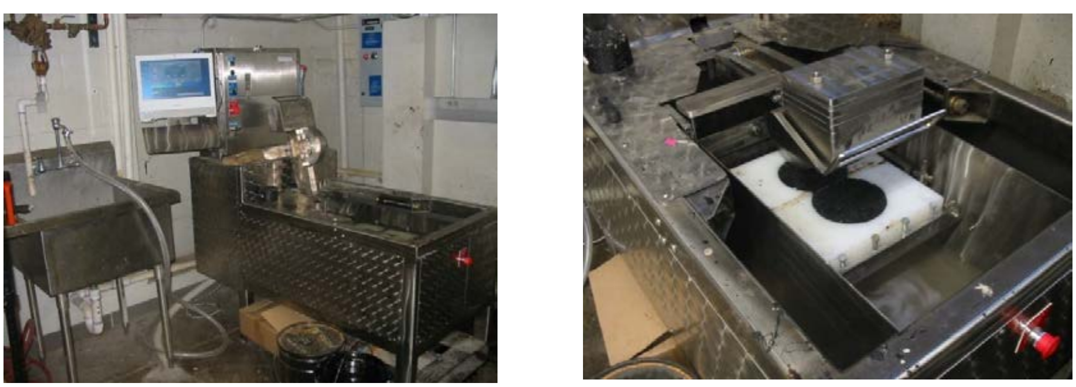
	30% RAP + & 70% Virgin		70% RAP + & 30% Virgin		100% RAP	
Sample ID	**HWT**	**DCT**	**HWT**	**DCT**	**HWT**	**DCT**
A1[**]	XXX	XXXX	XXX	XXXX	XXX	XXXX
A2	XXX	XXXX	XXX	XXXX	XXX	XXXX
Control	XXX	XXXX	XXX	XXXX	XXX	XXXX
Virgin	XXX	XXXX	XXX	XXXX	XXX	XXXX
Total	12	15	12	15	12	15

[*]note: X means one replicate

[**]note: A1 means mixture with optimum dosage of additive one

4.6 Rutting and Moisture Susceptibility Test of WIR Specimens

To evaluate the rutting potential and moisture susceptibility, the Hamburg Wheel Track test was performed on all three mixture types. The Hamburg Wheel Tracking device applies a constant load of 685 N through a steel wheel with a diameter of 203.5 mm and a width of 47.0 mm. The tests are run in a water bath that is heated to 50 °C after the test specimens are conditioned for 30 minutes. Figure 4-7 shows the Hamburg Wheel Tracking device and specimens ready for testing. The test is completed when the wheel has passed over the specimens 20,000 times for 6.5 hours or when the rut depth exceeds 20 mm.



(a) (b)

Figure 4-7 Hamburg Wheel Tracking Device (left) and Specimens Ready for testing (right)

The Hamburg Wheel Tracking Device measures rut depth throughout the test and reports four properties: 1) post-compaction consolidation, 2) creep slope, 3) stripping inflection point, and 4) stripping slope. The post-compaction consolidation occurs at around 1,000 wheel passes that is normally caused by the densification of the mixture. The creep slope is used to measure the rutting susceptibility of the mixture that measures the permanent deformation caused by the wheel passes. The stripping inflection point and the stripping slope are used to measure damaged caused by moisture. A mixture with a stripping inflection point less than 10,000 passes should be considered as moisture susceptible.

The specimens had a target air void content of 7.0 ± 2.0 %. Specimens were compacted with a height of 60 mm to fit the mold for the Hamburg Wheel Tracking device. 7.5 mm of material was removed from one side of the specimen so that they fit together in the specimen tray. Figure 4-8 shows the dimensions of the specimen and the mold.

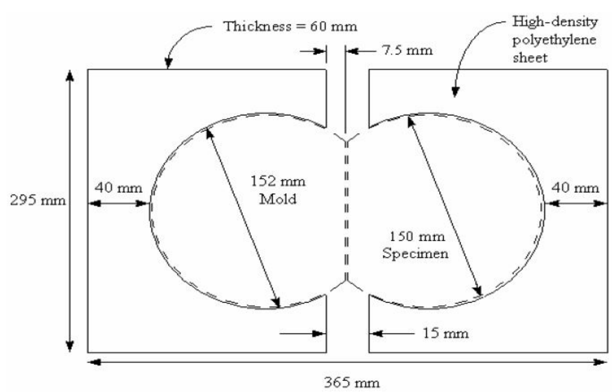


Figure 4-8 Dimensions of the Specimen and the Mold

The HWT Test results for each mixture group are summarized in Table 4-12 through Table 4-14 and plot in Figure 4-9 through Figure 4-11.

Table 4-12 Hamburg Wheel Test Results for Mixtures Including 30% RAP

Mix Type	Test ID	Air Voids, %	Total Number of Passes	Creep Slop	Stripping slope	SIP	Max. Rut Depth, mm
30-A1	30-A1-1	6.48%	20000	0.147	1.890	20000	11.0
	30-A1-2	6.95%	20000	0.122	2.051	20000	8.6
	30-A1-3	6.70%	20000	0.097	0.172	20000	8.3
	Average	6.71%	20000	0.122	1.371	>20000	9.3
30-A2	30-A2-1	7.13%	20000	0.090	0.156	20000	7.5
	30-A2-2	6.84%	20000	0.048	0.077	20000	9.5
	30-A2-3	6.78%	20000	0.194	1.776	9791	15.6
	Average	6.92%	20000	0.111	0.670	>20000	10.8
30-C	30-C-1	6.54%	20000	0.062	1.632	8436	5.5
	30-C-2	7.26%	20000	0.137	0.369	15104	5.6
	30-C-3	6.94%	20000	0.188	0.151	18760	4.1
	Average	6.91%	20000	0.129	0.717	14100.0	5.0
30-V	30-V-1	6.53%	20000	0.157	1.677	10856	19.6
	30-V-2	7.42%	20000	0.265	1.857	10373	20.0
	30-V-3	7.34%	20000	0.027	1.577	11257	19.7
	Average	7.10%	20000	0.150	1.704	10828.7	19.8

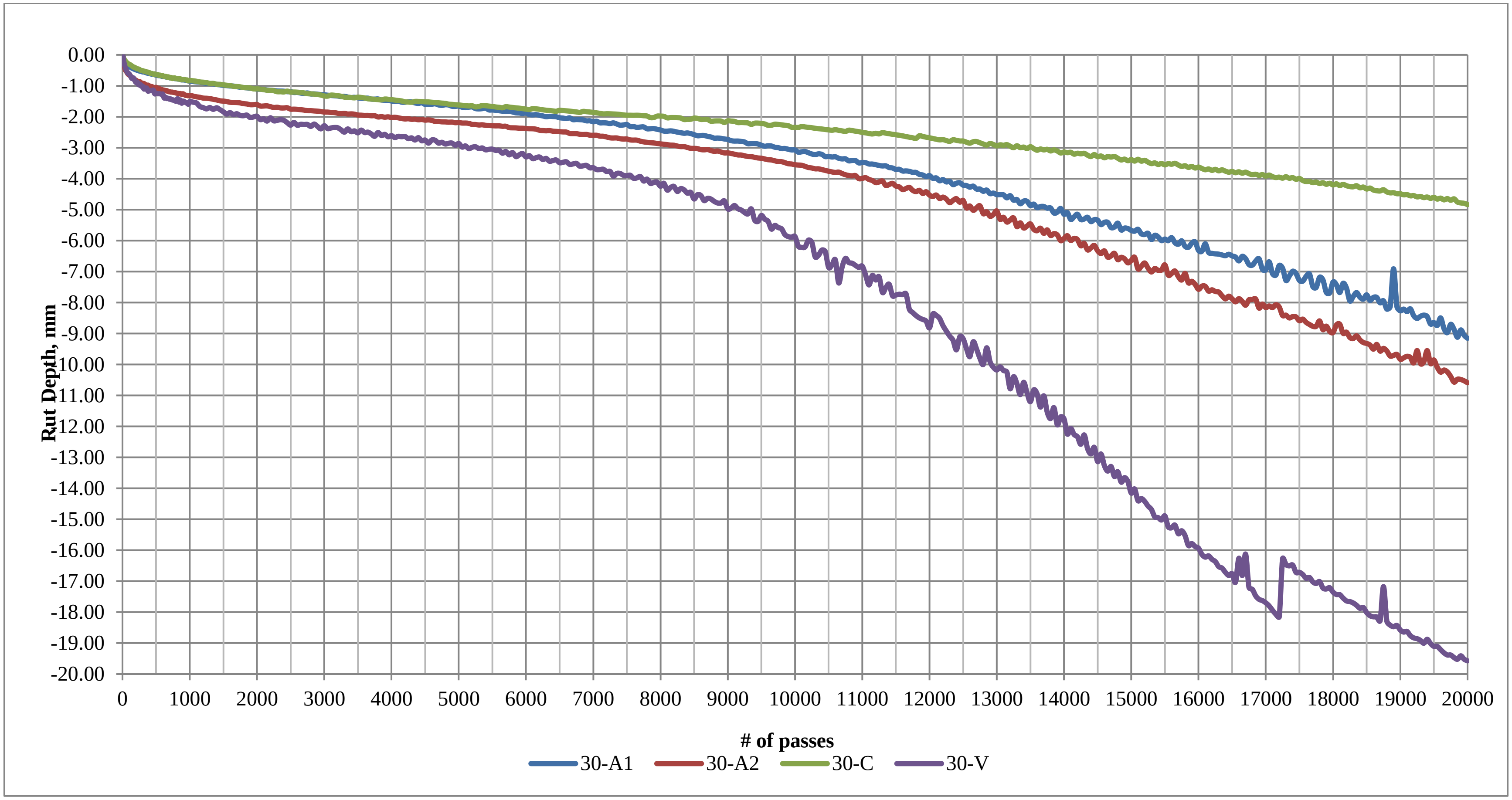


Figure 4-9 Hamburg Wheel Test Results for Mixtures Including 30% RAP

Table 4-13 Hamburg Wheel Test Results for Mixtures Including 70% RAP

Mix Type	Test ID	Air Voids, %	Total Number of Passes	Creep Slop	Stripping slope	SIP	Max. Rut Depth, mm
60-A1	60-A1-1	6.58%	20000	0.150	1.780	20000	2.2
	60-A1-2	6.75%	20000	0.128	2.140	20000	1.5
	60-A1-3	6.80%	20000	0.011	1.900	20000	2.4
	Average	6.71%	20000	0.1	1.9	>20000	2.0
60-A2	60-A2-1	7.03%	20000	0.850	0.168	20000	1.8
	60-A2-2	6.73%	20000	0.074	0.089	20000	3.4
	60-A2-3	6.98%	20000	0.184	0.045	20000	1.5
	Average	6.91%	20000	0.4	0.1	>20000	2.2
60-C	60-C-1	6.64%	20000	0.005	0.017	20000	0.8
	60-C-2	6.86%	20000	0.009	0.013	20000	0.8
	60-C-3	7.1%	20000	0.009	0.011	20000	0.7
	Average	6.86%	20000	0.0	0.0	>20000	0.8
60-V	60-V-1	6.79%	20000	0.036	0.059	20000	2.9
	60-V-2	7.35%	20000	0.056	0.085	20000	3.2
	60-V-3	6.76%	20000	0.047	0.103	20000	3.2
	Average	6.97%	20000	0.0	0.1	>20000	3.1

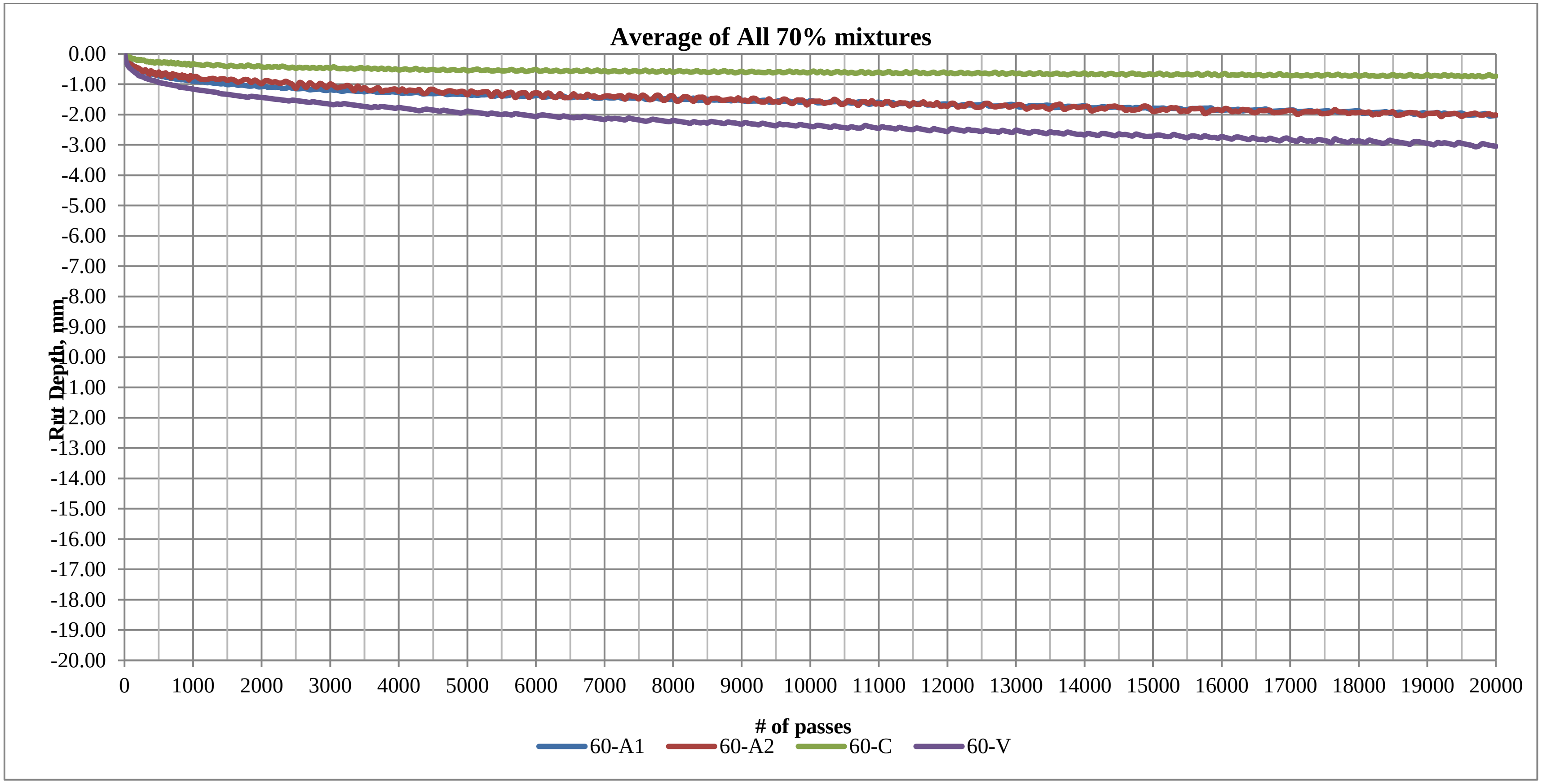


Figure 4-10 Hamburg Wheel Test Results for Mixtures Including 70% RAP

Table 4-14 Hamburg Wheel Test Results for Mixtures Including 100% RAP

Mix Type	Test ID	Air Voids %	Total Number of Passes	Creep Slop	Stripping slope	SIP	Max. Rut Depth, mm
100-A1	100-A1-1	6.64%	20000	0.052	0.085	20000	2.8
	100-A1-2	7.23%	20000	0.050	0.082	20000	2.8
	100-A1-3	6.56%	20000	0.110	0.182	20000	3.3
	Average	6.81%	20000	0.070	0.116	>20000	2.9
100-A2	100-A2-1	6.83%	20000	0.014	0.045	20000	2.0
	100-A2-2	7.14%	20000	0.022	0.040	20000	1.8
	100-A2-3	6.58%	20000	0.052	0.257	20000	3.4
	Average	6.85%	20000	0.029	0.114	>20000	2.4
100-C	100-C-1	7.18%	20000	0.027	0.048	20000	1.9
	100-C-2	7.27%	20000	0.033	0.010	20000	2.5
	100-C-3	6.75%	20000	0.018	0.085	20000	3.3
	Average	7.06%	20000	0.026	0.048	>20000	2.5
100-V	100-V-1	6.73%	14600	0.367	2.373	8962	20.1
	100-V-2	7.22%	10227	0.493	2.321	5126	20.0
	100-V-3	7.25%	20000	0.314	0.708	12409	16.6
	Average	7.06%	14942	0.392	1.801	8832.3	18.9

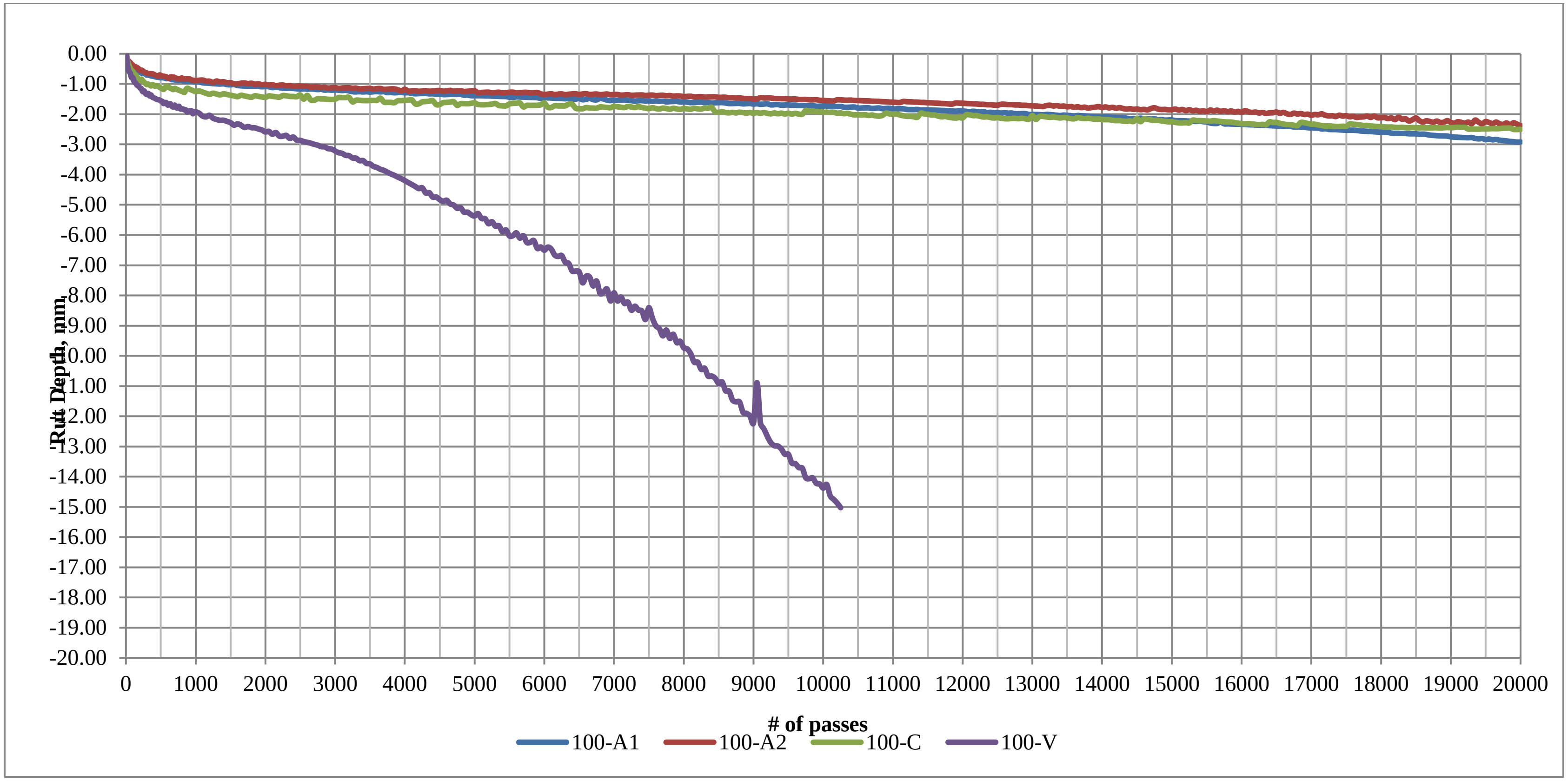


Figure 4-11 Hamburg Wheel Test Results for Mixtures Including 100% RAP

4.6.1 Mixtures with 30% RAP

As can be seen in Table 4-12 and Figure 4-9, all specimens successfully passed the test since the rut depth of all specimens was less than 20mm. In addition, all specimens did not have a moisture susceptibility problem because the SIP of all specimens were higher than 10,000. However, the virgin mixture and mixture without additive were very close to the moisture susceptibility point, especially for the virgin mixture. Adding additive 1 and 2 improved the resistance of mixture against moisture susceptibility. None of the mixtures had any problems for rutting because the RAP material made the mixture harder and enhance its rutting performances. Mixture with virgin aggregate had the higher rutting depth followed by mixture with additive 2 and 1 respectively. This fact showed that additives made the mixture soft, but it did not have significant effect on rutting.

4.6.2 Mixtures with 70% RAP

All specimens successfully passed the test with the average maximum rut depths less than 3.0 mm as it is illustrated in Table 4-13 and Figure 4-10. For all specimens, the max rutting depth was less than 30% RAP mixtures since there was more RAP material in 70% mixtures that made them harder than 30% mixture. The average SIP for all specimens was more than 20,000 passes which proved none of the specimen had a moisture susceptibility. It should be noted that the SIP values for most specimens were significantly higher than SIP of 30% mixture when the amount of RAP was increased from 30% to 70%.

4.6.3 Mixtures with 100% RAP

Table 4-14 and Figure 4-11 show that all specimens, except the virgin mixture successfully passed the test with the average maximum rut depths of 2.9 mm and less. The virgin mixture did not pass the test because its aggregate gradation which was almost same as the control mixture aggregate gradation. Both aggregate gradations did not meet the Superpave aggregate gradation criteria as mentioned before. However, control (100% RAP mixture) could pass the test because of the hardness of RAP material. The average SIP of all specimens except the virgin mixtures was greater than 20,000 passes.

4.7 Low-Temperature Performance Test of WIR Specimens

The Disc-Shaped Compacted Tension (DCT) test was conducted to evaluate the low-temperature cracking properties of mixtures with and without the additives.

4.7.1 Sample Preparation

Four samples were prepared for each mixture type since DCT is very sensitive test and result are rarely constant. In addition, during the sample preparation and test procedure, it is highly possible that a sample is broken. The first step of DCT specimen preparation is sawing across a diameter of the specimen using a water-cooled masonry saw. To ensure the consistency in air voids and asphalt content, the middle portion of the specimen was used after cutting both ends of the specimen using a saw. To produce a flat smooth face for attaching the gage points, the sample was smoothly sawn. A marking template was used to indicate the location of the 25 mm diameter loading holes and the notch. A core drill was used to drill the 25 mm (0.984 inches) holes, and a wet band saw is used to cut the notch. A completed DCT specimen is shown in Figure 4-12. The specimens were then allowed to completely dry, either using air or a desiccant, and the gage points are then attached. This specimen geometry was found to be satisfactory for asphalt mixtures with nominal maximum aggregate sizes ranging from 4.75 mm (0.187 inches) to 19 mm (0.748 inches). The specimen geometry can be seen in Figure 4-13.

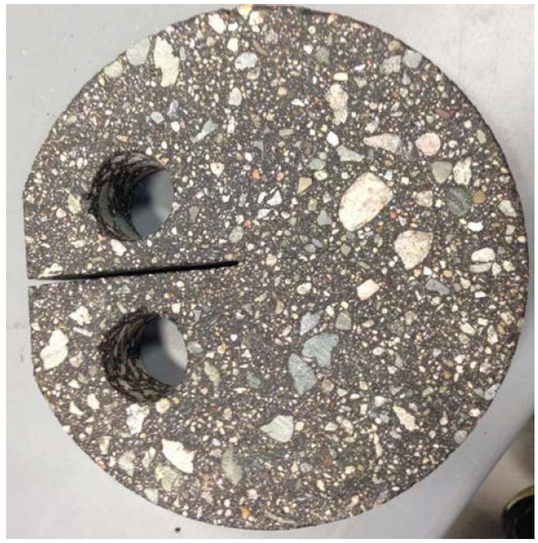


Figure 4-12 Completed DCT specimen

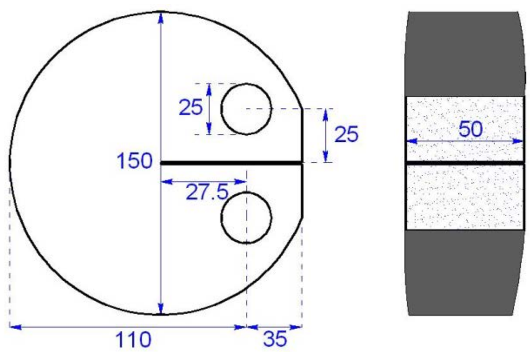


Figure 4-13 DCT specimen geometry (mm).

4.7.2 DCT Test Temperature

A standard test temperature for DCT specimens is to be 10°C warmer than the PG low temperature grade. A temperature for DCT testing adopted in this study is -12 °C since the PG grade of asphalt binder was used is PG 64-22.

4.7.3 DCT Test Operation

The DCT test is controlled by a constant crack mouth opening displacement (CMOD) rate of 0.017 mm/s (approximately 1.0 mm/min). This loading rate is fast enough to essentially eliminate the majority of creep behaviors of the mixture during testing. Data essential to the calculation of fracture energy are load and CMOD. Loads are plotted against CMOD, with the area under the curve being fracture work. When normalized for specimen thickness and initial ligament length, the area under the load vs. CMOD curve is known as fracture energy. Specimen thickness and initial ligament length were measured prior to DCT testing for adjustment of the test result. A specimen before and after testing with these measurements can be seen in Figure 4-14, with L indicating ligament length (straight line measured from end of notch where cracking is initiated to the edge of specimen) and B indicating thickness of specimen. Fracture energy is the energy required to create a unit surface fracture of the asphalt mixture. After testing, the specimen was pulled apart to view the path of crack propagation.

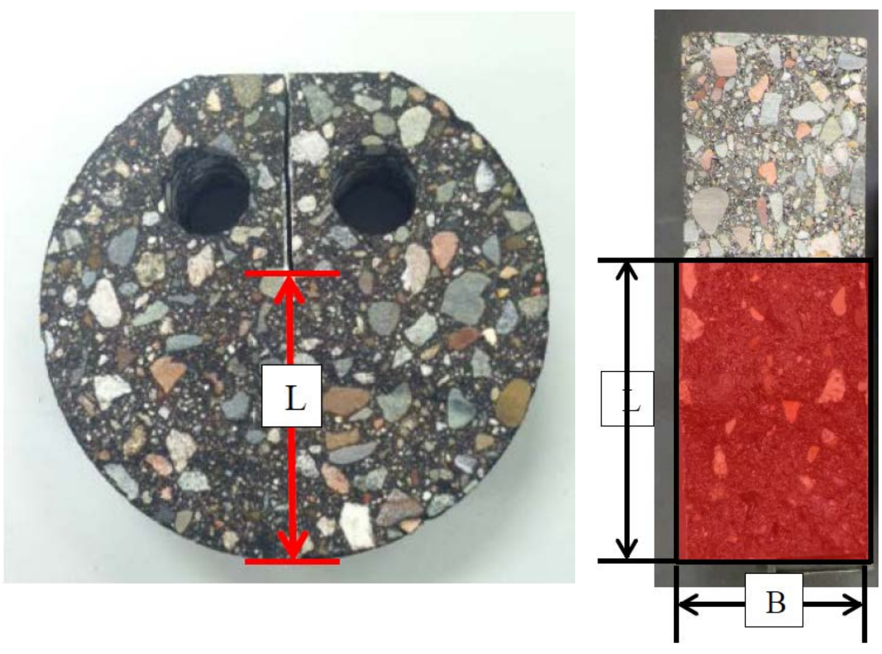


Figure 4-14 DCT specimen ligament length (L) and width (B)

4.7.4 Test Result

The fracture energy of all specimens are summarized in Table 4-15 to Table 4-17. The average fracture energy of three specimens for each mixture are plotted in Figure 4-15 to Figure 4-17.

Table 4-15 Fracture energy (J/m) 30% RAP mixture

	30% RAP Mixture Fracture Energy (J/m)				
Sample type	**Specimen 1**	**Specimen 2**	**Specimen 3**	**Specimen 4**	**Average**
30-A1	307	345	327	387	342
30-A2	307	247	267	307	282
30-C	336	--	321	362	339
30-V	377	--	372	414	387

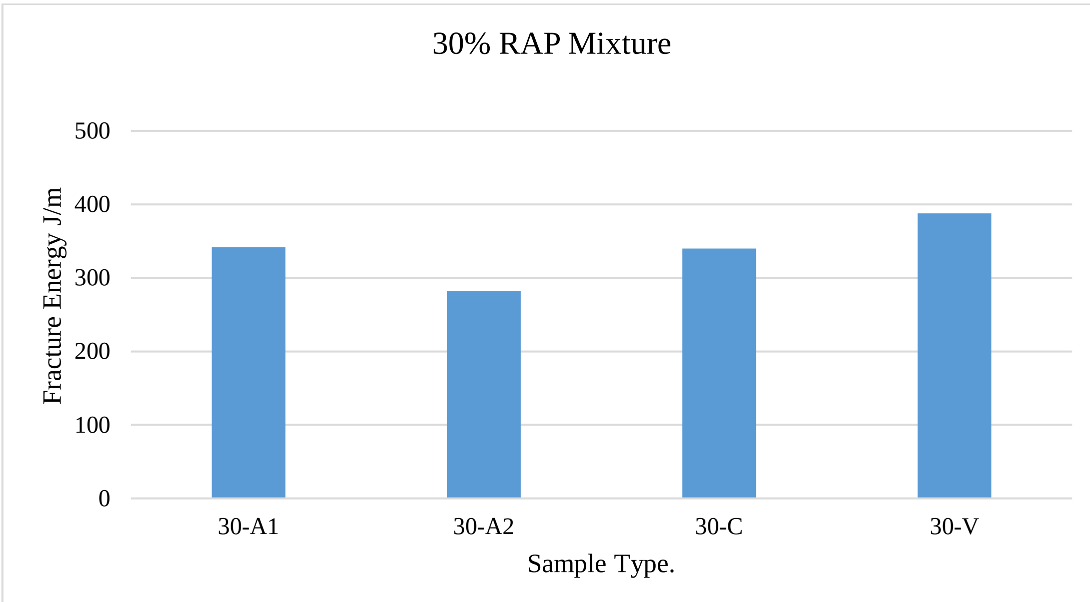


Figure 4-15 Average fracture energy of 30% RAP mixture

Table 4-16 Fracture energy (J/m) 70% RAP mixture

	70% RAP Mixture Fracture Energy (J/m)				
Sample type	**Specimen 1**	**Specimen 2**	**Specimen 3**	**Specimen 4**	**Average**
60-A1	311	202	291	--	268
60-A2	228	262	--	223	237
60-C	256	271	--	--	263
60-V	261	201	258	--	240

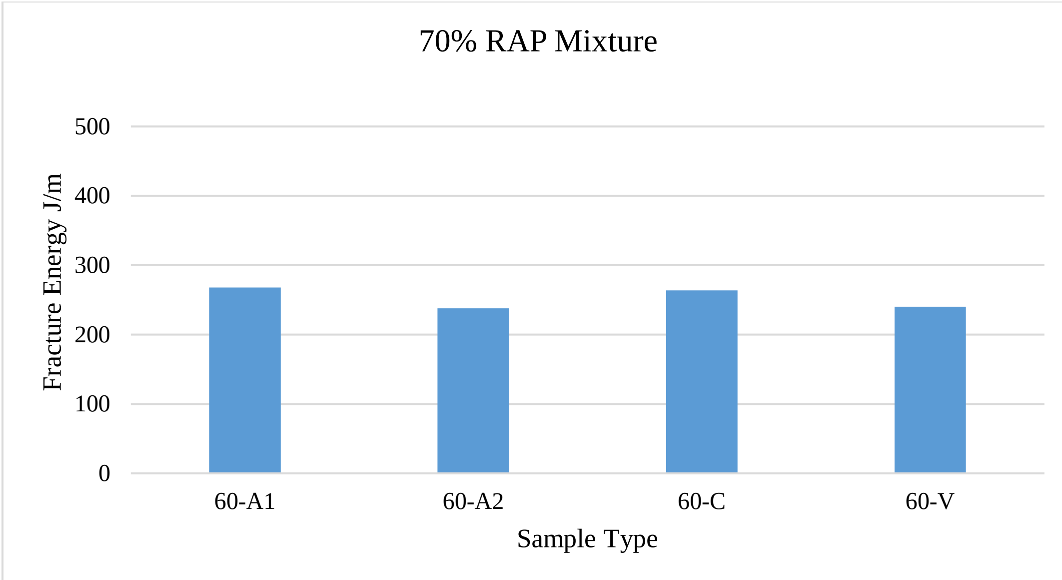


Figure 4-16 Average fracture energy of 70% RAP mixture

Table 4-17 Fracture energy (J/m) 100% RAP mixture

	100% RAP Mixture Fracture Energy (J/m)				
Sample type	**Specimen 1**	**Specimen 2**	**Specimen 3**	**Specimen 4**	**Average**
100-A1	320	284	--	283	296
100-A2	320	259	254	265	274
100-C	222	247	257	213	235
100-V	306	352	230	--	296

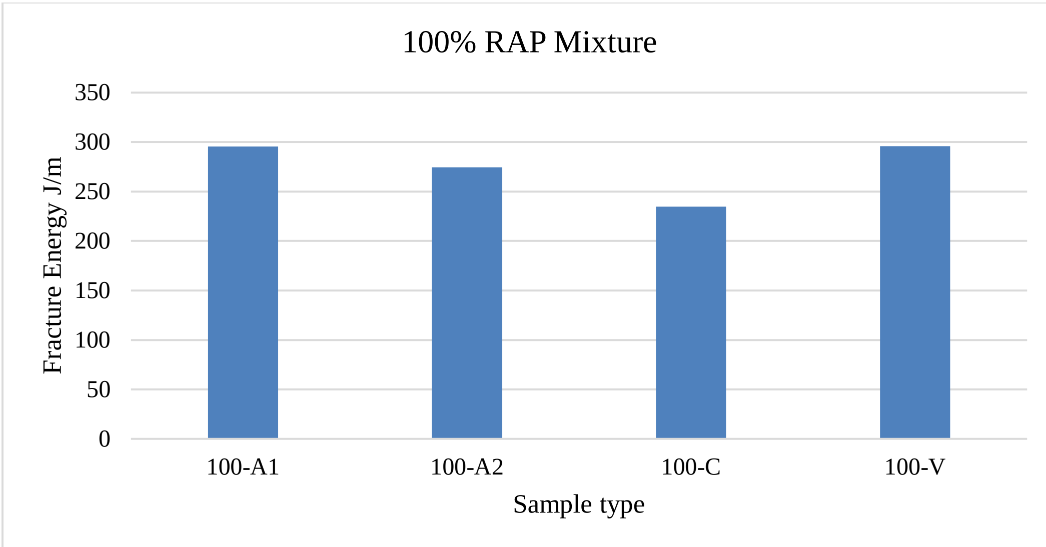


Figure 4-17 Average fracture energy of 100% RAP mixture

The plot of average Loads vs. CMOD for each group are shown in Figure 4-18 to Figure 4-20.

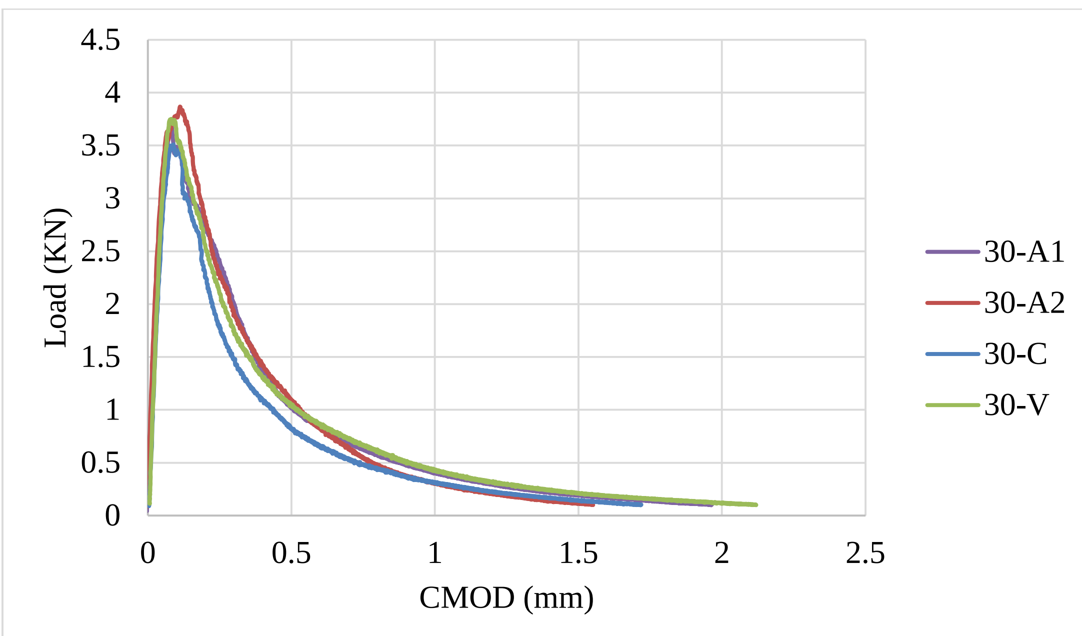


Figure 4-18 Load vs CMOD of 30% RAP specimens

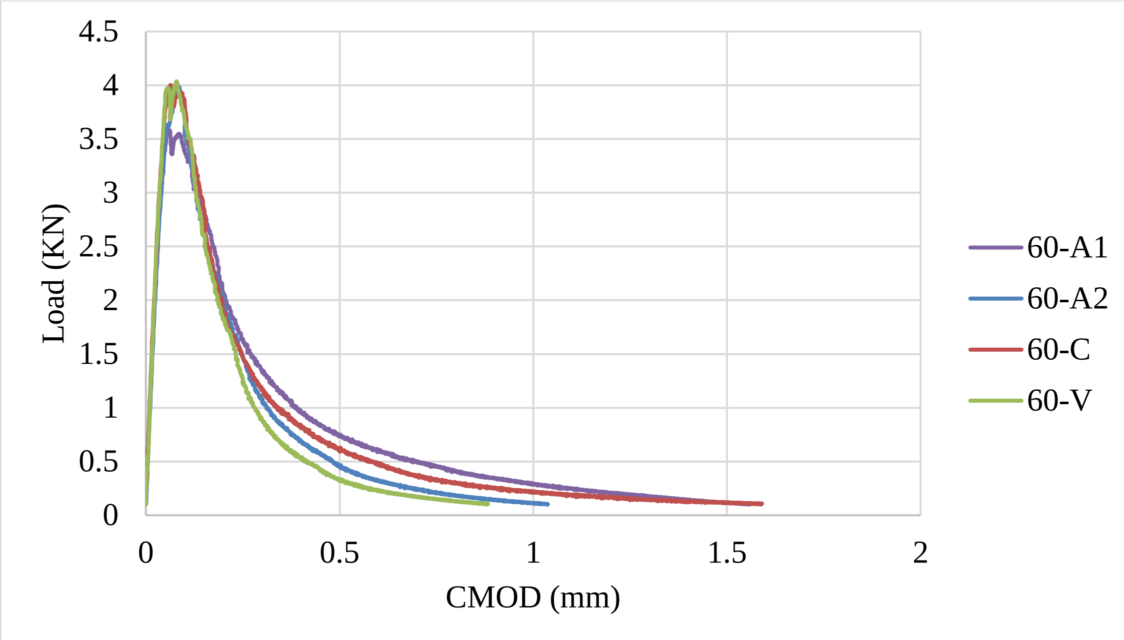


Figure 4-19 Load vs CMOD of 70% RAP specimens

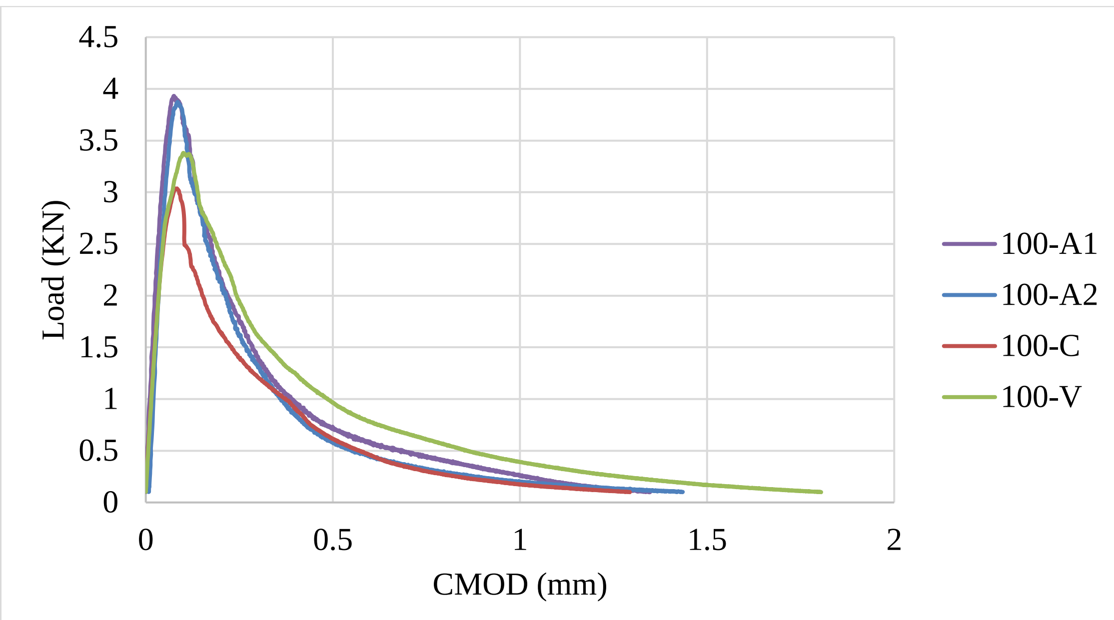


Figure 4-20 Load vs CMOD of 100% RAP specimens

Mixtures with 30% RAP

As can be seen in Table 4-15 and Figure 4-15, the fracture energy of all samples was around 250 J/m and close together. However, the average fracture energy of virgin specimens was higher than fracture energy of other specimens because of their asphalt binder. The asphalt binder of virgin mixtures was unaged and softer than the asphalt binder of other specimens. The fracture energy of specimens with additive 1 and control specimens were almost equal which proved that additive 1 improved the fracture properties of mixture. However, the fracture energy of specimens with additive 2 was lower than the fracture energy of control specimens that proved the additive 2 did not improve the fracture properties of mixtures. Figure 4-18 illustrates the CMOD vs load for 30% RAP mixtures. The maximum load of all specimens was very close to each other which proved most of the samples had same strength, but CMOD of virgin specimens was higher than other specimens that showed these specimens were more ductile.

Mixtures with 70% RAP

Figure 4-16 and Table 4-16 shows the average fracture energy of specimens with 70% RAP materials. The virgin specimens had lower fracture energy than control mixtures. This could be explained by the aggregate gradation. Same as 30% mixture, the additive 1 was more efficient and enhanced the fracture energy of aged mixtures with 70% RAP. The control and modified mixtures with additive 1 had same fracture energy. However, specimens with additive 2 had lower fracture energy than control mixtures which showed this additive could not improve the fracture properties of aged mixtures. It should be mentioned that since the percentage of RAP material was more than 30% mixture, for all specimens, the fracture energy of mixtures with 70% RAP was lower than the fracture energy of mixtures with 30% RAP. Figure 4-19 shows the CMOD vs Load of 70% mixtures. As can be seen in this figure, the control and virgin specimens had the higher maximum load that indicated more strength for these samples, but control and modified mixture with additive 2 had higher CMOD that proved these specimens had more flexible.

Mixtures with 100% RAP

The average fracture energy of all specimens can be seen in Table 4-17 and Figure 4-17. Virgin specimens had the highest fracture energy since these specimens were the softest and the most ductile specimens in 100% RAP mixture samples. Control specimens had the lowest fracture energy since they were aged and fragile. Both additives could improve the fracture properties of 100% mixture, but same as 30% and 70% RAP mixture, additive 1 was more efficient since it could improve the fracture energy of mixtures to the level of virgin mixtures more than additive 2. Additive 2 could improve the fracture properties of mixture, but not as significant as additive 1. It should be mentioned that the aggregate gradation of all specimens in this section, did not meet the aggregate gradation criteria of Superpave mixtures. Figure 4-20 illustrates the CMOD vs load plot of all specimens. As it was expected, the virgin specimens were more ductile since they had the higher value of CMOD. Modified specimens with additive 1 and 2 had more strength since they had higher value of the maximum load compare to other specimens.

4.7.5 Test Result for Each of the Additives

Table 4-18 and Figure 4-21 show the average fracture energy of specimens by the consideration of additive types.

Table 4-18 Average fracture energy of specimens by different additives

	Mixture Fracture Energy (J/m)	
RAP %	**Additive 1**	**Additive 2**
30	342	282
70	268	237
100	296	274

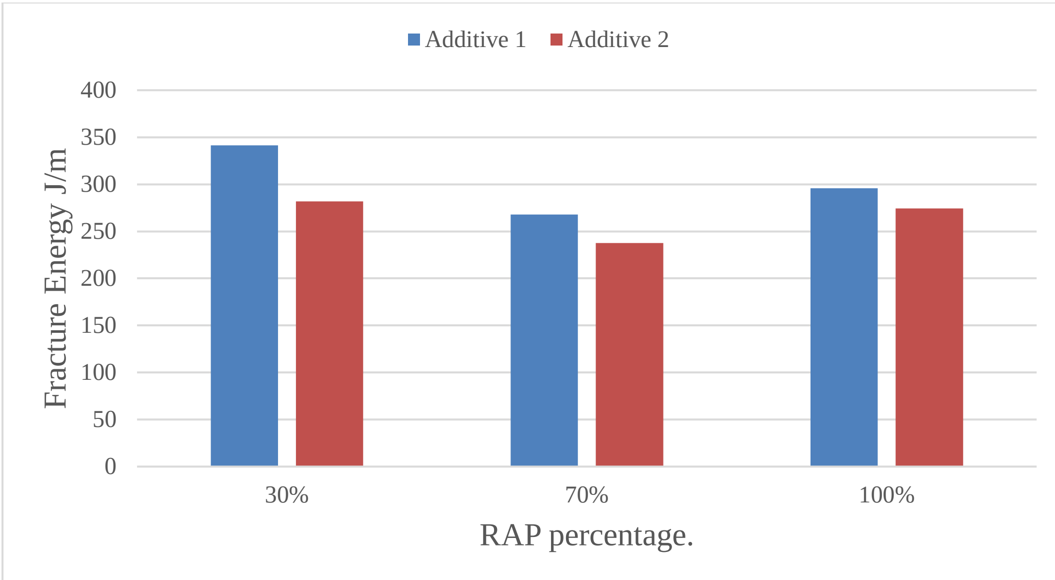


Figure 4-21 Average fracture energy of specimens by different additives

As can be seen in Figure 4-21, for all groups, adding additive 1 improved the fracture energy of aged specimens more than additive 2. This fact proved that additive 1 was better than additive 2 according to the low temperature fracture properties. By increasing the percentage of RAP from 30% to 70% and 70% to 100%, the fracture energy was decreased for 70% RAP mixtures, but increased for 100% RAP mixtures. It should be mentioned that although the fracture energy for 70% mixtures was decreased compare with 30% mixtures, this reduction in fracture energy was lower than our expectation. In other word, by adding 40% RAP materials to the mixtures, the lower fracture energy was expected than the fracture energy was measured for 70% RAP mixtures. For 100% RAP mixtures, the fracture energy was higher than 70% RAP mixtures, but lower than 30% mixture. It can be concluded that additives could improve the fracture properties of mixture, but it should be considered that 100% RAP mixture had the different aggregate gradation. Therefore, one of the possible reasons of this high fracture value could be aggregate gradation.

4.8 Performance Tests for Modified Mixtures with 7% Additive 1 and 11% Additive 2

As it was mentioned, the DCT test results of modified mixtures with the optimum dosage of additives were lower than the standard specifications. Therefore, conduction performance tests for modified mixtures with 7% additive 1 and 11% additive 2 (recommended by additives manufacture factory) were suggested to evaluate the performance of them. In this case, we can compare the performance of modified mixture with the optimum dosage of additives and modified mixture with 7% and 11% of additives. The detailed test plan is listed as Table 4-19.

Table 4-19 Mechanical performance test plan for modified WMA mixtures with 7% additive 1 and 11% additive2

	70% RAP + & 30% Virgin		**100% RAP**	
Sample ID	**HWT**	**DCT**	**HWT**	**DCT**
A1**	XXX*	XXX	XXX	XXX
A2***	XXX	XXX	XXX	XXX
Total	6	6	6	6

*note: X means one replicate
**note: A1 means mixture with 7% dosage of additive one
***note: A2 means mixture with 11% dosage of additive one

4.8.1 HWT Test Results

The HWT Test results for each mixture group are summarized in Table 4-20 and Table 4-21 and plot in Figure 4-22 and Figure 4-23

Table 4-20 Hamburg Wheel Test Results for Mixtures Including 70% RAP and 7% additive 1 and 11% additive2

Mix Type	Test ID	Air Voids, %	Total Number of Passes	Creep Slop	Stripping slope	SIP	Max. Rut Depth, mm
60-A1-7	60-A1-7-1	6.70%	7348	1.803	3.540	5833	20.0
	60-A1-7-2	7.10%	10750	1.064	2.092	9380	20.0
	60-A1-7-3	7.20%	11630	0.975	2.114	9477	20.0
	Average	7.00%	9909	1.3	2.6	8230.0	20.0
60-A2-11	60-A2-11-1	6.90%	8850	0.735	4.953	6311	20.0
	60-A2-11-2	6.94%	9412	0.940	3.340	6698	20.0
	60-A2-11-3	7.23%	10646	1.051	2.785	8071	20.0
	Average	7.02%	9636	0.9	3.7	7027	20.0
60-C	60-C-1	6.64%	20000	0.005	0.017	20000	0.8
	60-C-2	6.86%	20000	0.009	0.013	20000	0.8
	60-C-3	7.1%	20000	0.009	0.011	20000	0.7
	Average	6.86%	20000	0.0	0.0	>20000	0.8
60-V	60-V-1	6.79%	20000	0.036	0.059	20000	2.9
	60-V-2	7.35%	20000	0.056	0.085	20000	3.2
	60-V-3	6.76%	20000	0.047	0.103	20000	3.2
	Average	6.97%	20000	0.0	0.1	>20000	3.1

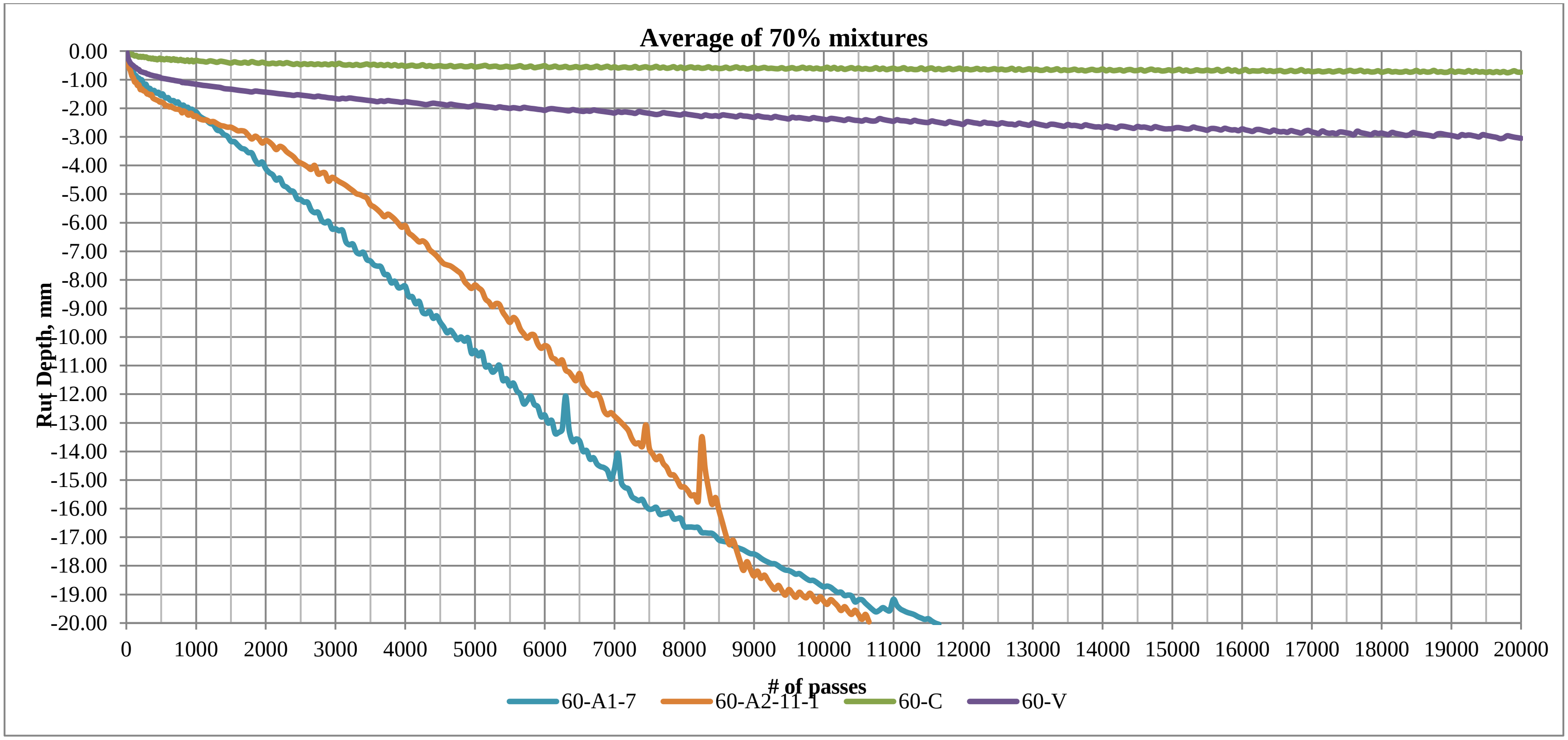


Figure 4-22 Hamburg Wheel Test Results for Mixtures Including 70% RAP with 7% additive 1 and 11% additive 2

Table 4-21 Hamburg Wheel Test Results for Mixtures Including 100% RAP and 7% additive 1 and 11% additive2

Mix Type	Test ID	Air Voids %	Total Number of Passes	Creep Slop	Stripping slope	SIP	Max. Rut Depth, mm
100-A1-7	100-A1-7-1	6.50%	15200	0.249	2.504	9052	20.0
	100-A1-7-2	6.80%	13838	0.454	2.232	9101	20.0
	100-A1-7-3	7.20%	15998	0.401	1.861	7051	20.0
	Average	6.83%	15012	0.368	2.199	8401.3	20.0
100-A2-11	100-A2-11-1	7.30%	19950	0.294	0.827	12045	14.5
	100-A2-11-2	7.10%	20000	0.237	1.818	16021	14.1
	100-A2-11-3	6.90%	20000	0.522	1.186	14260	12.8
	Average	7.10%	19983	0.351	1.277	14109	13.8
100-C	100-C-1	7.18%	20000	0.027	0.048	20000	1.9
	100-C-2	7.27%	20000	0.033	0.010	20000	2.5
	100-C-3	6.75%	20000	0.018	0.085	20000	3.3
	Average	7.06%	20000	0.026	0.048	>20000	2.5
100-V	100-V-1	6.73%	14600	0.367	2.373	8962	20.1
	100-V-2	7.22%	10227	0.493	2.321	5126	20.0
	100-V-3	7.25%	20000	0.314	0.708	12409	16.6
	Average	7.06%	14942	0.392	1.801	8832.3	18.9

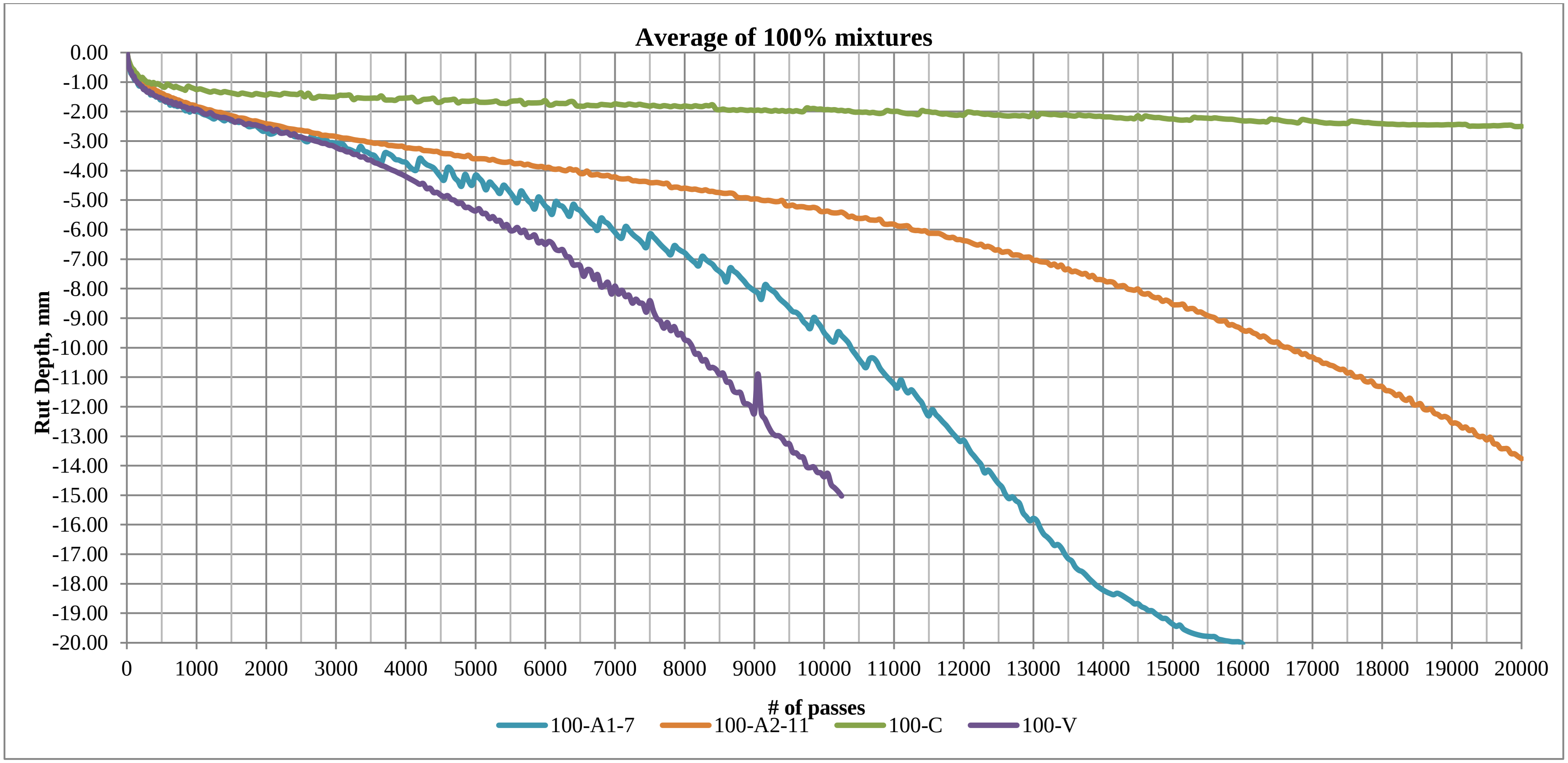


Figure 4-23 Hamburg Wheel Test Results for Mixtures Including 100% RAP with 7% additive 1 and 11% additive 2

Mixture with 70% RAP

As can be seen in Table 4-20 and Figure 4-22, all modified mixtures with 7% additive 1 and 11% additive 2 failed the HWT since the max rutting depth for all of them was 20 mm. In addition, since the average SIP of both mixtures with 7% additive 1 and 11% additive 2 were lower than 1000, it can be concluded that both modified mixture types had moisture susceptibility. One of the possible reasons for that result is high amount of additives. Additional amount of additives than optimum dosages caused softer mixture than modified mixture with optimum dosage. Therefore, the modified mixture with 7% additive 1 and 11% additive 2 was lose and failed.

Mixture with 100% RAP

Table 4-21 and Figure 4-23 show that modified mixtures with 7% additive 1 failed since the maximum rutting depth of all samples was 20 mm before 20,000 passes. However, modified mixtures with 11% additive 2 did not fail in this test since the average rutting depth of this type of sample was 13.8 mm. Although modified mixtures with additive 2 passed this test, they had moisture susceptibility since the SIP value of this mixtures was not higher than 15000. It should be mentioned that modified mixture with 7% additive 1 also had moisture susceptibility.

4.8.2 DCT Test Results

The fracture energy of all specimens are summarized in Table 4-22 and Table 4-23. The average fracture energy of three specimens for each mixture are plotted in Figure 4-24 and Figure 4-25.

Table 4-22 Fracture energy (J/m) 70% RAP mixture with 7% additive 1 and 11% additive2

	70% RAP Mixture Fracture Energy (J/m)				
Sample type	**Specimen 1**	**Specimen 2**	**Specimen 3**	**Specimen 4**	**Average**
60-A1	311	202	291	--	268
60-A1-7	313	362	654	--	343
60-A2	228	262	--	223	237
60-A2-11	337	275	284	--	298
60-C	256	271	--	--	263
60-V	261	201	258	--	240

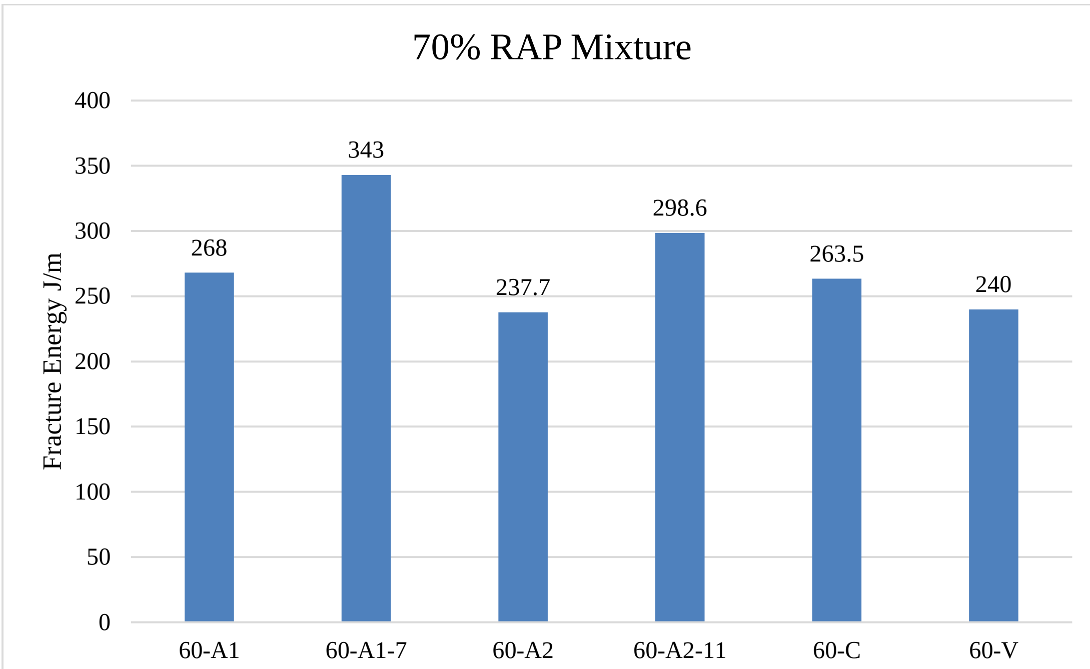


Figure 4-24 Average fracture energy of 70% RAP mixture with 7% additive 1 and 11% additive 2

Table 4-23 Fracture energy (J/m) 100% RAP mixture with 7% additive 1 and 11% additive 2

	100% RAP Mixture Fracture Energy (J/m)				
Sample type	**Specimen 1**	**Specimen 2**	**Specimen 3**	**Specimen 4**	**Average**
100-A1	320	284	--	283	296
100-A1-7	314	398	339	--	350
100-A2	320	259	254	265	274
100-A2-11	308	390	310	--	336
100-C	222	247	257	213	235
100-V	306	352	230	--	296

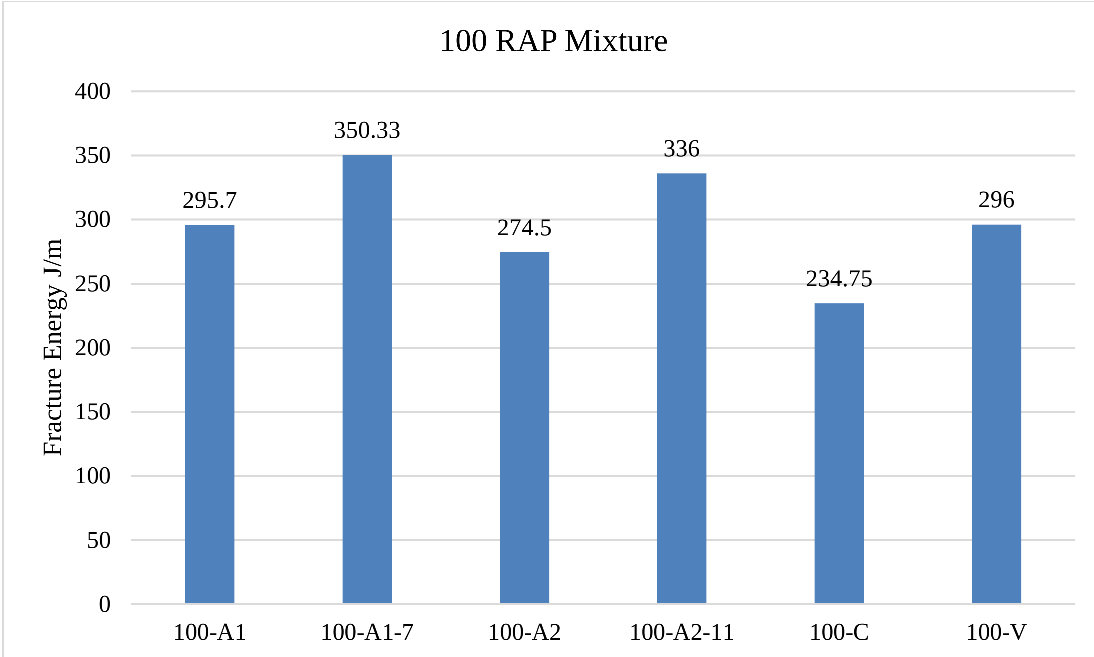


Figure 4-25 Average fracture energy of 100% RAP mixture with 7% additive 1 and 11% additive 2

As can be seen in Figure 4-24 and Figure 4-25, for both types of mixtures, increasing the percentage of additives from 3.99% to 7% for additive 1 and 7.77% to 11% for additive 2 could increase the fracture energy of modified mixtures. However, this increase was not enough and still it was below standard specifications.

4.8.3 Conclusion of Performance Tests for Modified Mixture with 7% Additive 1 and 11% Additive 2

From both HWT and DCT test it can be concluded that increasing the percentage of additives from optimum dosage to 7% for additive 1 and 11% for additive 2 could improve the low temperature fracture properties of mixture. However, still the fracture energy was lower than standard specifications. This is important because the major reason for conduction these performance tests for these modified mixture was increasing the fracture energy to meet the standard specifications which it did not happen. In addition, increasing the dosage of additives caused the rutting and moisture susceptibility for both types of mixture. Therefore, increasing the dosage of additives from optimum to the recommended value by additives manufacture factory is not suggested.

4.9 Summary of the Chapter

In this chapter, the performance properties of modified mixtures containing 30, 70 and 100 percent RAP were investigated. The mixtures with the optimum dosage of each additive were compacted. To evaluate the moisture susceptibility and routing of modified mixtures, the HWT test was conducted. The DCT test was considered to evaluate the fracture properties of mixtures at low temperatures. According to the HWT test result, none of the modified mixtures with additives had the moisture susceptibility and rutting. However, additive 2 made the mixture softer than additive 1, but it did not have a significant effect on rutting. DCT test result illustrated that additive 2 was more efficient for all mixtures, but it should be mentioned that the fracture energy of all samples was lower than the Superpave criteria. Investigation the result of all performance tests, it can be concluded that mixtures with 70% RAP had an acceptable performance. However, modification and chemical improvement of WIR additives can result in higher performance mixtures with more RAP materials. Figure 4-26 shows the summary of this chapter.

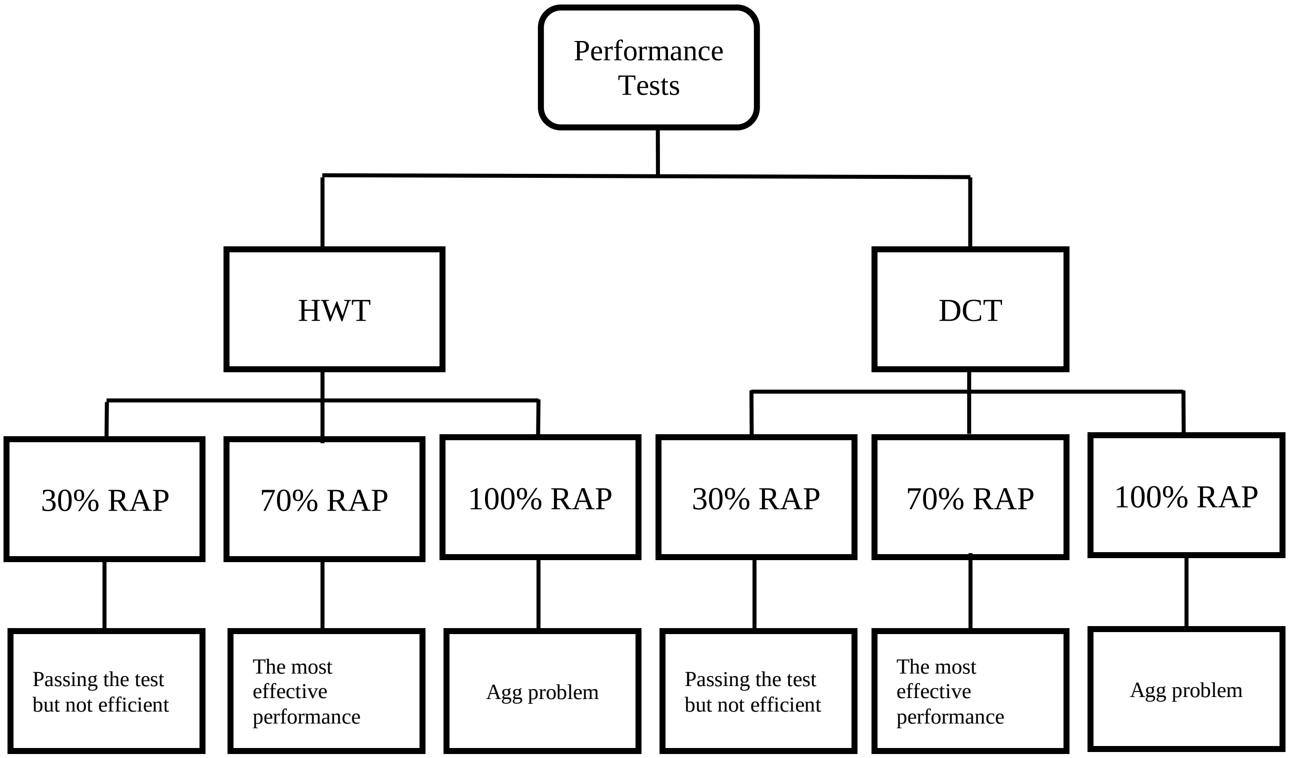


Figure 4-26 Chart. Summary of chapter 4

CHAPTER 5: SUMMARY OF FINDINGS, CONCLUSIONS, AND RECOMMENDATIONS

5.1 Summary

Hot in-place recycling (HIR) is an economical pavement rehabilitation strategy that can be used to recycle asphalt pavements. The process of HIR is to 1) soften the pavement surface distress with heat, 2) remove the existing surface, 3) mix it with an asphalt binder and additional virgin aggregate and 4) replace the recycled material on the original pavement site. Two major limitations of HIR are high mixing/compaction temperatures and a utilization of low amounts of recycling materials. For cold in-place recycling (CIR), damaging the aggregate gradation of milled mixtures is the major limitations. To improve limitations of both recycling methods, two Warm In-place recycling (WIR) is proposed while utilizing the eco-friendly additives. The usage of these additives is desired to decrease the mixing and compaction temperatures while increasing the RAP percentage in recycling process. The mixing and compaction temperatures in this new WIR recycling process are between those of CIR and HIR. The main objectives of this study are: 1) determination of the optimum dosage of two WIR additives by evaluation of rheological properties of modified asphalt binders and 2) evaluation of the mechanistic performances of modified mixture with the optimum dosage of additives containing different percentages of RAP.

In **chapter 3**, the Superpave tests were performed to evaluate the rheological properties of modified asphalt binders. This evaluation was to 1) identify the optimum dosage of WIR additives and 2) evaluate the effects of these additives on rheological and mechanical properties of asphalt binders. These tests were executed on two types of sample: 1) 100% aged asphalt binder and 2) 60% aged asphalt binder plus 40% virgin asphalt binder. The second sample type was proposed to: 1) verify the optimum dosage of additives calculated in the first sample type and 2) prepare modified mixtures with 70% RAP with the optimum dosage of additives of second sample type. The descriptions of asphalt binder tests along with result of tests are summarized as follows:

a) Dynamic Shear Rheometer (DSR) test: The major intent of DSR test is measuring the rheological properties of asphalt binder at high and intermediate temperatures. For virgin and RTFO aged asphalt binders, the test temperatures were 64 70, 76 and 82°C, while the frequency was 1.59 Hz. For the PAV aged asphalt binders, the test temperatures were 13, 19, 25, 31 and 37°C, while the frequency was 1.59 Hz. $G^*/\sin(\delta)$ should be greater than 1.0 kPa before aging in order to control rutting during transport, storage, and handling and greater than 2.2 kPa after RTFO to prevent rutting after the construction of asphalt mixtures. $G^*\times\sin(\delta)$ is the Superpave specification to control the fatigue cracking after long period in service. $G^*\times\sin(\delta)$ at the intermediate temperature should be less than 5000 kPa for PAV aged asphalt binders. For 100% aged sample, the maximum percentages for additives 1 and 2 were identified as 8.22% and 21.5%, respectively. The maximum percentages of additives 1 and 2 for second sample type (60% aged asphalt binder plus 40% virgin asphalt binder) were 6.99% and 21.23%, respectively.

b) Bending Beam Rheometer (BBR) Test: The BBR test is to investigate the low temperature performance (thermal cracking) of asphalt binders. In this study, -6, -12, -18, -24 and -30 °C were selected as test temperatures to determine the low temperature of the PG grade according to the dosage of additives. Two important parameters of BBR test are the stiffness and m-value. If the stiffness of a sample is less than 300 MPa and the m-value is greater than 0.3 at the specific temperature, the sample meets Superpave criteria at that temperature. The Superpave specifications of all groups (stiffness and m-value) were measured at -12 °C and by interpolation and extrapolation, the optimum dosages of additives were identified. For 100% aged asphalt binder sample, the optimum dosages were 3.99% and 7.77% for additives 1 and 2, respectively. For second type of sample (60% aged asphalt binder plus 40% virgin asphalt binder), 3.76% and 8.12% were identified as the optimum percentage for additives 1 and 2, respectively.

c) Rotational Viscometer (RV) Test: The RV test is to determine the viscosity at the high temperature and furthermore the workability of asphalt binders. The RV test for virgin asphalt binder can determine the mixing and compaction temperatures of asphalt mixtures during the construction as well. The viscosity was measured using a rotation viscometer at 85, 95, 105, 115, 125 and 135°C in this study. The spindle of 25mm in diameter and the rotation speed of 0.3 Hz were selected. The mixing and compaction temperatures of modified asphalt mixtures with the optimum percentage of additives were identified. For additive 1, the mixing temperature was between 132 °C and 136°C and the compaction temperature was between 120 °C and 125 °C. The mixing and compaction temperatures for additive 2 were identified as between 133 °C and 138°C and between 121 °C and 126 °C, respectively.

d) Fourier Transform Infrared (FTIR) Spectroscopy test: FTIR is a technique that can determine the functional characteristics of a material. FITR was also utilized to analyze the oxidative aging of asphalt binders by observing the amount change of carbonyl (C=O) and sulfoxide (S=O). The oxidative aging index was determined by the bond ratio changes before and after the aging. FTIR analysis was performed on the aged asphalt with and without the rejuvenators. Additive 2 reduced the oxidation of sulfur and carbon more than additive 1 since modified binders with additive 2 had lower value of sulfoxide and carbonyl indexes than modified binder with additive 1.

e) Multiple Stress Creep Recovery (MSCR) Test: This specification characterizes asphalt behavior at high temperature. The ultimate purpose of this test is quantification of the asphalt binder contribution to rutting resistance, especially with modified binder. This test method covers the determination of percent recovery and nonrecoverable creep compliance of asphalt binders. It is more desired to have a higher percentage of recovery and a lower value of creep compliance (J_{nr}). The test was started with the application of a low stress 0.1 kPa for 10 creep/recovery cycles, and then the stress was increased to 3.2 kPa, which was repeated for an additional 10 cycles. According to the percentage of recovery and creep compliance, Additive 2 had better performance since for both stress levels, modified binder with additive 2 had lower value of J_{nr} and higher value of recovery than modified binder with additive 1.

All asphalt binder test results were described in Chapter 3. First, the maximum and optimum percentages of additives were identified by DSR and BBR tests, respectively. Next, the mixing and compaction temperatures of mixture were indicated by RV. Finally, the aging and recoverability properties of modified asphalt binders were evaluated by FTIR and MSCR.

In **chapter 4**, the performance properties of modified mixtures containing 30, 70 and 100 percentages of RAP materials were investigated. The RAP material properties including aggregate gradation, moisture content and asphalt binder content were discussed along with the compatibility of RAP materials. Next, the mix designs of three mixtures with three different percentages of RAP materials were discussed. Finally, the modified mixtures with the optimum dosage of each additive were compacted. To simulate the WIR materials, the curing of 4 hours was adopted for RAP materials and the aggregated gradation of RAP materials was also modified. To evaluate the moisture susceptibility and rutting performance of modified mixture, the HWT test was conducted. The DCT test was considered to evaluate the fracture properties of mixture at low temperature. The description and results of HWT and DCT tests are summarized as follows:

a) Hamburg Wheel Track (HWT) test: The tests were run in a water bath that was heated to 50 °C after the test specimens were conditioned for 30 minutes. A constant load of 685 N through a steel wheel with a diameter of 203.5 mm and a width of 47.0 mm was applied. Four properties including:

1) post-compaction consolidation, 2) creep slope, 3) stripping inflection point, and 4) stripping slope were measured for each sample. The specimens had a target air void content of 7.0 ± 2.0 %. Mixtures containing 30% RAP passed the test. Adding additives 1 and 2 improved the resistance of mixture against moisture susceptibility. 70% RAP mixtures successfully passed the test with the max rutting depth less than that of 30% RAP mixtures. The average SIP for 70% RAP specimens was higher than 20,000 passes, which was significantly higher than SIP of 30% mixtures. This proved that none of the specimens had a moisture susceptibility. 100% RAP mixtures, except the virgin mixtures, successfully passed the HWT test and the average SIP of all specimens, except the virgin mixtures, was greater than 20,000 passes.

b) Disc-Shaped Compacted Tension (DCT) test: The DCT test was conducted to evaluate the low-temperature cracking properties of modified mixtures. Since the PG grade of asphalt used in this project is PG -22, temperature for DCT testing in this study was recommended to be -12 °C. The DCT test is controlled by a constant crack mouth opening displacement (CMOD) rate of 0.017 mm/s (approximately 1.0 mm/min). Loads were plotted against CMOD, with the area under the curve being fracture work. The average fracture energy of 30% RAP mixtures was lower than the minimum level required by AASHTO standard. The fracture energy of specimens with additive 1 and control specimens were almost equal which proved that additive 1 did not improve the fracture properties of mixture. However, the fracture energy of specimens with additive 2 was lower than the fracture energy of control specimens that proved additive 2 lowered the fracture resistance of mixtures. 70% RAP mixtures did not meet the AASTO standard. Similar to the 30% RAP mixture, the additive 1 was more effective in improving the fracture resistance of aged mixtures than additive 2. The fracture energy of 70% RAP mixtures was lower than that of 30% RAP mixtures because it contains a higher amount of RAP materials. However, this small amount of reduction in fracture energy was less than expected although the percentage of RAP was increased by 40%. 100% RAP mixtures did not meet the AASHTO standard either. Similar to 30% and 70% RAP mixtures, additive 1 was more effective in increasing the fracture resistance than additive 2.

5.2 Observations

The key observations made from experimental testing and analytical studies described in this document are highlighted as follows.

1) Although the mixing and compaction temperatures of modified asphalt binder were measured by RV test accurately, the workability of modified mixtures with additive 2 was lower than that of modified mixtures with additive 1. It is recommended that higher mixing and compaction temperatures should be used for the modified mixtures with additive 2.
2) According to the producer of the additives, a high shear mixer is not required to mix additive 2 with asphalt binders. However, modified asphalt binder with additive 2 prepared without the high shear mixer seemed not be completely uniform and consistent in some cases.

3) During the preparation of some DSR samples with additive 2, a small amount of liquid (mix of the additive and asphalt binders) was observed in DSR silicon mold. It can be postulated that it was the result of imperfect mixing process mentioned above.

5.3 Conclusions

The following conclusions were drawn from the results of the experimental and analytical studies presented in this document.

1) Proposed rheological test method was able to identify the optimum dosage of additives accurately since the second round of tests verified the result, and the performance of modified mixtures with the optimum percentage of additives was acceptable.
2) The RV test result proved that the mixing and compaction temperatures of modified asphalt mixture were decreased by about 20ºC for each of the additives. One of the major objectives of this study was to decrease the mixing, compacting and scarification temperatures during the in-place recycling process. This test showed that with these new eco-friendly additives, reduction of 20 ºC in scarification, mixing and compaction steps in recycling process can be achieved. Therefore, it is recommended that a temperature of 80 ºC should be adopted during the scarification process, a temperature between 120 ºC and 130 ºC for compaction process and 130 ºC to 135 ºC for mixing process.
3) The MCSR test results illustrated that the additives, specifically additive 2, improved the recoverability of modified asphalt binders. However, the

creep compliance did not improved for both additives. Overall, additive 2 peformed bether that additve 1 based on the MSCR test.

4) The FTIR test results showed that both additives improved the aging properties of aged asphalt binders. Additive 2 can be considered more effective because it lowered the aging index more than additive 1.

5) DSR test results proved that adding 5% of additive 1 decreased the high temperature PG grade of aged asphalt binder by one grade but adding 7.5% of additive 2, did not change the PG grade. Adding 10% of additive 1 decreased the PG grade by two grades and adding 15% of additive 2 decreased it by one grade. Overall, additive 1 made the asphalt binder softer that additive 2.

6) BBR test results illustrated that adding every 5% of additive 1 and every 15% of additive 2 could decrease the low temperature PG grade of aged asphalt by one grade. Additive 1 exhibited a better performance based on the BBR test.

7) HWT test results exhibited that the modified mixtures did not have the moisture susceptibility and rutting because of the high percentage of RAP. However, adding WIR additives made the mixtures softer. Since the mixture with 100% RAP material did not meet the superpave aggregate gradation and mix design specifications, it is recommended that the 70% RAP mixtures should be adopted based on the HWT test.

8) Although the DCT test results were not consistent, and did not meet AASHTO specifications, they proved that adding additives, specifically

additive 1, were able to improve the low temperature fracture properties of mixtures. Although the 100% RAP mixtures exhibited almost identical fracture energy as 70% RAP mixtures, 70% RAP mixture is recommended based on the DCT test. Both 70% and 100% RAP mixtures exhibited slightly lower fracture energy than 30% RAP mixtures. But, the reduction of fracture energy of 70% RAP mixtures was not significant compare to the 30% RAP mixtures.

These preliminary results demonstrate the feasibility of WIR. According to the rheological and performance tests in this study, it can be concluded that adding 4% of additive 1 and 8% of additive 2 should be able to decrease the mixing and compaction temperatures of modified mixture by 20ºC. They also could increase the percentage of recycled material up to 70% especially in a warm climate area where a cold temperature is not a concern. It should be mentioned that, overall, additive 1 performed better than additive 2 while making the mixture softer, but additive 2 could improve the recoverability and aging properties of mixture better than additive 1.

5.4 Future Research

In this study, a series of tests were conducted to evaluate the rheological properties of modified asphalt binder with two new eco-friendly additives and the mechanistic performances of modified mixtures. Although some findings were obtained from the test results, more improvements can be achieved to better develop Warm In-place recycling process. The following future research should be conducted:

1) Overall, additive 1 performed better that additive 2. But, additive 2 enhanced the aging and recoverability of modified asphalt binder. Therefore, blending of the desired fractions of additive 1 and 2 could

produce a new additive that may improve all properties of modified mixtures.

2) Although DCT and HWT tests were performed in this study to evaluate performance of modified mixtures, more performance tests such as beam fatigue and dynamic modulus tests can help evaluated these additives better.
3) To develop a new WIR process, two new eco-friendly additives were evaluated in this study. To further improve the WIR process, the new in-place heating/milling/paving machine that includes a pollution control unit should be developed.

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