Design and Characterization of Cold Recycled Foamed Asphalt Mixtures with High RAP Contents

by

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ABSTRACT

Cold recycling with foamed asphalt has been considered as a sustainable pavement rehabilitation technology. Although different mix design methods have been proposed, there is no widely accepted standard. The effects of mineral additives on the performances of cold recycled mixes are not well understood in either laboratory or field curing conditions. Moreover, there is a lack of guidance on how to evaluate cold recycled mix resistance to permanent deformation and cracking using laboratory testing methods. To address these research needs, this study included three tasks, which are (1) improvement of the mix design method, (2) investigation of the effects of mineral additives, and (3) evaluation of different laboratory testing methods. In the mix design study, the compactive effort based on the Superpave gyratory compactor (SGC) was determined by matching laboratory-compacted densities with the field-compacted densities. Accordingly, an SGC-based approach was developed to determine the optimum water content (OWC) of reclaimed asphalt pavement (RAP) material which is the water content needed to achieve the maximum dry density. However, the OWC does not yield the maximum indirect tensile strength (ITS) which is the primary mix design objective. To maximize the ITS, a multiple linear regression model for determining the optimum total water content (OTWC) was developed and validated. For the mineral additive study, recycled mixtures were prepared with cement, hydrated lime, fly ash, and asphalt plant baghouse fines. Results showed that the mixtures with cement had the highest ITS and the best resistance to moisture damage in both laboratory and simulated-field curing conditions compared with mixtures containing the other mineral additives. A curing experiment showed that mixtures cured in the typical laboratory condition (40°C for 3 days) had similar rankings regarding the dry or wet ITS results as those cured in a 100-day simulated-field condition. In the laboratory testing study, dynamic modulus ($|E^*|$) of small-scale specimens was used to assess the undamaged viscoelastic properties of cold recycled mixes. However, this method may not be ready for implementing in pavement M-E design. Resistance of cold recycled mixtures to permanent deformation was

characterized by the strain accumulation rate in the steady state (k_{st}) and the strain at the beginning of the steady state (ε_{st}) from Flow Number (FN) test. The Flexibility Index (FI) from the Illinois Flexibility Index Test (I-FIT) and the proposed Fracture Work Factor (FWF) based on ITS test data was used to evaluate the cracking resistance. The variabilities of these test results for cold recycled mixes were higher than those of typical HMA materials. These laboratory testing results showed the RAP gradation has a statistically significant effect on the ITS, $|E^*|$, FI, and FWF results. It may also impact the results from FN test (k_{st} and ε_{st}). The cold recycled mixes with finer RAP had higher ITS, $|E^*|$, and the FN test results but lower FI results compared to those with coarser RAP. The effect of virgin binder was only significant on the ITS result and may also have impact on the results from FN test (k_{st} and ε_{st}). It is worth mentioning that the conclusions from this study were based only on test results of the limited materials. Further research using more materials and validation with field conditions and performance are needed.

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LIST OF ABBREVIATIONS

ANOVA	Analysis of Variance
ARRA	America Reclaiming and Recycling Association
AMPT	Asphalt Mixture Performance Tester
CCPR	Cold Central-Plant Recycling
CIR	Cold In-Place Recycling
COV	Coefficient of Variation
ER	Expansion Ratio of foamed asphalt
HL	Half-Life of foamed asphalt
FAC	Foamed Asphalt Content in mixture
FWF	Fracture Work Factor, a proposed index to evaluate cracking resistance of the recycled mixture
ITS	Indirect Tensile Strength
MWC	Mixing Water Content
NMAS	Nominal Maximum Aggregate Size
OFAC	Optimum Foamed Asphalt Content in mixture
OMC/OWC	Optimum Moisture/Water Content, this study used OWC specifically for reclaimed asphalt pavement material.
OTWC	Optimum Total Water Content for mixture
RAP	Reclaimed Asphalt Pavement
TSR	Tensile Strength Ratio

TWC	Total Water Content for mixture
XRD	X-Ray Diffraction, used to evaluate mineral components of the baghouse fines
$\|E^*\|$	Dynamic Modulus
ε_{st}	The accumulated strain at the beginning of steady state during Flow Number test
k_{st}	Strain accumulation rate in the steady state during the Flow Number test
N_{st}	The number of load cycle at the beginning of steady state during the Flow Number test

CHAPTER 1 INTRODUCTION

1.1 Background

Cold recycling has been considered as a sustainable technology for pavement rehabilitation (Chan et al. 2010, Stroup-Gardiner 2011, Thenoux et al. 2007). Cold recycled mixtures may contain up to 98 percent of reclaimed asphalt pavement (RAP), which saves large quantities of virgin aggregate and reduces fuel consumption for heating and drying (Diefenderfer et al. 2016). Because cold recycled mixtures are produced at ambient temperature, greenhouse gases generated during paving and compaction are significantly reduced.

Two recycling agents that have been commonly used in cold recycling are foamed asphalt and emulsified asphalt. These recycling agents mix with RAP at an ambient temperature and provide bonding between particles. However, the recycling agents are quite different in terms of the stabilizing mechanism. Use of foamed asphalt in cold recycling was first proposed and patented by Dr. Csanyi in the late 1950s (Csanyi 1957). The foaming process was achieved by adding steam into hot liquid asphalt to reduce the viscosity of asphalt binder. In 1968, Mobil Oil Australia acquired the patent and modified the production procedure by using cold water to foam asphalt binder. This procedure was found more convenient for in-place recycling (Muthen 1998). The modern asphalt foaming technique introduces water and air into hot asphalt binder. Water vapor trapped within the binder expands the asphalt volume and reduces its viscosity to facilitate the distribution of the foamed asphalt binder among RAP particles to form non-continuous bonding (Diefenderfer et al. 2016, Muthen 1998). Application of asphalt emulsion in road construction began in the 1920s. Asphalt emulsions are a dispersion of small asphalt droplets in water by means of emulsifier to prevent the droplets from coalescing. An asphalt emulsion has a much lower viscosity than hot asphalt binder (James 2006). After the emulsion breaks (a term used to describe when the asphalt binder separates from the water), asphalt coats the particles with a thin film (Baumgardner 2006).

Regardless of the type of recycling agent, cold recycled mixtures can be produced either by cold central-plant recycling (CCPR) using a simple plant typically located adjacent to the project site, or by cold in-place recycling (CIR) using processing and mixing equipment on the roadway. CCPR production allows the use of RAP milled from the current project or other RAP sources and provides better control for gradation and additive content. Fractioned coarse and fine RAP materials can be added into two separate bins with pre-determined proportions to control the gradation of RAP in cold recycled mixtures. Mineral additives can be added to the mixture at a control speed through a metering unit. CCPR mixtures may be placed in more than one lift and/or the construction of new lanes. After produced in the central-plant, the recycled mixtures are typically transported to the construction site in dump trucks. Since the CIR production technique recycles an asphalt pavement in-situ within a single pass, the recycled pavement thickness is often between 50 - 125 mm (2 - 5 inches) (Diefenderfer et al. 2016). Studies have shown that CCPR and CIR mixtures have similar resilient moduli and dynamic moduli (Diefenderfer et al. 2016, Asphalt Academy 2009). Therefore, this study focuses on cold recycled foamed asphalt mixtures regardless of production methods. The findings and conclusions of this research are applicable to both CCPR and CIR production techniques.

The cold recycling technology with foamed asphalt had several advantages. First, foamed asphalt improves the shear strength and moisture damage resistance of reclaimed asphalt pavements (RAP). Second, strength of the recycled mixture is comparable to cement-stabilized materials but more flexible and fatigue resistant. Additionally, cold recycling saves energy because of no requirement for heating during production. The greenhouse gas emissions during construction are also reduced (Muthen 1998).

Cold recycling technology with foamed asphalt has become more and more popular in the U.S. and worldwide since the 1990s. In US, more than 11 states have implemented this technology in pavement rehabilitation. Worldwide, it has been widely used in several countries including Greece, Canada, South Africa, Australia, New Zealand, and China (Diefenderfer et al. 2016, Chan et al. 2010, Muthen 1998, Thenoux et al. 2007, Loizos 2007, Saleh 2004, Yu 2005).

Although a few different mix design procedures have been used by different organizations (Asphalt Academy 2009, Wirtgen GmbH 2012, and ARRA 2016), there is no national standard for the design of cold recycled mixtures with foamed asphalt. The existing methods use different laboratory compactive efforts and are generally based on dated methods such as the Marshall or the modified Procter hammers. Typically, the optimum total water content (OTWC) was determined based on its relationship with the RAP material's optimum water content (OWC). However, there was no commonly used relationship. The modified Proctor test was often used for determining the RAP's OWC from the plot of moisture content versus maximum dry density. However, the modified Proctor test to obtain the OWC is time consuming and few asphalt laboratories are equipped to conduct the method. In addition, few studies focused on the effects of using mineral additives in cold recycled mixtures (Nosetti et al. 2016, Thanaya et al. 2009, Cross and Young 1997). Moreover, there is no agreement on how to appropriately characterize cold recycled mixtures for specifications, construction control, or for input in structural pavement design methods. Although some research provided results of several laboratory tests for cold recycled mixtures, the variabilities of the results and their correlations with the field performances had not been reported (Cox and Howard 2016, Diefenderfer and Apeagyei 2014, Kim et al. 2009, Thomas and Kadrmas 2007, Muthen 1998). Few studies had investigated the effects of RAP gradation on the laboratory test results (Kim and Lee 2006, Kim et al. 2009). No study has been found related to the effect of virgin binder performance grade (PG) on cold recycled mixtures. Table 1.1 summarizes the major concerns to be addressed based on the literature review.

Table 1.1 Problems in Design and Characterization of Cold Recycled Foamed Asphalt Mixtures

Topics	Problems
Mix Design	• No national standard laboratory mix design procedure • The laboratory compactive effort has not been validated by matching with the field compactive effort • Operating the modified Proctor test equipment to determine the OWC of RAP material is time consuming and requires intensive labor if manual hammer is used. The mechanical hammer may not be readily available in typical asphalt laboratories. • No agreement on the OTWC of cold recycled mix
Mineral Additive	• Few studies on the effects of mineral additives on cold recycled mixes based on literature review • Unknown effects of additives on strengths of mixtures in different curing conditions in both short- and long-terms
Laboratory Testing	• Few studies on the effect of RAP gradation and no study on the effect of virgin binder PG on the laboratory test results • No standard dynamic modulus test method for cold recycled mixtures • No guidelines for assessing the cracking and permanent deformation resistance of cold recycled mixtures

1.2 Objectives

Based on the current issues in cold recycled foamed asphalt pavement technology, the primary objectives of this study were:

(1) Improve the mix design procedure for cold recycled foamed asphalt mixtures.
(2) Investigate the effects of different mineral additives on the indirect tensile strengths (ITS) of cold recycled foamed asphalt mixtures in both laboratory and a simulated-field curing condition (i.e., outdoor curing exposed to sunlight and wind as discussed in Section 5.1).
(3) Investigate suitable methods to evaluate performance-related characteristics of cold recycled mixtures, including strength, dynamic moduli, resistances to permanent deformation and cracking and to determine the effects of RAP gradation and binder type/content on the laboratory testing results.

1.3 Scope

The improvements to the mix design procedure focused on three aspects including the laboratory compactive effort, the determining method for the OWC of RAP material, and the determining method for the optimum total water content in the mixture. Thirteen different mixtures produced in the laboratory were used for this study.

The effects of different mineral additives on the indirect tensile strength (ITS) of cold recycled foamed asphalt mixtures were evaluated after laboratory condition (3 days at 40°C) and a simulated field curing condition (outdoor environment in Auburn, Alabama during winter time and covered before raining). In the laboratory curing condition, the effects of additive type, additive content, and curing moisture were studied. In a simulated field curing condition, the effects of curing time, top load and confinement were evaluated. The top load was to simulate the hot mix asphalt (HMA) surface layer placed above the cold recycled pavement layer. HMA cores with same mass (about 965 g) and height (about 54 mm) were placed on top of the cold recycled mixture specimen as the top load during curing.

The characteristics of the six cold recycled mixtures were evaluated using the indirect tensile strength (ITS) test, dynamic modulus test, Flow Number test, and the Illinois Flexibility Index Test. A new parameter based on the ITS test was proposed to evaluate the cracking resistance. Other two new results based on the Flow Number test were proposed for evaluating the resistance to permanent deformation. Variability of these laboratory test results for cold recycled mixes were evaluated and compared to the variability of typical HMA test results. Also, the laboratory test results of the six cold recycled mixes were compared to those of typical HMA materials.

1.4 Organization of Dissertation

This dissertation contains a literature review of the design and characterization of cold recycled foamed asphalt mixtures in Chapter 2, including an in-depth look at material components, typical mix design methods, use of mineral additive types and contents, as well as the previous studies on the laboratory testing methods.

Chapter 3 summarizes the materials used in this research including eight RAP materials, three foamed asphalt binders, four mineral additives, and fifteen proposed cold recycled foamed asphalt mixtures.

Chapter 4, 5, and 6 discuss three aspects of the cold recycled mixes including mix design, mineral additives, and laboratory testing. Discussion for each study consists of the methodology, results analysis, and summary. Chapter 4 describes three proposed improvements to the typical mix design procedure and describes the design procedures after improvements. Chapter 5 discusses the use of mineral additive in the cold recycled mixes, including the effects of additives types and contents on the ITS of the mixtures. The test results obtained from the laboratory curing condition and the simulated field curing condition are discussed and compared. Chapter 6 focuses on the laboratory testing methods for evaluating the six cold recycled mixes. The effects of RAP gradation and binder types on the test results were determined. Finally, the conclusions and recommendations of this dissertation are summarized in Chapter 7.

CHAPTER 2 LITERATURE REVIEW

This chapter summarizes the literature related to cold recycling technology with foamed asphalt. The literature review focused on the mix design methods, use of mineral additives, and laboratory testing methods.

2.1 Mix Design

Current cold recycling mix design procedures for cold recycled foamed asphalt mixture consists of six steps. Figure 2.1 shows these general steps based on the recommended methods from the American Reclaiming and Recycling Association (ARRA) and Wirtgen (ARRA 2016, Wirtgen GmbH 2012). After the materials are selected, there are two targets to be determined from the design. The first is the optimum total water content (OTWC) to achieve the maximum compacted density. The second target is the optimum foamed asphalt content (OFAC) to achieve the minimum strength criteria. The specific criteria are discussed in Section 2.1.4.2.

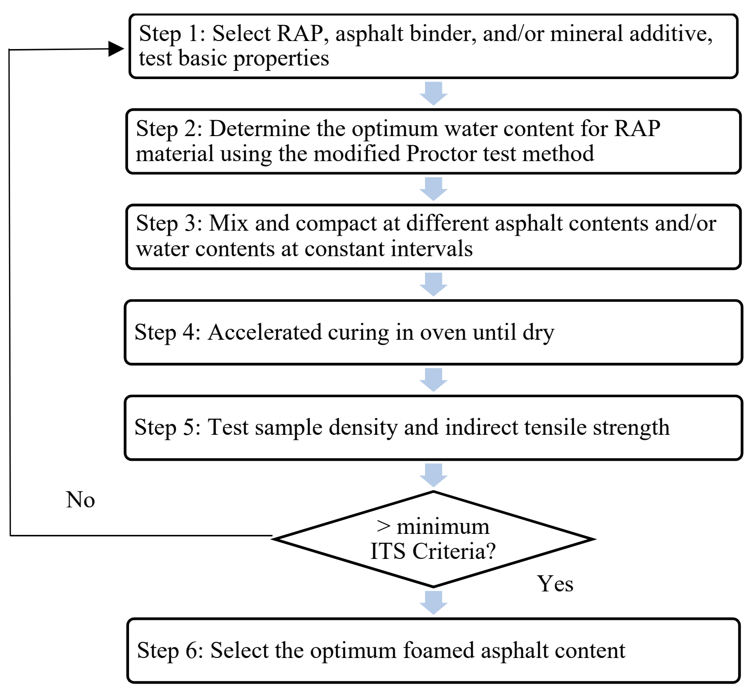


Figure 2.1 Typical Mix Design Procedures for Cold Recycled Foamed Asphalt Mixtures (ARRA 2016, Wirtgen GmbH 2012)

2.1.1 Material Preparation and Testing

Cold recycled foamed asphalt mixtures discussed in this study only focused on recycling the reclaimed asphalt pavement (RAP). Figure 2.2 illustrates the material components in a recycled mixture. The coarse RAP particles form structural skeleton. The fine RAP particles and the mineral additive (if any) blend with foamed asphalt to form "mastic" (Sakr and Manke 1985, Asphalt Academy 2009). The water used in the mixture facilitates mixing and compaction.

The objectives of mix design include selection of appropriate materials and the determination of the optimum proportion for each material to meet the design requirements. Similar to HMA design, cold recycled mix with foamed asphalt follows a step-by-step design process including RAP evaluation and selection, foamed asphalt preparation, mixing, compaction, curing, and performance testing. However, the cold recycled mix design procedures are different from HMA design procedures, regarding the characterization of foamed asphalt, determination of the OTWC for mixture, laboratory compactive effort, and curing condition.

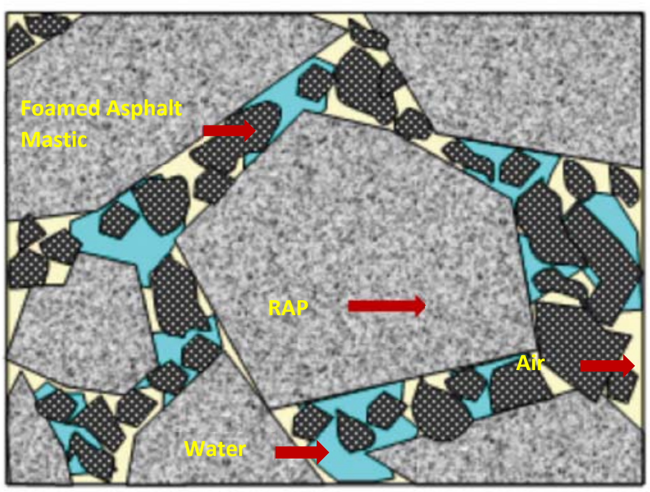


Figure 2.2 Material Components in the Cold Recycled Foamed Asphalt Mixture (Asphalt Academy 2009)

2.1.1.1 Preparing and Testing RAP

Characteristics of the RAP may need to be tested to determine gradation, percent of fines (passing No.200 sieve), flat and elongated (F&E) ratio, RAP binder condition and content, and the optimum water content.

Preparation

Cold recycled mixtures contain up to 98% RAP. RAP is typically stored in one unfractionated stockpile or two different stockpiles with only coarse or fine fraction. It may be difficult to sample RAP with a consistent gradation due to the segregation in stockpile. Fractionated RAP has also been used for cold recycled mixtures. Kim et al. (2007) studied foamed asphalt mixes with seven different RAP sources. RAP was sampled after large particles (retained on 25 mm sieve) were removed. Each RAP type was separated into five sizes (retained on 19, 9.5, 4.75, 1.18 mm sieve, and passing 1.18 mm sieve) and then combined before mixing. Similarly, Eller and Olson (2009) suggested separating RAP into three different fractions and blending.

Cosentino et al. (2012) studied a gradation modification technique for RAP in cold recycling mix design, which is re-blending RAP following maximum density gradation formula in FHWA T5060.27 1988. The performance of compacted RAP was evaluated at a room temperature by using the uniaxial creep test and the Limerock Bearing Ratio (LBR) strength test. Test results showed that remixing to the maximum density did improve the LBR strength and reduce creep strain rate, compared to unprocessed RAP.

Testing

Gradation testing is the most common method to characterize RAP. Diefenderfer et al. (2012) suggested the consistency of RAP used in mix design and field construction should be assured. They recommended performing RAP gradation quality control testing routinely before construction started. Muthen (1998) recommended the maximum plastic index and an ideal gradation range for the RAP used in the mix design. Different organizations including Asphalt Recycling and Reclaiming Association (ARRA), Wirtgen GmbH, and Asphalt Academy have recommended the RAP gradation ranges based on their experiences (as shown in Figure 2.3). The gradation band recommended by Asphalt Academy can be used for both cold recycling and FDR. The gradation bands from Wirtgen GmbH and ARRA are only suitable for cold recycling.

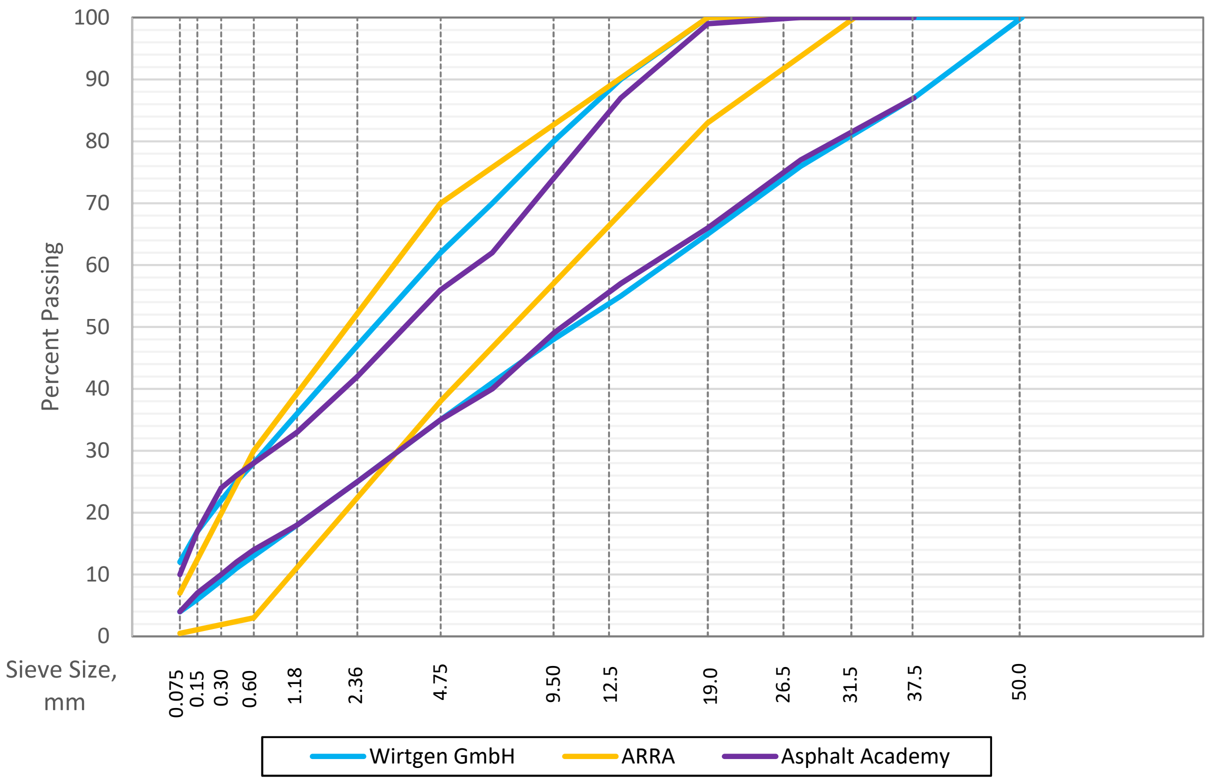


Figure 2.3 Gradations Recommended for Cold Recycled Foamed Asphalt Mixtures (Wirtgen GmbH 2012, ARRA 2016, Asphalt Academy 2009)

As shown in Table 2.1, different organizations used different gradation test methods. The gradation test method for RAP/aggregate suggested by Wirtgen (2012) was following ASTM D422 with minor change. The oven-drying temperature was set at 40°C below the softening point instead of at 110°C before and after washing to prevent RAP particles from sticking together (Shunmugam 2014). However, this standard was originally used for FDR material testing with soil in the mixture. Because this study focused on recycling RAP materials only which do not have much fines below #200 sieve, ASTM C136 and ASTM C117 suggested by ARRA (2016) were adopted to perform sieve analysis (ASTM C117 was used to determine the percent of fines passing #200 sieve).

Table 2.1 Summary of Different Gradation Test Methods

Organization	Gradation Test Method
ARRA (2016)	ASTM C117 & C136
Wirtgen (2012)	ASTM D422
Asphalt Academy (2009)	South Africa TMH1 (A1, B4)

It may not be enough to characterize RAP only by its overall gradation curve since RAP material has higher variability in binder content, aggregate type, and gradation. Other methods have been proposed to characterize RAP. Cross (2003) found 3:1 flat and elongated (F-E) particles percentage had significant impact on the compacted density. Fewer gyrations were required to achieve the target density as the F-E particles percentage increased. Perraton et al. (2015) proposed a method to distinguish RAP from different sources based on the percent passing a control sieve after applying compaction by the modified Proctor method. The RAP material was considered as particles conglomerated by the binder with potential to break apart during compaction. Temperature was found to have a significant effect on the percent passing control sieve (PCS). Results at a lower temperature (5°C) tended to have higher repeatability than results at a higher temperature (40°C). Results with aggregate samples were not temperature dependent as RAP. Additionally, as sieve size increases, the results of PCS had lower repeatability.

Moisture needs to be monitored and controlled in the cold recycling process. The moisture content in RAP/aggregate should be tested before mixing to determine how much additional moisture is needed. After mixing with foamed asphalt and mineral additive, the moisture content should be verified by QC testing which is typically determined using the oven-drying method. However, the current method for drying aggregate and hot mix asphalt (HMA) may not be suitable for RAP. Drying aggregate (or soil) requires heating at 110 ± 5°C (230 ± 9°F) overnight until moisture loss is below 0.1% between two measurements (ASTM D2216-10, AASHTO T255-00). For HMA, drying temperature is even higher at 163 ± 14 °C (325 ± 25 °F), and the sample is dried initially for 90 minutes and then measured every 30 minutes until no more than 0.05% moisture loss between two measurements (AASHTO T329-13).

RAP material coated with asphalt needs to be treated carefully as well. The oven temperature for drying RAP should be lower than the softening point because higher temperatures may soften the binder and causes particles to stick together (Shunmugam 2014). The Field Land Highway (FLH) addendum of

AASHTO T308 standard suggests drying RAP to constant weight at 60°C before determining binder content. Shunmugam (2014) suggested drying RAP at 40°C. The softening point is often used to evaluate the tendency of the material to flow at elevated temperatures (ASTM D36). Jones et al. (1993) summarized properties of various asphalt binders tested at five different laboratories (Western Research Institute, Pennsylvania State University, University of Texas at Austin, Asphalt Institute, and Michigan State University). They found PG70-22 binder had a softening point of 51.6°C (125°F), and the average softening point of PG64-22 binder was 48.7°C (119.6°F). Because typical asphalt binder used in Alabama is PG 67-22, the softening point of PG 67-22 binder should be around 50.2°C (122.3°F) by interpolation. A drying temperature below 50°C would be safe to prevent RAP particles from sticking together because the binder coated on RAP may be aged.

2.1.1.2 Prepare and Testing Foamed Asphalt

Csanyi (1957) proposed an asphalt foaming procedure by injecting steam into hot liquid binder. This method used an adjustable nozzle connected with a steam line and an asphalt binder line to produce foamed asphalt in the throat of nozzle above the tip. This system equipped with a boiler for generating the steam. Two types of foamed asphalt could be produced by adjusting the nozzle. One was named discrete foam and the other was named concentrated foam. The discrete foam produced at 60-90 psi steam pressure and 50-80 psi binder pressure was in the form of separated individual bubbles. While the concentrated foam was made at a much lower pressure and contained large joined bubbles that formed amass of foam when it was emitted from the nozzle. After years of development, modern asphalt foaming technologies introduce water and air into hot asphalt binder. When water vapor is trapped in asphalt, it generates bubbles and expands the asphalt volume. Then the foamed asphalt binder with reduced viscosity disperses easily among RAP particles (Muthen 1998, Diefenderfer et al. 2016).

Laboratory Foamer

In the laboratory, foamed asphalt is typically produced in a specialized foamer that can heat binder to a certain temperature and pressurize the binder, air, and water at a desired pressure. However, foamers manufactured by different companies may have significant differences regarding the working pressure, foaming process, and dispense method. Consequently, their foaming capacity may also be different. The InstroTek foamer produces foam by pressurizing binder and water. Foam is dispensed at a speed of 16-20 g per second through a long tube. Another foamer manufactured by Pavement Technology Incorporation (PTI) adds atomized water into binder which is dispensed by gravity. Dispense speed is lower at 14-20 g per second. The Wirtgen foamer produces foam in an expansion chamber where pressurized air and water are injected into the binder. Its dispense speed is faster than others (100 grams per second). Comparing these three types of laboratory foamers in terms of their basic parameters, foaming mechanism, foaming capability, and coating ability with heated aggregates, the Wirtgen foamer was found to be the best because it produced foam with best quality (Yin et al. 2015). A comparison between the InstroTek and the Wirtgen foamer was also conducted by Arega et al. (2014) based on the measurements of foaming characteristics. These two devices had different nozzle types and dispensing methods. To dispense 200 grams of foamed asphalt, the InstroTek foamer may need 10-12 seconds, while the Wirtgen foamer only needs 2 seconds. The dispensing speed agreed with the findings by Yin et al. (2015). The two foamers obtained a similar range of foaming properties (expansion ratio and half-life) while the InstroTek foamer had relatively lower expansion ratio. Because the measured expansion ratio decreased quickly in the first few seconds, the timing of measurement was also important.

Foaming Parameters

As discussed above, the foaming process is basically introducing water into hot asphalt binder so evaporated water causes expansion, reducing surface tension and the viscosity of asphalt binder. Foaming properties of binder are affected by many factors such as temperature, foaming water content, binder source, grade, and binder additives. Several indices have been proposed to evaluate the foaming characteristics such as the expansion ratio, half-life (HL), foam index, and surface area index. These characteristics were used to determine the binder temperature and the foaming water content. The most commonly used indices are the ER and HL. The value of ER is determined as the ratio between the maximum volume and the original volume of binder. The HL is the time duration for the foamed binder to collapse into half of its maximum volume, indicting the stability of foamed asphalt (Wirtgen GmbH 2012, TG2). Higher ER and longer HL were considered to ensure the asphalt with good foaming properties that can provide better coating for the granular materials (Yin et al. 2016). These two indices are inversely related, which means increasing the ER will decrease the HL. The selected foaming water content optimizes both indices. Figure 2.4 shows the method used to determine the optimum foaming water content based on the ER and HL. This is the most common method used by several organizations (Asphalt Academy 2009, ARRA 2016, Wirtgen 2012).

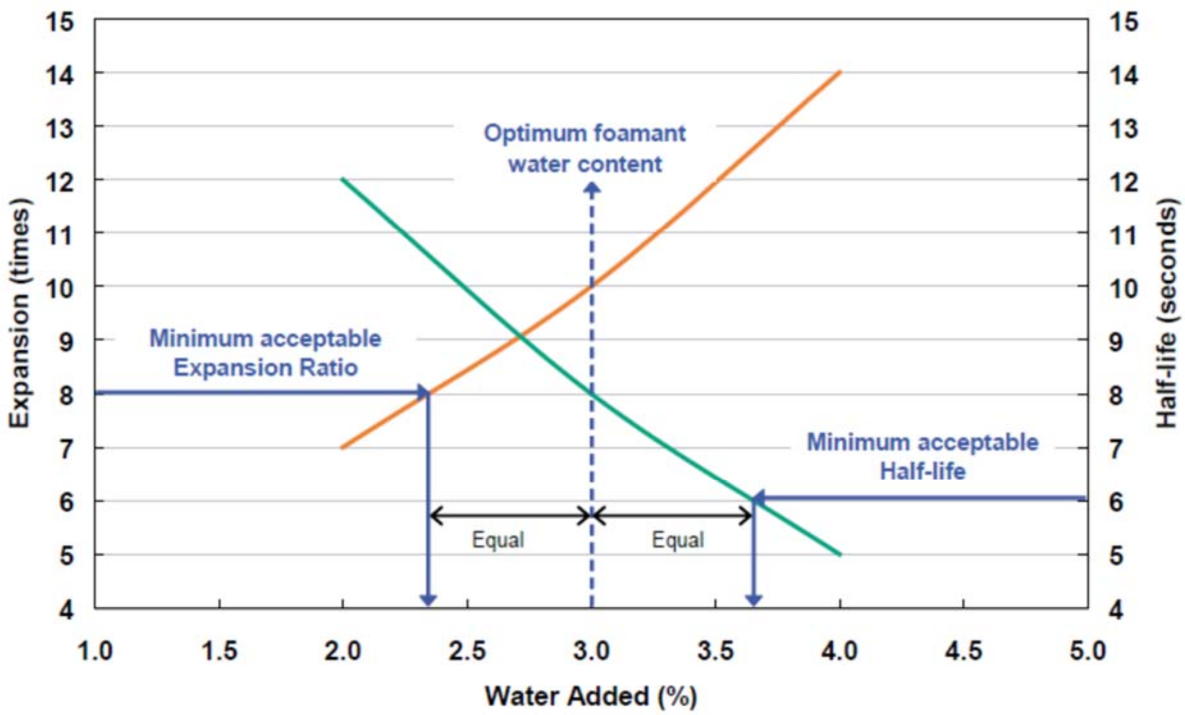


Figure 2.4 Determination of Optimum Foaming Water Content Based on ER and HL (Asphalt Academy 2009)

As shown in Figure 2.4, the minimum ER and HL values were used as criteria to find the range of foaming water content, the mid-point of which was determined as the optimum foaming water content. Different ER and HL criteria have been proposed in previous studies based on the literature review. Table 2.2 summarizes some of these criteria.

Table 2.2 Summary of the ER and HL Requirements for Foamed Asphalt

Organization/Authors	Min. Expansion Ratio	Min. Half-Life (seconds)
Wirtgen GmbH (2012), ARRA (2016)	8	6
Asphalt Academy[1] (2009)	8 or 10	6
Kuna et al. (2014)	10	6
Eller and Olson (2009)	10	10
Muthen (1998)	10	12

Note: 1. Expansion ratio of 8 was recommended for aggregates above 25 °C, while a ratio of 10 was recommended for the aggregates at 10-25 °C.

These two indices must be obtained right after foamed asphalt is produced. They can be quick feedbacks to the designer for the foaming properties of asphalt and an approximate range of the optimum foaming water content. However, these two empirical indices rely mostly on the measurements of height of foam on a dipstick and the foam decay time. The accuracy of such measurements is also dependent on the operator's experience. Other than ER and HL, the foam index (FI) was proposed as a composite foaming parameter combining values of the ER and HL. It was determined based on the ER decay curve. Basically, higher FI indicates higher foaming properties (Jenkins 1999). Yin et al. (2016) proposed a surface area index (SAI) to characterize the foaming property. It was calculated as the ratio between total surface area of foam asphalt bubbles at 60 second and the surface area of the original binder. Basically, foamed asphalt with higher SAI had more surface and provides better coating on the heated aggregate.

Arega et al. (2014) investigated foaming properties using laser measurement of expansion. They proposed the rate of collapse (k) of foamed asphalt at semi-stable stage a few seconds after foaming may better indicate foaming characteristics than the ER value, because it is determined in an unstable condition when large bubbles collapse at a very fast rate right after foaming. The semi-stable stage starts after 10 seconds and lasts for minutes. Because more stable bubbles gradually rise and collapse at this stage, it may be more representative of the foaming process. They also studied the effects of binder PG, binder source, and foaming water content on the characteristics of foamed asphalt. The results showed that binders with different PG tended to have different ER and k. Even at the same PG, binders from different sources had different foaming characteristics. Foamed asphalt with higher foaming water content expanded much more initially but collapsed faster in the semi-stable condition.

Foaming characterization is to determine the optimum foaming water content added in the asphalt. The methods summarized above are effective and quick solutions based on observation but the mechanical properties of foamed asphalt were not investigated. To better understand the foaming mechanism, some studies have focused on two aspects of mechanical properties: surface tension of foam bubbles and viscosity after foaming. Arega et al. (2015) studied the influence of bubble surface tension on the binder's foaming property using a bubble pressure device. The calculated results showed that surface tension did not significantly affect bubble size distribution and the theoretical maximum expansion ratio (ER). Increase of surface tension caused by more foaming water, decreased the effective water content for foaming but increased the bubble's collapse rate (k-value). Temperature reduction on the foam was also detected as surface tension increased. Instead of surface tension, Saleh (2007) proposed a new method to evaluate the asphalt foaming properties using the viscosity. Average viscosity by Brookfield rotational viscometer in 60 seconds after foaming was found to better explain and distinguish the foaming properties of different binder than using empirical methods such as the ER and HL, or the FI. The conclusion was based on the measurement of nine different asphalt binders from seven different sources. Wirtgen's WLB10s foamer was used to foam these binders. He suggested a method to determine the optimum foaming water content corresponding to the lowest viscosity. Yu et al. (2013) calculated the Zero Shear Viscosity (ZSV) of foamed asphalt binder with 0, 1, 2, and 3% water content. For the unmodified binder, the ZSV value increased at 1% water and gradually decreased as more water was added. All the foamed asphalt with more than 1% water shows higher ZSV value than those of asphalt with 0% water. For the modified binder, the ZSV value monotonically increased as water content increased.

The foamed asphalt properties directly affect the distribution of the foamed asphalt in the cold recycled mixture and how the strength is developed in the mixture. Therefore, the foamed asphalt properties may affect the performance of cold recycled mixtures. However, correlation between the foamed asphalt properties and field performance has not been investigated based on the literature review.

Asphalt Binder used to Produce Foamed Asphalt in Cold Recycling

Table 2.3 summarizes the asphalt binders used to produce foamed asphalt in cold recycling from twelve studies in ten states. These binders were sorted into two categories. Most binders used for foaming are the typical binders in those regions. Softer binders rather than the typical were more commonly used in the northern states. This may be because these softer binders can better react with the RAP binder to improve the over-all mix properties slowly in colder regions.

Table 2.3 Binders Used to Produce Foamed Asphalt in Cold Recycling

Category of Binder Used for Foaming	Organization/Author	State	Binder PG
Typical Binder in the Region	Timm et al. (2014), Ma et al. (2017)	AL	PG 67-22
	Chen et al. (2006)	TX	PG 64-22
	Fu et al. (2011)	CA	PG 64-16
	Diefenderfer and Apeagyei (2015)	VA	PG 64-22
	Thomas (2000)	KS	PG 64-22
	Schwartz and Khosravifar (2013)	MD	PG 64-22
Softer Binder than the Regional Typical	Mohammad et al. (2003, 2013)	LA	PG 58-22, PG 58-28
	WIDOT	WI	PG 46-34, PG 52-34
	Kim and Lee (2006)	IA	PG 52-34
	Eller and Olson (2009)	MN	PG 52-34

2.1.2 Mixing and Compaction

2.1.2.1 Laboratory Mixing

Mixing and compaction procedure directly affect the quality of cold recycled foamed asphalt mixtures. During the mixing procedure, the equipment and the time are the two major factors. Three commonly used mixers include the blender-type, bucket-type, and twin-shaft pug mill. Appropriate mixing time was recommended between 20 and 60 seconds. Table 2.4 summarizes the mixing procedures applied by different organizations and researchers.

Table 2.4 Summary of Laboratory Mixing Procedures

Organization/Authors	Mixing equipment	First mixing[1]	Second Mixing[2]	Mixing Temperature
Eller and Olson (2009)	blender type mixer	40-60 s	60 s	Room temperature
Asphalt Academy (2009)	Pug mill mixer	—	20-30 s	Room temperature
Kuna et al. (2014)	Twin-shaft pug mill mixer	—	60 s	20 ± 2 °C
Wirtgen GmbH (2008)	Twin-shaft pug mill mixer	30 s	60 s	Room temperature
Wirtgen GmbH (2012)	Twin-shaft pug mill mixer	>10 s	30 s or until uniform	Room temperature

Notes:

1. The first mixing occurs after water and cement are added but before foamed asphalt is introduced.
2. The second mixing occurs after foamed asphalt is added.

Asphalt Academy (2009) suggested that different mixers can cause up to 25% difference between mixtures' strengths. A laboratory pug mill mixer was recommended rather than the blender-type mixer because it better simulated mixing process in the field or plant. Because the efficiency of mixing between laboratory and field plant, mixing time in the laboratory pug mill needs to be longer than field mixing. Typical mixing time in the laboratory was suggested as 20-30 seconds.

Wirtgen GmbH (2012) recommended adding the ideal mixing water content (75% RAP material's OMC by the modified Proctor test) and mixing the RAP and/or aggregate until the "fluffed" state or without visible dust. Additional water and mixing time may be required in this step. However, if there is too much water, the mix should be rejected. At this step, mixing was recommended at least 10 seconds before adding foamed asphalt. Further mixing for 30 seconds after adding foamed asphalt was suggested. Finally, the compaction water (25% RAP material's OMC) should be added and mix until uniform.

2.1.2.2 Laboratory Compaction

Many different methods have been used to compact cold recycled foamed asphalt mixtures in the laboratory. Based on the literature review, the methods include the modified proctor test method, Marshall hammer compaction, gyratory compaction method, and the vibratory method as shown in Table 2.5. The gyratory and Marshall compaction methods were more common. The Marshall compaction method of 75 blows was widely accepted while the method using gyratory compactor had no agreement on the compactive effort. Kim et al. (2007) found specimens compacted by 30 gyrations using the Superpave gyratory compactor (SGC) and those compacted by 75 blows of Marshall hammer had similar densities. They recommended the SGC method in the mix design procedure rather than Marshall hammer because the SGC compaction could obtain clear peak ITS result at the optimum foamed asphalt content and the compacted density was more sensitive to the change of foamed asphalt content, compared to the Marshall compaction

Table 2.5 Summary of Compaction Procedure

Compactor	Organizations/Authors	Compactive effort	Specific Parameters
Gyratory	Lee and Kim (2003)	25 gyrations	600 kPa pressure and 1.25° angle; 100-mm mold
	[1]Cox and Howard (2016); [2]Kim et al. (2007); [2]ARRA (2016)	30 gyrations	600 kPa pressure and 1.25° angle; 150-mm or 100-mm mold
	Maccarrone (1995)	85 or 120 gyrations depends on pavement layer thickness	240 kPa pressure and 2° angle; 100-mm mold
	Brennen et al. (1983)	20 gyrations	1,380 kPa pressure and 1.25° angle; 100-mm mold
Marshall	Kim et al. (2007); Muthen (1998); Brennen et al. (1983); Wirtgen (2012); ARRA (2016)	75 blows each side	10 lbs. hammer, 18 in. drop; 100-mm mold
Modified Proctor Hammer	Wirtgen (2012)	55 blows each layer	10 lbs. hammer, 18 in. drop; 4 layers; 150 mm mold;
Vibratory	Asphalt Academy (2009)	10 to 25 second per layer depends on traffic	100-mm or 150-mm mold, hammer weight 5 or 10 kg; 1 or 2 layers depends on traffic

Note:
1. Compacting 30 gyrations in a 150-mm mold.
2. Compaction 30 gyrations in a 100-mm mold.

2.1.2.3 Laboratory vs. Field Compaction

The laboratory compactive effort used in mix design should match the field compactive effort. However, there were few studies comparing the two compactive efforts. Compacted density was typically used to compare the different efforts.

Diefenderfer et al. (2012) summarized a pavement recycling project on I-81 in Virginia and compared the densities and the ITS results of the laboratory- and field-compacted specimens. Cold recycled foamed asphalt mixtures used in the CIR and CCPR base layers were designed following Wirtgen's method. Specimens were compacted using the Marshall hammer for 75 blows to the dry density of 125 pound per cubic foot (pcf). They measured field density by nuclear density gauge (direct transmission method) showed that the average field density was 127.4 pound per cubic foot (pcf), which was a little higher than the target field density of 122.5 pcf (98% of laboratory dry density). The QC test results showed most specimens passed the minimum ITS criterion, which was 48.5 psi (95% of mix design ITS). Additionally, field cores were taken 3 months after construction to measure density and other properties (Diefenderfer and Apeagyei 2014). The average density was ranged from 128.6 to 141.8 pcf, significantly higher than the mix design density (125 pcf). They also found CIR and CCPR sections had the same densities with no significant difference, but the top-half of the cores had significantly higher density than the bottom-half.

Jiang et al. (2011) from Jiangsu Institute of Transportation in China compared two laboratory compaction methods with the field compactive effort in a cold recycled foamed asphalt pavement rehabilitation project on highway G328. The two laboratory compaction methods were 30 gyrations using the SGC and 75 blows each side using the Marshall Hammer. Laboratory-compacted specimens were cured at 40°C for 3 days before testing. Density, ITS, and Marshall stability were used for comparison. They found field-compacted density (measured from cores taken after construction) was close to gyratory-compacted density. Marshall hammer-compacted density was a little lower than gyratory - compacted and field-compacted densities. The bulk specific gravities were 2.232 in the field, 2.210 for gyratory-compactor, and 2.149 for Marshall hammer. In comparisons of ITS and Marshall stability, the gyratory-compacted specimens had higher dry and wet ITS results than the Marshall hammer-compacted specimens and field cores. But the specimens compacted by both laboratory methods had similar stability. Their stability results were higher than the stability results of the field cores.

Ceccovilli (2007) summarized several CIR emulsion stabilization projects in the U.S. Specimens were compacted using the SGC for 30 gyrations. An example of field compactive effort was documented as 1 pass of a static roller, 7 passes of a pneumatic roller, and 3 passes of a vibratory roller. However, the types of rollers were not specified. Air void contents in the laboratory and field were found similar in nine different projects.

Schwartz and Khosravifar (2013) compared the modified Proctor compactive effort with field compactive effort in an evaluation of cold recycled foamed asphalt base materials containing 2.2%

foamed asphalt and only RAP without aggregate. In the laboratory, a standard Proctor hammer was used to compact specimens until the modified Proctor compactive effort (2,700 N-m/m^3) was achieved. They found that bulk density of laboratory-compacted specimens was only 119.2 pcf, much less than the density of field cores (134.8 pcf).

In 2006, Wirtgen GmbH applied a cold central-plant recycling (CCPR) technology for XiBao Expressway rehabilitation in China. The RAP/aggregate blends contained 70% RAP, 28% percent new aggregate, and 2% cement. About 2.5% foamed asphalt was used as a recycling agent. Water was added at 3.0 - 3.5% to aid mixing and compaction. The Marshall hammer (75 blows per side) was used to compact specimens for ITS test. The field compaction pattern was recorded as 1 static pass using Hamm HD130, followed by 3 - 4 vibratory passes using the same machine at a lower amplitude, and ending with 8-10 passes using a Hamm HD150 TT pneumatic roller. Exact laboratory and field densities were not documented, but the degree of compaction was in the range of 98.6 – 101.1 percent (Yu, 2005).

Some studies of compaction focused on the cold recycling using asphalt emulsions. Although foamed asphalt and asphalt emulsion are categorized as different recycling agents, the findings of emulsion compaction studies may be helpful in studying the compaction of cold recycling with foamed asphalt. Cross (2003) studied seven cold recycled asphalt emulsion projects and found samples compacted by 32 gyrations using the SGC achieved the target density (103% of field density) and 12% voids in total mix (VTM). Therefore, he recommended 30 gyrations for compacting cold recycled asphalt emulsion mixtures right after mixing. He also recommended a little higher compactive effort of 35 gyrations to compact the mixture to target density after initial curing (after the emulsion breaks). The current standard compaction procedure (ASTM D7229) for cold recycling mix design is based on using asphalt emulsion as a recycling agent and dense graded RAP/aggregate. ASTM D7229 specifies 33 ± 3 using the SGC at 600 ± 20 kPa pressure as the compactive effort and mentions the specimen's compacted density tends to be constant after 40 gyrations.

Swiertz et al. (2012) proposed a compaction method for cold recycled asphalt emulsion mixture using the modified SGC. They drilled holes on the side of the mold and used a bottom plate on the compactor to collect water during compaction. Compaction pressure was set as 300 kPa instead of 600 kPa. The number of gyrations was determined as 75 but this laboratory compactive effort was not compared with the compactive effort in the field.

2.1.3 Laboratory Curing

2.1.3.1 Lab vs. Field Curing

Maccarrone (1995) investigated curing condition of cold recycled foamed asphalt mixtures and found that mixtures cured at 60°C for 3 days and cured in the field for 12 months had similar moduli. Ruckel et al. (1983) also compared three accelerated laboratory curing methods with field curing conditions in dry and moderate climates. After mixing and compaction, specimens were left in the mold at room temperature for 24 hours before extrusion to avoid potential damage. This short-term curing was similar to the 1-day field curing in terms of strength and stability. After extrusion, a portion of these specimens were continued curing for 1 day at 40°C; this intermediate-term curing simulated curing conditions for 7 – 14 days in the field. Finally, some of these samples were cured for 2 additional days (a total of 3 days after extrusion) at 40°C. This long-term curing was considered to simulate field curing conditions for 30 - 200 days. Table 2.6 summarizes the curing methods for the cold recycled foamed asphalt mixtures from some recent studies. The most commonly used method is curing at 40°C for 3 days.

Table 2.6 Summary of Curing Procedures in Cold Recycled Foamed Asphalt Mix Design

Organization/Authors	**Curing methods**
Muthen (1998)	3 days at 60°C
Kim and Lee (2006); Kim et al. (2007); Eller and Olson (2009); ARRA (2016)	3 days at 40°C
Kuna et al. (2014)	in mold for 1 day at room temperature + 3-5 days at 40°C
Iwanski and Kowalska (2013)	in mold for 1 day + 3 days at 40°C
Asphalt Academy (2009)	For level-1 design (<3 million ESALs): 3 days at 40°C; For level-2 and level-3 design (>3 million ESALs): 20 hours at 30°C unsealed + 2 days at 40°C sealed
Wirtgen GmbH (2012)[1]	3 days at 40°C

Note: 1. Before curing, mixtures with coarse RAP/aggregate were left in mold for 1 day at room temperature.

2.1.3.2 Effect of Curing Methods on Laboratory Testing Results

Roberts et al. (1984) found that curing temperature had more influence on the wet strength than the dry strength. Curing at 60°C accelerated the strength development much more than curing at lower temperatures (35 or 24°C). The strengths of specimens cured at 60°C were almost twice as much as those of specimens cured at the lower temperatures after 7 days. After 14 days' curing at 24°C, when specimens were moved to 60°C condition for additional 7 days, the strength increases were still small and the final strengths were much lower than those conditioned at 60 °C for 7 days. Maccarrone (1995) studied the effect of curing time on laboratory testing results and found that most of the resilient modulus was developed in the first 7-day curing, although there was a small increase until 30 days. Kim et al. (2007) found that specimens cured at 40 °C for 3 days yielded lower ITS results than specimens cured at 60 °C for 2 days. They suggested 40 °C for 3 days because this temperature was closer to the field condition of the cold recycled base. Cardone et al. (2015) proposed a two-stage curing theory for cold recycled asphalt emulsion mixture with cement. They found that moisture loss by evaporation in the initial stage controlled the increase of modulus. But excessive moisture loss hindered the hydration effect, and therefore, may limit the development of modulus.

Kim et al. (2010) studied the effect of curing time and moisture content on the CIR mixture's properties. Curing parameters selected were initial curing time in air, oven-curing temperature, and oven-curing time. They found that the cured specimens with lower moisture content had higher ITS results, dynamic moduli, and Flow Number results. They also found that the ITS result increased slowly during the early curing stage before the moisture content dropped below 1.5%.

Ruckel et al. (1983) investigated curing methods for cold recycled foamed asphalt mixtures. First, they studied the effect of curing temperature and in-or-out of mold curing on the moisture loss and strength development. Specimen cured faster at 60°C than at room temperature. In-mold curing slowed down the moisture loss at room temperature but the difference between in- or out-of-mold curing became insignificant when curing at 60 °C. However, curing in-mold significantly reduced Marshall stability whether it was cured at room temperature or at 60°C.

2.1.4 Volumetric and Strength Testing

2.1.4.1 Volumetric Properties Tests

The volumetric properties are often measured before conducting strength test. Saleh (2006) tested the volumetric properties of cold recycled foamed asphalt mixture in accordance with AS2891.8-1993 standard. He found that the air void contents (V_a) of foamed asphalt mixtures with aggregate were between 6 and 10% which was about twice the air void content of compacted HMA specimens. The voids filled with asphalt (VFA) was ranged from 41.7 to 44.2%, much lower than VFA of the HMA specimens (about 65 – 75%). The voids in mineral aggregate (VMA) of cold recycled foamed asphalt mixture was around 15%. Kim et al. (2007) found the air void content decreased as more foamed asphalt content increased in the cold recycled foamed asphalt mixture. Air void content was determined based on measured bulk specific gravity and the theoretical maximum specific gravity. In the literature, most studies did not specify the methods used for determining the volumetric properties. In a recent study conducted by Cox and Howard (2016), an automatic vacuum-sealing method (AASHTO T331) was suggested to measure bulk specific gravity (G_{mb}) because the air void content in cold recycled asphalt specimens was typically greater than 9%. Table 2.7 lists different test methods used for measuring volumetric properties for cold recycled mixes.

Table 2.7 Summary of Volumetric Properties Determining Methods

Organization/Authors	**G_{mb} or bulk density**	**G_{mm}**
Cox and Howard (2016)	AASHTO T331	ASTM D6857
ARRA (2016)	AASHTO T166	AASHTO T 209
Asphalt Academy (2009)	South Africa TMH1-B14	—
Wirtgen GmbH (2012)	Calculated by dry weight and dimensions	—

2.1.4.2 Strength Tests

Strength tests for cold recycled foamed asphalt are typically used for determining the optimum foamed asphalt content. The most common strength test method for cold recycled foamed asphalt mix design is the ITS test. The ratio of test results in wet and dry condition is known as tensile strength ratio (TSR) and is also used in design criteria. The ITS criteria are established according to a specific curing condition. Table 2.8 summarizes the design criteria for ITS used by different organizations (ARRA, Asphalt

Academy, and Wirtgen GmbH). These values are applicable for compacted specimens cured for 3 days at 40°C.

Table 2.8 Summary of Mix Design Criteria

Organization	ITS (dry)	ITS (wet)	TSR
ARRA (2016)	> 310 kPa (45 psi)	—	> 0.70
Asphalt Academy (2009); Wirtgen GmbH (2012)	> 225 kPa (32.6 psi)	ITS (wet) > 100 kPa (14.5 psi)	> 0.50 (Wirtgen)

Kim and Lee (2006) tested the strengths of the cold recycled foamed asphalt mixtures. After 3 days' curing at 40°C, both dry- and wet-conditioned specimens were tested for Marshall stability and the ITS. They suggested using the wet ITS for determining the optimum foamed asphalt content (FAC) because the cold recycled mixes had high moisture susceptibility. Also, this test was more sensitive to the FAC than the Marshall stability test. In other words, the peak ITS was easier to obtain than the peak stability.

Saleh (2007) found that cold recycled foamed asphalt mixtures had lower ITS results compared to HMA and the effect of mineral filler was significant on the ITS. Iwanski and Chomicz-Kowalska (2013) studied the effect of FAC on the ITS and Marshall stability. They found that stability increased as more foamed asphalt was used, but it started to decrease when FAC was above 2.5%. The ITS results increased monotonically when FAC increased. Eller and Olson (2009) suggested conditioning methods for the dry and wet ITS tests. The dry ITS specimens were kept in an oven at 25°C for 2 hours and the wet ITS test specimens were also conditioned at 25°C but subjected to a series of treatment including 20-minute soaking, 50-minutes vacuum saturation (at 50-mm Hg), and a 10-minute soaking without vacuum. He and Wong (2008) suggested using the peak wet ITS to determine optimum FAC in a range of 1 - 5% for the cold recycled mixture.

2.1.5 The optimum foamed asphalt content and total water content

Foamed asphalt content (FAC) and total water content (TWC) are the two most important factors in mix design. Foamed asphalt provides bonding between particles. Water helps mixing by breaking up agglomerations and distributing foamed asphalt and facilitates compaction by reducing friction (Csanyi, 1956). Targets of the cold recycle foamed asphalt mix design are the optimum foamed asphalt content (OFAC) and the optimum total water content (OTWC). These two targets can be determined simultaneously or separately.

2.1.5.1 Determine the OFAC and OTWC simultaneously

When optimum FAC and TWC were determined simultaneously, specimens were fabricated with different combinations of FAC and TWC levels. The optimum FAC and TWC can be determined at the peak density or strength (Roberts et al. 1984, Kim and Lee 2006, Saleh 2004, and Saleh 2006).

Roberts et al. (1984) found that the OFAC and the OTWC could be combined together as the optimum total fluid content (total fluid content including both foamed asphalt and water). In both dry and wet ITS test, the total fluid content (FAC + TWC) to achieve the peak ITS results were about the same. For example, the strength of specimens with 0.5% water and 1.0% foamed asphalt was equivalent to those with 1.0% water and 0.5% foamed asphalt. Kim and Lee (2006) proposed a method to select OFAC and OTWC using a partial factorial experiment design. There were thirteen combinations with five FAC and four TWC for totally three RAP gradations. They found the OFAC, corresponded with the peak density, the ITS, and the Marshall stability. The OTWC was found 0.5 – 1.0% lower than the RAP material's OWC result.

Saleh (2004) studied the OFAC and OTWC based on the contour plot of resilient modulus and bulk density at different combinations of FAC and moisture content. Two groups of aggregates with identical gradations but different fine fractions (passing 0.075 mm sieve) were used for recycling. The OMC of both aggregates were the same at 6.0%. But after mixing with foamed asphalt, one group had OTWC equals 120% of the OMC, while the other group had OTWC equals 100% of the OMC. In another study conducted by Saleh (2006), the same contour plot of resilient modulus was applied to determine the OFAC and OTWC. He found the asphalt binder source had little effect on the two design targets. But the peak resilient modulus was observed at the same total liquid content (about 10.2%) for most of the cold recycled mixes.

2.1.5.2 Determine the OFAC and OTWC separately

When FAC and TWC are considered independently in mix design, OTWC in the mixture is often determined first, followed by the OFAC. The OTWC is determined based on its relationship with the

RAP material's OWC. Then, the OFAC is determined corresponding to the peak strength (or density) at the determined OTWC. Sakr and Manke (1985) proposed a regression model (as shown in Equation 2.1) to determine the OTWC using factors such as OMC of fine aggregate, percent of fines passing No.200 sieve (PF), and foamed asphalt content (AC). Fine aggregate was compacted by the standard Proctor hammer. The specimens were molded by the Hveem gyratory-shear compaction method.

$$\mathrm{OTWC} = 8.92 + 1.48 \times \mathrm{OMC} + 0.40 \times \mathrm{PF} - 0.39 \times \mathrm{AC} \tag{2.1}$$

Where:
OTWC = optimum total water content in mixture (%);
OMC = optimum moisture content of fine aggregate (%);
PF = percent of fines passing No.200 sieve (%); and
AC = asphalt content (%).

Maccarrone (1995) added water to RAP/aggregate at 100% optimum water content (OWC) before foamed asphalt was introduced. This pre-wetting procedure was considered to help dispersion of foamed asphalt and assist compaction. Muthen (1998) suggested using the modified Proctor test to determine the OMC of aggregate and reducing the OMC by FAC as the OTWC. Kim et al. (2007) studied the OFAC of seven different RAP sources assuming the same OTWC. The OFAC was determined in a range of 1.5 - 2.5% corresponding to the peak ITS result and the peak Marshall Stability. They found the stiffness of RAP binder correlated well with the OFAC. Basically, RAP material with stiffer residual binder required a higher OFAC. Also, the effect of RAP binder content was not significant. And the RAP materials with coarser gradations tended to have lower OFAC results. Eller and Olson (2009) summarized cold recycled foamed asphalt mix design practice in Minnesota and proposed a concise design method. They pointed out the OTWC of the foamed asphalt mixture should be a little less than the RAP material's OWC because foamed asphalt has some lubricating effect. However, the compactive effort for determining the OWC was not specified. Wirtgen GmbH (2012) suggested to add 100% of RAP material's OWC determined by the Modified Proctor as the level of OTWC for mixture. ARRA (2016) recommended the OTWC for the mixture as 100% RAP material's OWC but also stated that 75% of the RAP material's OWC may be used if there was excessive water extruded from the SGC mold during compaction. Kuna et al. (2014) proposed a mix design method using the specimens compacted in the gyratory compactor. In the study, the OTWC was determined using the dry and wet ITS results, as well as the ITS modulus. The OTWC was found to be 75-85% of RAP/aggregate's OMC determined by the modified Proctor test.

2.2 Mineral Additives

In addition to RAP, foamed asphalt or asphalt emulsion, and water, it is common to add a mineral filler component to enhance its strength and reduce moisture damage susceptibility. The most common mineral additive in cold recycling is Portland cement, although some organizations also suggested hydrated lime as alternative additive (Maccarrone 1995, Asphalt Academy 2009, Wirtgen GmbH 2012). Fly ash and limestone fines have also been mentioned as potential additives (Eller and Olson 2009).

Cement and hydrated lime are commonly used at up to 1.5% of the mixture. A few studies have compared different additives and suggested the optimum additive content for cold recycled asphalt mixtures. Nosetti et al. (2016) compared the effects of cement and hydrated lime on the indirect tensile strength (ITS) of cold recycled mixtures using foamed asphalt and emulsified asphalt as recycling agents. They found mixtures with 0.5% of additives had much higher wet ITS and tensile strength ratio (TSR) than those with no additive. But improvements in ITS and TSR were limited when additive content increased from 0.5% to 2.0%. Mixtures with hydrated lime had equivalent ITS and TSR as the mixtures with cement. Thanaya et al. (2009) pointed out that adding 1 - 2% cement significantly increases early-age and long-term strengths of cold recycled emulsified asphalt mixtures.

Use of fly ash in cold recycled mixtures is limited but it is still a potential alternative because of its lower cost. Cross and Young (1997) verified the improvement in the moisture damage resistance of cold recycled emulsified asphalt mixtures by using fly ash. Fly ash production across the U.S. in 2015 was about 40 million metric tons with only 50% used (American Coal Ash Association 2017). Available survey data shows the price of fly ash was $74 per metric ton in 2013, about 33% lower than the price of cement ($111), and 49% lower than the price of hydrated lime ($144) based on the data of 2016 (Heavey et al. 2015, Statista 2017, US Geological Survey 2017). Another alternative additive is asphalt plant baghouse fines. The mineralogical composition of baghouse fines will depend on the sources of aggregates used in the asphalt plant. Eller and Olson (2009) suggested limestone fines as a mineral filler for cold recycled mixtures. Most studies on cold recycling additives were based on test results obtained after laboratory curing.

Table 2.9 summarizes the recommendations of additives used in the cold recycled foamed asphalt mixture from some organizations and researchers. Cement was suggested more frequently than hydrated lime or any other type of mineral additive. Fly ash and limestone fines have been mentioned as potential additives. Use of baghouse fines in cold recycled foamed asphalt mixtures was not found in the literature.

Table 2.9 Summary of Additives in Pavement Cold Recycling

Organization/Authors (year)	Cement	Hydrated Lime	Fly Ash	Baghouse fines	Limestone fines
Asphalt Academy (2009)	≤1.0% and less than asphalt content	≤1.5%	Can be used but no recommended amount	—	—
Wirtgen (2012)	≤1.0%	≤1.5%	Can be used but no recommended amount	—	—
ARRA (2014, 2016)	≤1.0% and minimum 3:1 asphalt to cement ratio	—	—	—	—
VDOT (2016)	Can be used but no recommended amount	Can be used but no recommended amount	—	—	—
Eller and Olson (2009)	≤1.5%	≤1.5%	≤1.5%	—	Can be used but no recommended amount
Saleh (2006), Saleh (2007)	1 and 2%	Used as mineral filler	C-ash was used as mineral filler	—	—
Cross and Young (1997)	—	—	≥ 7%	—	—

Note: 1. All the dash sign indicates the additive is not mentioned in the organization's specification.

2.3 Laboratory Testing

Several laboratory testing methods have been used to characterize cold recycled foamed asphalt mixtures. Table 2.10 summarizes these test methods except for the ITS test and the Marshall Stability test. Discussion on the testing parameters of some methods follows.

Table 2.10 Laboratory Testing Methods for Cold Recycled Mixtures

Authors (year)	Resilient modulus	Dynamic modulus	Dynamic creep test	FN/RLPD	Wheel tracking test	Raveling Test	Fracture Energy	Nottingham fatigue	Monotonic triaxial tests
Muthen (1998)	✓		✓						
He and Wong (2008			✓						
Saleh (2007)	✓						✓	✓	
Kim et al. (2009)		✓		✓					
Iwanski and Kowalska (2013)	✓								
Thomas and Kadrmas (2007)						✓			
Diefenderfer and Link (2014)		✓							
Diefenderfer and Apeagyei (2014)	✓	✓		✓					
Khosravifar et al. 2015		✓		✓					
Cardone et al. (2015)		✓							
Diefenderfer et al. (2016)		✓							
Cox and Howard (2016)	✓				✓		✓		
Jenkins et al. (2007)									✓

2.3.1 Resilient Modulus and Dynamic Modulus

Diefenderfer and Apeagyei (2014) conducted several mechanical tests on the field cores obtained from I-81 cold recycling bases layers. They found that resilient moduli from CIR and CCPR sections were statistically the same. Saleh (2007) used the resilient modulus to evaluate the temperature sensitivity of cold recycled foamed asphalt mixture. As temperature increased, the resilient modulus of cold recycled mixtures had a smaller decrease than HMA. Also, the cold recycled foamed asphalt mixtures were found to have lower fracture energy and fatigue life compared to HMA. Adding mineral filler may have improvements on these performances.

Some studies focused on the change of resilient modulus subjected to the repeated loading repetitions. Loizos et al. (2007) found the backcalculated modulus based on the FWD testing results increased under heavy traffic loading on a Greece highway. Timm et al. (2015) found the backcalculated modulus virtually showed no change under heavy traffic loading at the NCAT Pavement Test Track.

Cardone et al. (2015) studied the effect of curing methods on dynamic modulus of cold recycled mixture with cement. They found that the dynamic modulus of cold recycled mixtures can be plotted on the same curve at reference temperature by shifting the tested modulus results. The shift factor was related to the loading frequency. They also pointed out that modulus may not increase if excessive moisture evaporates after initial curing period or too little moisture was left for hydration of cement.

Diefenderfer and Link (2014) tested the dynamic modulus of CCPR foamed asphalt mixtures placed on three sections at the NCAT Pavement Test Track. Dynamic modulus test specimens were cored and trimmed from 6-inch diameter cylindrical samples compacted by SGC. Testing was conducted in an asphalt mixture performance tester (AMPT) in accordance with AASHTO TP79 but without testing at the high temperature (60°C). Four temperatures (4.4, 21.1, 37.8, and 54.4 °C), four frequencies (0.1, 1, 10, 25 Hz), and four confinements (0, 34.5, 68.9, and 137.9 kPa) were selected as parameters. Five replicates were used for each testing condition. They found that the effect of temperature on dynamic modulus was significant. The effect of confinement on moduli was significant at higher temperatures and lower frequencies.

Diefenderfer et al. (2016) performed dynamic modulus testing on small-scale specimens obtained from the field cores of 24 projects across the U.S. and Canada. Different cold recycling production methods (CIR, CCPR, and Full-depth reclamation [FDR]), recycling agents (emulsified and foamed asphalt), and mineral additives (cement and lime) were compared based on the trimmed test results at 10 Hz. The three different recycling production methods were statistically the same at low temperature (4.4 °C) but became different when temperature was intermediate (21.1°C) or high (37.8°C). Although the CIR and CCPR mixtures' dynamic moduli had similar temperature dependency, the CIR specimens had higher moduli than the CCPR specimens. This finding agreed with their previous findings on I-81 that the CIR mixtures had higher moduli than the CCPR mixtures at lower temperatures (Diefenderfer and Apeagyei 2014). With cement added, the moduli of foamed asphalt and emulsified asphalt mixtures were statistically the same. Master curves' envelopes of the mixtures with different recycling agents were mostly overlapping with only a little difference at lower and higher temperatures. Moduli of specimens with additives (lime and cement) had less temperature dependency than those without additives. Specimens with cement had higher moduli than those with lime at high temperatures. But at low temperatures, the samples with lime showed higher stiffness. Because the specific mix design information was not provided, conclusions of these comparisons may not apply to other mixtures with different mix designs.

2.3.2 Permanent Deformation

Diefenderfer and Apeagyei (2014) used the Flow Number (FN) test to analyze the resistance of cold recycled foamed asphalt mixtures to permanent deformation. They found that CIR and CCPR mixtures had no significant difference. Also, the effect of confinement during testing was found to be significant. A confinement procedure was recommended for the Flow Number test because this can simulate the confining stress seen in the field.

Khosravifar et al. (2015) conducted the triaxial repeated load permanent deformation (RLPD) tests on cold recycled foamed asphalt samples obtained from field cores. The test was similar to the Flow Number test. Testing was conducted at 39°C with 69 kPa confining stress and 483 kPa deviator stress. All the tests were completed at 10,000 cycles. None of the cold recycled foamed asphalt mixtures demonstrated tertiary flow in strain development. The recycled mixtures had satisfactory resistance to permanent deformation. The final permanent deformation of the recycled mixtures was 0.4 - 0.8 %, less than the final strain on the HMA specimens (1.1%).

He and Wong (2008) investigated the effect of moisture content on permanent deformation of cold recycled foamed asphalt mixtures. Dynamic creep testing was conducted in accordance with British Standard DD226. A repeated load was applied for 1,800 cycles with 100 kPa axial stress at 30°C. Resistance to permanent deformation was indicated by the creep strain rate (in log-log scale) and the ultimate strain. Results showed that an increase in RAP content did not affect resistance to permanent deformation. Mixtures with softer binder had improved resistance to the permanent deformation in the dry condition. However, those with stiffer binder tended to perform better in the wet condition.

2.3.3 Cracking

Saleh (2007) found that cold recycled foamed asphalt mixtures had lower fracture energy and fatigue life compared to HMA. This can be improved by adding mineral filler. Cox and Howard (2016) developed a mix design framework for cold recycled mixtures by establishing laboratory testing results' criteria for the recycled mixtures with emulsion and/or cement to balance the resistance to cracking and rutting. They suggested using the indirect tension fracture energy test to evaluate the cracking resistance and using asphalt pavement analyzer (APA) wheel tracking test to characterize the mixture's rutting resistance.

Wegman and Sabouri (2016) proposed a method for determining the cracking resistance of cold recycled mixtures (including both foamed asphalt and asphalt emulsion mixtures) based on the semi-circular bending (SCB) test. A single-notched specimen was loaded monotonically at a controlled mouth opening displacement (MOD). Testing was completed when the loading stress decreased below 0.5 kPa. They proposed a fracture index value for energy named "FIVE" to indicate the cracking resistance. The value of FIVE was the fracture work normalized by cracking surface area. This method was compared with the result of the disc-shaped compact tension (DCT) fracture energy on three cold recycled mixtures. The two methods ranked mixtures in the same way but the "FIVE" method was considered more convenient.

2.3.4 Other Laboratory Testing Methods

Shearing

Long and Ventura (2003) found that cold recycled foamed asphalt mixture with higher density had better shear resistance indicated by higher friction angle and cohesion. Jenkins et al. (2007) evaluated the shear performance of cold recycled foamed asphalt mixtures with different aggregates. The monotonic triaxial test results showed that friction angle reduced but cohesion did not as foamed asphalt content increased. This trend was more significant in continuously-graded crushed aggregate than a uniformly-graded sand-calcrete blend. Adding active fillers such as cement and lime also had a more significant effect for graded crushed aggregate, increasing both friction angle and cohesion. Cement was better than lime regarding the improvement to cohesion.

Raveling

Thomas et al. (2007) proposed a test method to characterize cold recycled asphalt mixtures. The method included a raveling test to simulate the raveling effect occurs in the field under traffic before the compacted mixture was fully cured. A Hobart mixer was modified and installed with an abrasion head and hose in contact with the specimens. To pass this test, the specimen should have no more than 2% raveling loss after the abrasion head rotates for 15 minutes. This test method was originally developed for cold recycled asphalt emulsion mixtures and documented in ASTM D7196 and the ARRA cold recycling guideline (2015). However, this method may be too harsh for the cold recycled foamed asphalt mixtures. No study using this method for foamed asphalt mixture was found in the literature.

2.4 Summary

Although cold recycled foamed asphalt pavement technology has been used for more than 50 years, there is still a lack of consensus on many aspects such as mix design procedures and laboratory testing methods.

- Several mix design methods have been used but there is no standard method in US. The procedures have different laboratory compaction methods and methods for selecting the optimum total water content for mixing and compacting. Also, further investigations on the effects of RAP materials and foamed asphalt binder on the properties of cold recycled foamed asphalt mixtures are needed.
- The most common mineral additive in cold recycling is the Portland cement, although some organizations also suggested hydrated lime as alternative additive. Fly ash and limestone fines have also been mentioned as potential additives. Use of fly ash in cold recycled mixtures was limited but it may be a potential alternative because of its lower cost. Effect of the additives on the cold recycled foamed asphalt mixtures has not been fully understood. Most studies on the cold recycling additives were based on test results obtained from laboratory curing while the in-situ pavement curing condition may be much different. More studies are needed to evaluate the effects of additives on the mixtures based on the test results obtained from laboratory and field curing conditions.

- Different laboratory testing methods have been used to evaluate the dynamic modulus, resistance to permanent deformation, and susceptibility to cracking. However, the variability of different test results for the cold recycled foamed asphalt mixtures has not been summarized. There was no standard test method for evaluating the cold recycled mixes. No field validation was performed to evaluate the laboratory test methods' effectiveness.

CHAPTER 3 MATERIALS

The goal of mix design for cold recycled foamed asphalt is to select proper materials to determine the optimum foamed asphalt content (OFAC), the optimum total water content (OTWC), as well as mineral additive type and content that will provide the density and strength to meet the structural needs. This research focused on using RAP as the only granular materials in cold recycling without additional aggregate.

Figure 3.1 shows four different material components in cold recycled foamed asphalt mixture and the properties that influence the mix design. RAP material is the most important material since its content can be up to 98% of the mixture by weight. The preparation method, gradation, binder content and other properties of RAP are the major factors in mix design. Foamed asphalt is produced using a laboratory foamer. The quality of foamed asphalt is often dependent on the PG and source of virgin binder. Characterization of foamed asphalt is determining the viscosity of hot asphalt and stability of foam after water is introduced. Foamed asphalt with the optimum moisture content at the desired temperature need to meet the design criteria of foaming properties. Water plays an important role in cold recycled foamed asphalt mixtures because it facilitates mixing and compaction, and distribute foamed asphalt. This study used tap water. In addition, mineral additives help to increase the curing rate, enhance the early-age strength, and improve the resistance to moisture damage. The following sections discuss all the materials used in this study.

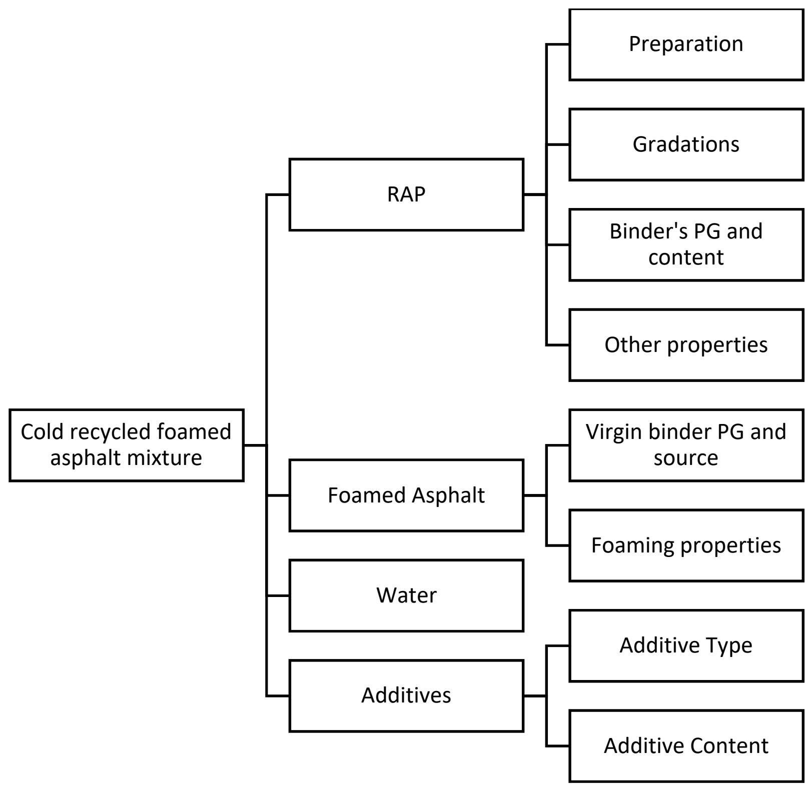


Figure 3.1 Material Components in the Cold Recycled Foamed Asphalt Mixture

3.1 RAP Materials

Preparation

In this study, RAP was sampled from a stockpile following AASHTO T2 and stored in 5-gallon buckets. This sampling procedure was found to obtain RAP materials with consistent gradations between buckets. As shown in Figure 3.2, the steer loader was used to obtain sufficient RAP from a big stockpile and make a small flat pile for sampling. The RAP was sampled into buckets at different locations on the flat-top pile. After sampling, the RAP materials were put in large pans and dried in front of a fan for one week because the stockpile was not covered. During this drying process, the RAP materials were stirred at least twice to make sure uniform drying. Residual water content was randomly tested at 110°C to verify drying status. The drying method at 40°C recommended by ARRA (2016) and Wirtgen GmbH (2012) was not applied in this study.

Figure 3.2 Sampling of RAP Materials

Gradation

RAP gradation was tested using two methods. The first method was to conduct sieve analysis without ignition procedure (ASTM C136 and C117). The second method was testing gradation for the aggregates after ignition procedure (ASTM D6307).

Table 3.1 shows eight different RAP materials (type A through H) used in this study. Each RAP material was processed (crushed and screened) from the pavement millings except for type G and H that were obtained by crushing field cores. Gradation classification was determined based on relative location of the gradation curve (without ignition) in the recommended range by Wirtgen GmbH (2012). The nominal maximum aggregate size (NMAS) was defined as one sieve size larger than the first sieve to retain more than 10 percent of the material (Brown et al. 2009).

Table 3.1 Summary of RAP Materials

Type of RAP	Gradation Classification	NMAS (mm)	Source of RAP	Binder Content (%)
DF (A)	Fine	12.5	[1]EAP, Opelika, AL	4.92
DM (B)	Medium	19.0	[2]I-81, VA	4.85
DC (C)	Coarse	19.0	[3]EAP, Opelika, AL	4.10
VM1 (D)	Medium	19.0	[4]US 280, Opelika, AL	5.19
VC1 (E)	Coarse	25.0	[5]Auburn, AL	5.36
VC2 (F)	Coarse (Open-Graded)	12.5	[6]US 280, Opelika, AL	3.48
VM2 (G)	Medium	19.0	[7]US 280, Opelika, AL	4.95
VF (H)	Fine	12.5	[8]US 280, Opelika, AL	N/A

Note:
1. Type A is fractionated RAP from the Opelika plant of East Alabama Paving (EAP).
2. Type B was transported from the Interstate-81 site in Virginia for paving cold recycled mix sections on the 2012 NCAT Test Track.
3. Type C was a combination of two fractionated RAP materials (type A and F) from EAP to achieve a desired coarse gradation.
4. Type D was milled from US Highway 280 and used in the laboratory mix design for cold recycled pavement sections on US 280.
5. Type E was sampled from the auburn plant of D & J Enterprise, Inc.
6. Type F was sampled from the Opelika plant of EAP.
7. Type G was crushed to a medium gradation using pavement cores from US 280.
8. Type H was crushed to a fine gradation using pavement cores from US 280.

Tables 3.2 and 3.3 summarize the gradations of eight RAP materials before and after the ignition process. Gradations before ignition are listed in Table 3.2. These gradations were considered closer to the condition of RAP in the field because the asphalt binder did not deform or interact with other materials during production and construction at ambient temperature. The gradation without ignition was also recommended by Wirtgen GmbH (2012) and ARRA (2016) to control the quality of RAP material. This study followed ARRA recommended gradation testing procedures (ASTM C136 and C117) instead of the Wirtgen's method (ASTM D422) because the latter one was originally proposed for soil which had been withdrawn in 2016. Table 3.3 shows gradations of the gradations of aggregates after ignition. These gradations were finer than those in Table 3.2. Because they may not represent the RAP condition in the field, these post-ignition gradations were only used as reference. Type H RAP was not tested for binder content and gradation after ignition due to availability of the material.

Table 3.2 Summary of RAP Gradations Before Ignition

Sieve size (mm)	Percent Passing							
	A	B	C	D	E	F	G	H
31.5	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
25	100.0	100.0	100.0	98.7	94.2	100.0	98.5	100.0
19	100.0	99.5	99.7	94.1	85.1	100.0	95.4	99.2
12.5	97.5	84.9	76.1	83.7	69.7	92.1	86.7	95.1
9.5	87.1	69.6	58.6	74.2	57.0	60.5	75.8	89.3
4.75	59.3	40.5	31.3	54.1	34.6	5.9	52.5	69.5
2.36	39.7	21.3	21.1	37.9	22.4	3.7	32.7	48.2
1.18	26.9	11.2	15.0	24.5	15.6	3.0	18.9	31.5
0.6	17.3	6.6	10.3	13.4	10.0	2.6	9.8	19.2
0.3	10.4	4.2	6.6	6.5	5.9	1.9	4.8	10.1
0.15	6.3	3.0	4.1	3.1	3.5	0.9	2.0	4.9
0.075	3.7	2.2	2.5	1.6	2.3	0.3	0.6	1.8

Table 3.3 Summary of RAP Gradations after Ignition

Sieve size (mm)	Percent Passing						
	A	B	C	D	E	F	G
31.5	100.0	100.0	100.0	100.0	100.0	100.0	100.0
25	100.0	100.0	100.0	100.0	100.0	100.0	100.0
19	100.0	97.6	99.7	99.4	100.0	100.0	100.0
12.5	98.6	89.3	93.9	95.2	98.0	97.7	95.8
9.5	92.1	81.7	78.4	89.0	90.2	82.3	89.2
4.75	68.4	61.7	51.0	73.9	65.7	64.6	71.5
2.36	50.7	45.3	38.7	59.7	48.9	27.9	55.9
1.18	40.3	33.6	31.3	46.8	36.2	23.4	45.4
0.6	31.8	25.8	25.2	34.9	27.9	18.6	35.5
0.3	23.6	18.8	18.9	24.7	17.7	14.9	24.5
0.15	17.1	14.0	13.4	17.6	12.0	10.6	16.8
0.075	11.8	10.7	8.8	11.3	7.4	7.01	10.1

Figure 3.3 shows five RAP materials' gradations before ignition following ASTM C136 and C117. These RAP materials have been used to design cold recycled mixes with sufficient strengths to meet the Wirtgen's ITS criteria. Other three RAP materials not used to design cold recycled mixes are not shown. The recommended gradation range by Wirtgen GmbH (2012) was plotted for comparison. However, all these RAP materials had the gradations out of the recommended range on some sieves, especially on the finer side. This indicated the recommended range by Wirtgen GmbH (2012) may be too restrictive. This range may not be able to achieve in the field since the in-place recycler cannot accurately control the RAP gradation and most contractors typically used the RAP sources available only screening the RAP on the top sieve.

RAP materials were stored in five-gallon buckets. To maintain the gradations during mix design, a mechanical splitter was used to reduce the RAP to the testing size (at least ten kilograms of RAP for a single batch of mixing). More than seven different buckets for each type of RAP (type A, B, and C) were randomly tested for gradation using dry sieve method (ASTM C136) throughout this research. The gradation of each RAP was consistent (as shown in Figure A.13 – A.15 in Appendix A).

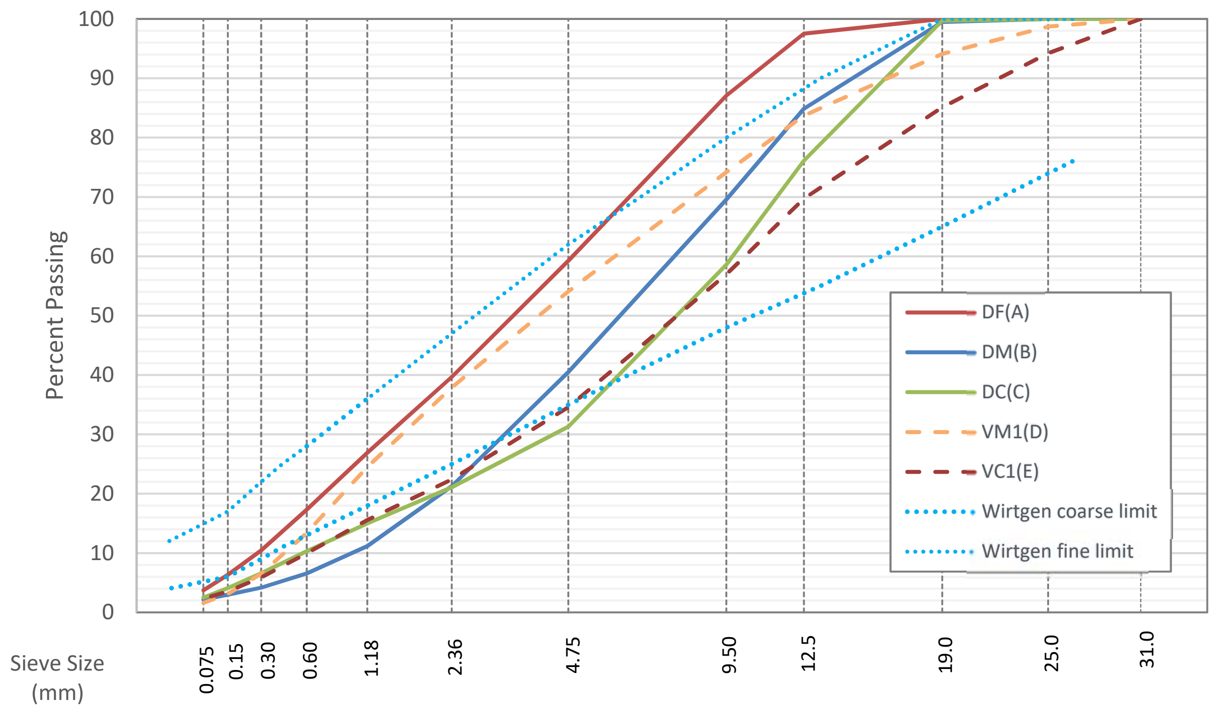


Figure 3.3 Gradations of RAP Materials Used to Design Cold Recycled Mixes

RAP Binder's PG and Content

Three RAP materials (type A, B, and C) used to determine the effects of binder and RAP on mix design and laboratory testing results were extracted and recovered (ASTM D2172 and D7906). The recovered binder was tested using dynamic shear rheometer (DSR) and bending beam rheometer (BBR) to determine the performance grade (AASHTO T315, T313, and MP1). These RAP materials were also tested by the ignition method to determine the binder content following ASTM D6307. The true grade of recovered binder and the binder content are summarized in Table 3.4. These RAP materials had similar binder content while the type-C RAP was lower than others. The true grades of these RAP materials were also similar. Type-A and type-C were the same because they were obtained from the same source. The low-temperature PG of Type-B RAP was slightly lower.

Other Properties

These RAP materials' appearances were examined visually and their binder's status were checked by-hand to evaluate if the RAP was active or inactive following the procedure suggested by Wirtgen GmbH (2012). If the RAP is active, it will have resistance to compaction. If it is inactive, the RAP can be treated as "black rock". Based on the observations, all the RAP materials had a dull grey color. When squeezed by hand, they were not sticky and the large chunks broke into piece easily. Also, the fineness modulus and the 3:1 flat-elongated (F-E) particles' percentage for the three RAP materials (A, B, and C) were measured. The fineness modulus was determined based on the gradation of the RAP materials without ignition (ASTM C125), indicating an overall size distribution of the RAP particles. Typically, higher FM indicated coarser RAP gradation. The F-E particles' percentage was measured following ASTM 4791 because its effect was found significant on the compacted density (Cross 2003). Higher F-E particles' percentage indicates the RAP material includes more flaky particles during compaction. Table 3.4 also summarizes the fineness modulus (FM) and F-E particles' percentage for the three mixtures. The FM results could distinguish these RAP materials based on their gradations. The type-B RAP material had higher F-E particles' percentage than other two RAP materials. This may be because the type-B RAP material was from another source.

Table 3.4 Summary of the RAP Materials and Binder Foaming Properties for the Mixtures

RAP Type	RAP Binder Content (%)	Recovered Binder True PG	Fineness Modulus	3:1 F-E Particles (%)
DF (A)	4.92	103.6 - 12.7	4.5	1
DM (B)	4.85	103.5 - 9.3	5.4	5
DC (C)	4.10	103.6 - 12.7	5.5	1

3.2 Foamed Asphalt

Three types of virgin asphalt binder were used to produce foamed asphalt, including two PG 67-22 binders from different refinery plants and one PG 58-34 binder. The binder type was used to distinguish these binders because not only binder PG but also binder source may have significant effects on the binder foaming properties (Newcomb et al., 2015). Foaming properties for these binders are shown in Table 3.5. Binder PG 67-22 has been commonly used in the state of Alabama for asphalt pavement construction. Even with the same PG, the two PG 67-22 binders still had different foaming properties (ER and HL). Binder PG 58-34 (B) produced a foamed asphalt with the highest ER and HL. Binder PG 67-22 (C) could not produce stable foamed asphalt at 160°C so the foaming temperature was increased to 170°C. This binder had marginal foaming properties based on the criteria recommended by Wirtgen GmbH and ARRA.

Table 3.5 Summary of Virgin Binder PG and Foaming Properties of the Foamed Asphalt

Virgin Binder	Binder Source	Foaming Properties			
		Temp. (°C)	Optimum Foaming Water Content (%)	Expansion Ratio (times)	Half-life (sec.)
PG 67-22 (A)	Lake City, FL	160	1.8	11	9
PG 58-34 (B)	Alberta, CA	160	2.0	19	10
PG 67-22 (C)	Birmingham, AL	170	1.3	9	6

3.3 Mineral Additives

This study was meant to answer questions regarding the effects of different mineral additives (cement, hydrated lime, baghouse fines, and fly ash) on the cold recycled foamed asphalt mixtures. Therefore, four additives selected for testing were Portland cement (Type I/II), hydrated lime (Type S), fly ash (Class C), and baghouse fines from a local asphalt plant. These additives are very fine with most particles passing 0.075-mm sieve. The composition and properties of cement followed ASTM C150 standard. The hydrated lime conformed to the ASTM C270 specification for masonry mortar when mixed with masonry sand. The composition and properties of fly ash met the ASTM C618 standard. A control mixture with no additive was used for comparison. Table 3.6 summarizes these additives and the assumed potential reactions contributing to the mixtures' strength. Literature review did not find directly related studies of these additives' reactions during cold recycling. The references in Table 3.6 were listed to define these reactions only and not specifically related to cold recycling. Cement, hydrated lime, and fly ash were standard products with consistent qualities but the property of baghouse fines may be dependent on the aggregate types used at the plant. The baghouse fines (BHF) sample was tested by X-Ray Diffraction (XRD) analysis to determine the mineral components. The major component was found to be silica (SiO_2) and dolomite [$CaMg(CO_3)_2$]. These two components were not active and did not react with other materials in the cold recycled mix. Therefore, this BHF may function mainly as a filler to extend the binder volume and made it stiffer. The BHF from other plants with different aggregate sources (e.g., more limestone content) may have different reaction in the mixture.

Table 3.6 Summary of Mineral Additives

Additive	Type or Source	Potential Reactions Contributing to Mixes' Strength
Cement	I/II	Hydration (Mehta and Monteiro 2014)
Hydrated lime	Type S	Asphalt-lime reaction[1] (Sebaaly et al. 2006)
Fly ash	Class C	Cementitious reaction (Mehta and Monteiro 2014)
Baghouse fines	EAP Opelika Plant	Asphalt extender[2] (Kriech and Rucker 1985)

Note:

1. Several physico-chemical reactions occur in the asphalt-lime reaction. For example, lime particles can absorb polar components of asphalt and form an interlayer.
2. Asphalt extender means that baghouse fines increases asphalt volume and lowers air void content.

Cement was used as the default additive at 1.0% by mass of dry RAP for the majority of the cold recycled mixes in this study as shown in Table 3.7, while other mineral additives were only used to replace cement in mix 5 to determine the effects of mineral type and content on the cold recycled mixes in the mineral additive study. There were two reasons to use 1.0% cement consistently in the mix. First, cold recycled foamed asphalt mixtures require additives to reduce moisture susceptibility. This small amount of cement was found to significantly improve the resistance to moisture damage based on previous experiences in a cold recycling rehabilitation project on US Hwy 280. Higher cement content may reduce flexibility and increase the cracking susceptibility of the mixtures (Wirtgen GmbH 2012, Asphalt Academy 2009). Second, the effects of cement on the mixture's performances would be the same, and therefore, did not need to be considered when investigating other effects.

3.4 Cold Recycled Foamed Asphalt Mixtures

Table 3.7 shows fifteen cold recycled foamed asphalt mixtures with different RAP materials, foamed asphalt, and additive contents. Mixtures used for the additive study are adapted from mix 5 with the only modification being additive type and amount. Cement was added to all the mixtures except for mix 11 and 12 to improve durability, asphalt-aggregate bonding, and resistance to moisture damage. Mixes 11 - 12 had no cement, allowing to evaluate the effect of cement content on the determined OTWC of the cold recycled mixes. Mix 13 with 1.5% cement was only used to validate the proposed OTWC determining method. Mix 9, 14 and 15 were used in the improvement study of the laboratory compactive effort.

Table 3.7 Summary of Mixtures with Different Binder and RAP Materials

Mix No.	Asphalt Binder	RAP	Additive (Type I/II Cement)
1	PG 67-22 (A)	DF (A)	1.0% Cement
2	PG 67-22 (A)	DM (B)	
3	PG 67-22 (A)	DC (C)	
4	PG 58-34 (B)	DF (A)	
5	PG 58-34 (B)	DM (B)	
6	PG 58-34 (B)	DC (C)	
7	PG 58-34 (B)	VM1 (D)	
8	PG 58-34 (B)	VC1 (E)	
9[1]	PG 67-22 (C)	VM1 (D)	
10	PG 67-22 (C)	VC1 (E)	
11	PG 58-34 (B)	DF (A)	No Cement
12	PG 58-34 (B)	DM (B)	
13	PG 67-22 (C)	DF (A)	1.5% Cement
14[2]	PG 67-22 (C)	VM2 (G)	Varied cement contents
15[2]	PG 67-22 (C)	VF (H)	

Notes:

1. Mix 9 used in the mix design study had 1.0% cement content. When mix 9 was used in the study of laboratory compactive effort, it had varied cement contents at 0.6, 1.0, and 1.5%.
2. Mix 14 and 15 used in the study of laboratory compactive effort contained different cement content at 0.6, 1.0, and 1.5%.

CHAPTER 4 MIX DESIGN STUDY

This chapter introduces improvements proposed for the cold recycled foamed asphalt mix design. Three improvements in different steps of design process were discussed first, followed by the proposed mix design method.

Table 4.1 summarizes the mix design procedure to be improved and the objectives of each improvement. The first improvement was to determine the laboratory compactive effort because there was no standard effort documented for cold recycled foamed asphalt mixtures. The second improvement was for the method to determine RAP material's optimum moisture content. The same compactive effort determined from the first improvement was used to compact RAP. This study examined whether this method can be used to obtain the compaction curve and determine the OWC result corresponding to its peak density. The third improvement was to propose a mathematical model to determine the OTWC for cold recycled mixes directly without additional testing for the mixtures at different total water contents.

Table 4.1 Proposed Improvements to the Mix Design Method

No. of Improvement	Design Procedure	Objective of Improvement
1	Compactive effort	• Use Superpave gyratory compactor instead of Marshall compactor and the modified Proctor hammer. • Determine the number of gyrations using SGC.
2	Test the optimum moisture content (OWC) of RAP materials	• Evaluate the suitability of SGC compactive effort to compact RAP and determine its optimum moisture content.
3	Determine the optimum total water content (OTWC) for the cold recycled mix	• Establish a model to determine the OTWC to maximize the mix's ITS so testing effort can be saved.

4.1 Improvement on the Laboratory Compactive Effort

4.1.1 Methodology

The objective of this improvement was to determine an appropriate compactive effort using Superpave gyratory compactor (SGC) for the cold recycled foamed asphalt mixtures. The compactive effort should result in specimens with densities similar to that achieved by good construction practices in the field. The relationship between SGC-compacted density and field density needs to be established. However, the correlation data was very limited in the literature because few cold recycling projects had used the SGC compactive effort in cold recycled foamed asphalt mix design. To overcome this difficulty, this study used the typical compactive effort - Marshall hammer 75 blows on each side to correlate with field densities and the SGC- compacted densities. Therefore, the relationship between field and SGC-compacted density can be established indirectly. The number of gyrations required to achieve field density should be the laboratory compactive effort. The steps to conduct this study are shown in Figure 4.1.

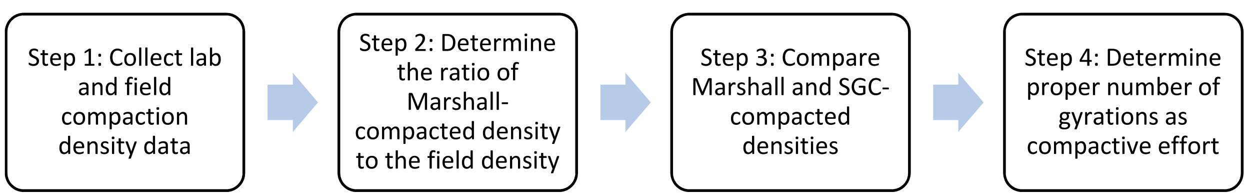


Figure 4.1 Steps to Determine the Laboratory Compactive Effort Using Superpave Gyratory Compactor

4.1.2 Results and Discussions

The results of density were collected from six different cold recycled foamed asphalt pavement sections, including three sections on the NCAT Test Track, one section on Lee Road 159 in Alabama, and two sections on Interstate 81 in Virginia. The cold recycled pavement layers in these sections had thickness larger than 100 mm (Timm et al. 2015, Pavetrack 2016, Diefenderfer and Apeagyei 2014). Figure 4.2 shows the comparison of the field density and laboratory-compacted densities for these sections. Marshall compactive effort (75 blows each side) was used in the laboratory. The comparison showed the two results were similar but the field density in most sections were slightly higher than the laboratory-compacted densities using Marshall compactive effort. The ratio between two densities were calculated and shown in Figure 4.2. Except for the ratio of 99.4% for one section in I-81, other sections had field density at least 101.4% of laboratory density.

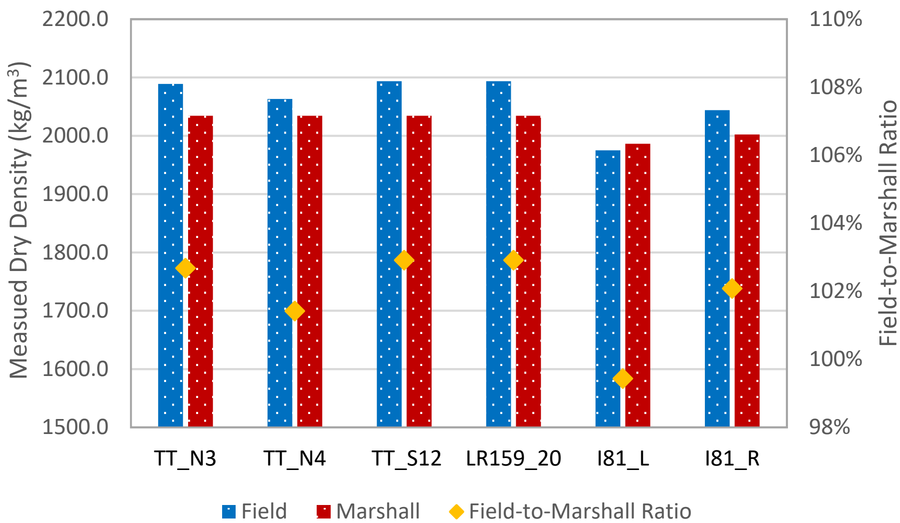


Figure 4.2 Comparison of Field Density and Laboratory-Compacted Dry Density Using Marshall Compactive Effort

As shown in Table 4.2 The average field density was about 101.9% of the average laboratory-compacted density using Marshall compactive effort. Once the ratio of Marshall method's density to the SGC compacted density is known, the relationship between field density and laboratory SGC compacted densities can be determined.

Table 4.2 Comparison of Average Field Density and Laboratory-Compacted Densities

Compactive Effort	**Dry Density (kg/m^3)**
Avg. Field (pcf)	2059.7
Avg. Marshall (pcf)	2021.0
Field-to-Marshall Ratio	101.9%

Figure 4.3 shows the comparison of three laboratory-compacted densities using Marshall compactive effort, SGC 30 gyrations, and SGC 35 gyrations. The densities were measured from three

laboratory-produced cold recycled foamed asphalt mixtures at 2 and 3% foamed asphalt contents. These mixtures contained the same binder but different RAP materials as shown in Table 3.7. Mix 9 was slightly finer than Mix 14 but coarser than Mix 15.

Although different cement contents were used for different compactive efforts due to the mix design consideration (i.e., 0.6% for Marshall, 1.0% for SGC 30 gyrations, and 1.5% for SGC 35 gyrations), the differences were considered to have negligible effect on the laboratory densities. Figure 4.3 shows the Marshall compacted densities were similar to the two SGC-compacted densities for mix 9 but became lower than SGC-compacted densities for mix 14 and 15. The major differences in mix 14 and 15 were between two SGC-compacted densities. This may be because the RAP gradation in mix 9 was more stable so that the density was not sensitive to the change of compaction effort. In addition, higher foamed asphalt contents did not improve the laboratory compacted densities.

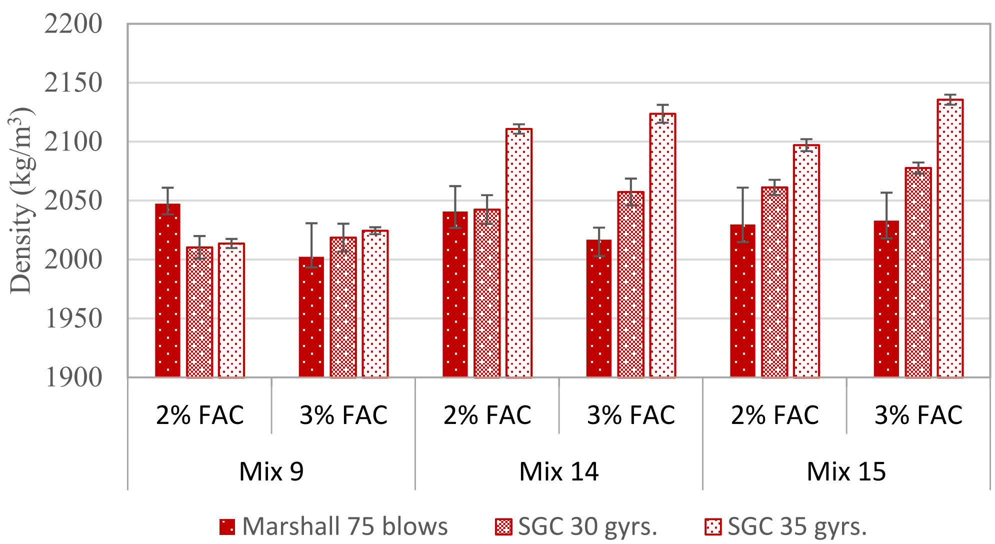


Figure 4.3 Comparison of Measured Dry Densities Using Different Laboratory Compaction Methods

The ANOVA was not used for evaluating the effects of compactive effort, mix, and FAC altogether due to violated assumption of equal variance. Instead, Tukey's test was performed to evaluate the effect of compactive efforts for each mixture at each foamed asphalt content. Table 4.3 shows the Tukey's test results for the mixtures. Only mix 9 and mix 14 with 3% FAC had valid grouping results.

The assumption of equal variance in Tukey's test for mix 14 with 2% FAC and mix 15 with both FAC were violated. This was because the Marshall-compacted specimens had higher variability than SGC-compacted specimens. The coefficient of variation (COV) of Marshall-compacted densities doubled compared to the COV of SGC-compacted densities. The available grouping results showed that the compactive effort effect was more significant for mix 14 than for mix 9.

Table 4.3 Summary of Tukey's Test Grouping Results for Evaluating the Effect of Compactive Effort

Compactive Effort	Mix 9		Mix 14		Mix 15	
	2%FAC	3%FAC	2%FAC	3%FAC	2%FAC	3%FAC
M-75	A	A	N/A	C	N/A	N/A
SGC-30	B	A	N/A	B	N/A	N/A
SGC-35	B	A	N/A	A	N/A	N/A

Table 4.4 summarizes the differences from the Marshall densities to the two SGC-compacted densities. The mixtures after SGC compaction had higher density than Marshall compaction except for mix 9 with 2% FAC. The average change in density from the Marshall compacted specimens to the SGC-compacted specimens were also calculated. The results showed that the compactive effort of SGC 30 gyrations had 100.8% of Marshall density and the density compacted by SGC 35 gyrations was 102.8% Marshall density. Since the field density was 101.9% of Marshall compacted density, the field density should be slightly higher than the SGC 30 gyrations' density but lower than SGC 35 gyrations' density.

Table 4.4 Change of Densities Comparing the Two SGC-Compacted Densities to the Marshall-Compacted Density

Mix No.	FAC (%)	% Density Change from M75[1] to SGC30[2]	% Density Change from M75 to SGC35[3]
Mix 9	2	-1.8	-1.6
	3	+0.8	+1.1
Mix 14	2	+0.1	+3.4
	3	+2.0	+5.3
Mix 15	2	+1.6	+3.3
	3	+2.2	+5.0
Average % Change		+0.8	+2.8

Note:

1. Densities of specimens compacted by Marshall compactive effort.
2. Densities of specimens compacted by SGC 30 gyrations.
3. Densities of specimens compacted by SGC 35 gyrations.

The comparison of densities showed the typical Marshall compactive effort may be good to match with the field compaction effort. However, compared to the Marshall compaction method, the SGC compaction method was reported to obtain more apparent peak ITS at the optimum foamed asphalt content and the SGC-compacted density was more sensitive to the change of foamed asphalt content (Kim et al. 2007). Also, this study found the SGC compacted specimens had more consistent densities than the Marshall-compacted specimens. Therefore, the compactive effort of SGC 30 gyrations was selected for the mix design procedure of cold recycled foamed asphalt mixtures. As long as the laboratory-compacted specimens satisfy the design strength criteria, the mixture compacted to higher density in the field is expected to have sufficient strength. Because cold recycled mixes contain water, the water driven out of mold needs to be cleaned after each compaction. Some Superpave gyratory compactors may need to add waterproofing features to prevent possible corrosion. Some SGC models have actuator rams that push upward on a base plate to compact the specimen. To avoid negative consequences caused by water draining from specimens during compaction, the following ideas are suggested. First, use a paper disc under specimens on the base plate to absorb a portion of water. Second, add a ring-shaped pad of waterabsorptive material around the actuator to collect the water and replace the pad before it becomes saturated. Third, grease the water-affected area inside of the SGC and/or apply an anti-corrosion coating.

4.2 Improvement on the Testing Method for RAP Material's OWC

4.2.1 Methodology

RAP material can be compacted to its maximum dry density after mixing with the optimum amount of water. This water content is called optimum water content (OWC) and is used as guidance for determining optimum total water content (OTWC) in cold recycled asphalt mixture. Wirtgen GmbH (2012) recommended using the modified Proctor method to compact RAP in accordance with AASHTO T180 (or ASTM D1557) and using 100% OWC of the RAP material as OTWC for the foamed asphalt mixture. However, this compactive effort for RAP did not match with that used for compacting foamed asphalt mixtures. Also, the modified Proctor test equipment may not be available for every asphalt laboratory. The basic mechanical testing equipment may cost as much as $7,000 (Gilson, Inc. 2017). The manual testing equipment is much cheaper but requires intensive labor or longer testing time.

In this study, the compactive effort of SGC 30 gyrations was found to be suitable for compacting the mixture during mix design process. Therefore, this compactive effort was also used to compact RAP at different water content to determine the OWC. This study proposed a method to prepare RAP material and determine its OWC, which was similar to the procedure specified in ASTM D1557. Figure 4.4 shows the basic steps of this method. A description of each step follows.

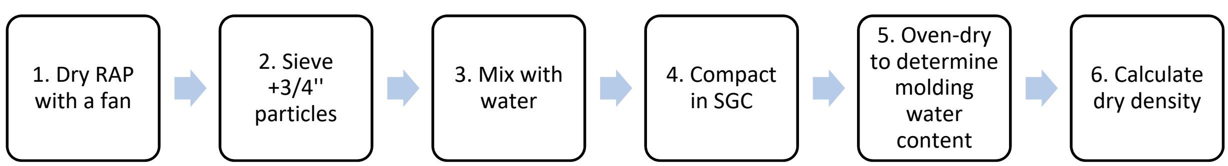


Figure 4.4 Procedure to Determine RAP OWC Using SGC

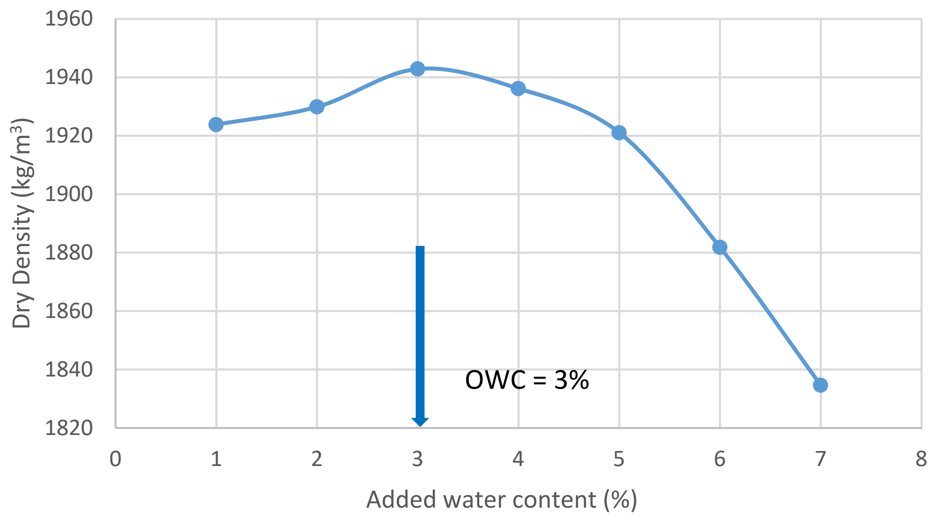
(1) Dry RAP with a fan for a week;
(2) Remove particles above ¾-inch sieve for use with 100-mm diameter molds;

(3) Add water to RAP at different water contents (e.g., 3.0, 4.0, 5.0, and 6.0 percent for RAP material A); At each water content, prepare at least 5-kg of RAP (using mechanical splitter to reduce RAP to test size) to compact three replicates. Add water slowly and mix by hands with a scoop until uniform;
(4) Load wet RAP into a 100-mm mold and compact for 30 gyrations at an internal angle of 1.16° in SGC;
(5) Record the compacted height and wet weight after compaction. Dry the compacted specimens overnight at 110°C to determine the molding water content. The molding water content is typically lower than the added water content. This step is only used as a quality control procedure to estimate the actual amount of water added as a reference.
(6) Calculate dry density using Equation 4.1. It was modified from the dry density's calculating equation in ASTM D1557. The added water content was used instead of the molding water content. because the accurate molding water content was difficult to obtain due to fast water drain especially when excessive water was used during compaction. Then, plot the dry densities against water contents and determine the OWC at the maximum dry density. Based on the results obtained from this study, the difference between the determined OWC results based on the added water and the molding water contents were small (less than 0.7%).

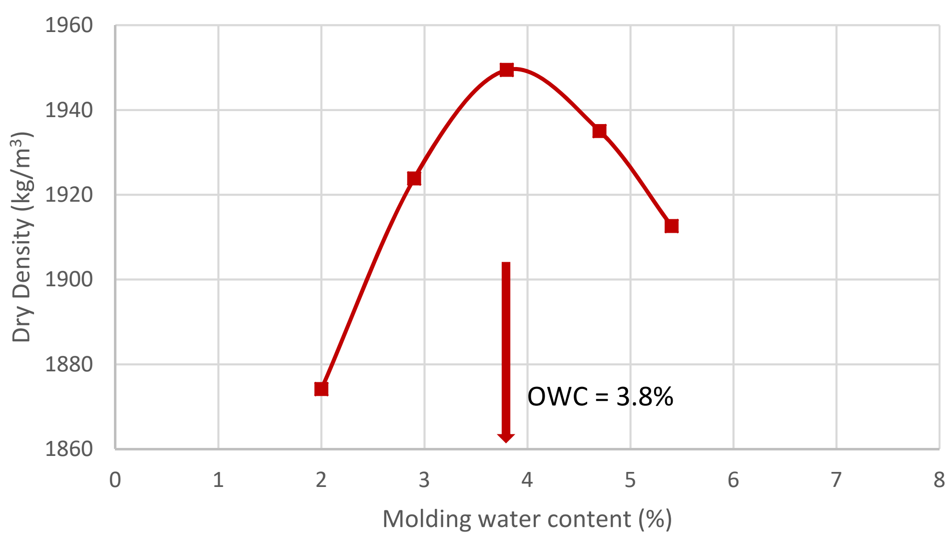
$$\rho_d = \frac{\rho_w}{1 + \frac{w}{100}} \tag{4.1}$$

Where:
ρ_d= dry density of compacted specimen (kg/m^3);
ρ_w= wet density of compacted specimen (kg/m^3);
w= water content in the mixture (%).

Figure 4.5 shows a typical RAP material's compaction curve using SGC 30 gyrations and the modified Proctor compactive effort. Both methods show the peak dry density of the RAP material corresponding to the OWC result. The SGC compaction curve in Figure 4.5(a) was built by plotting dry density against the added water content. While the modified Proctor test compaction curve in Figure 4.5(b) was built by plotting dry density versus the molding water content.



(a) Typical Dry Density vs. Added Water Content by SGC (e.g., DM(B) RAP)



(b) Typical Dry Density vs. Molding Water Content by Modified Proctor Test (e.g., DM(B) RAP)

Figure 4.5 Comparison of Compaction Curves by the SGC and the Modified Proctor Test Method

Four RAP materials were compacted using both the SGC method and the modified Proctor method to determine OWC and peak dry density from the compaction curve. Table 4.5 shows the RAP materials and the range of water content used during compaction. These RAP materials had different gradations from fine to coarse. At least four water contents with 1% interval were used for every RAP material. A little higher range was used for the modified Proctor method because it tended to give higher OWC result than by the SGC method.

Table 4.5 Experimental Plan for Determining the RAP Material's OWC

RAP Materials	Selected Water Contents for Testing (%)	
	Proposed SGC method	Modified Proctor Method (ASTM D1557)
DF (A)	3, 4, 5, 6	5, 6, 7, 8
DM (B)	1, 2, 3, 4, 5, 6, 7	2, 3, 4, 5, 6
VM1 (D)	2, 3, 4, 5, 6, 7	3, 4, 5, 6, 7, 8
VC1 (E)	2, 3, 4, 5	4, 5, 6, 7

4.2.2 Results and Discussion

The OWC results determined by the SGC method and the modified Proctor method are summarized in Table 4.6. For both methods, finer RAP tended to have higher OWC than coarser RAP. The SGC method had lower OWC of RAP than the modified Proctor method. This was expected because the SGC compactive effort was found higher than the modified Proctor method in another study at NCAT (Ma et al. 2018). Table 4.6 shows these RAP materials were compacted to similar densities using two compaction methods, but the SGC method yielded much lower OWC. The OWC results from the modified Proctor method were also more sensitive to changes in RAP gradation than those of SGC method. Regarding the maximum dry density, the results from the modified Proctor method were slight higher than the result from the SGC. As shown in Figure 4.6, densities from these two methods had strong correlation (R-squared = 0.887). This indicates SGC method can also characterize RAP materials based on the compacted density at the optimum water content.

Table 4.6 Summary of RAP OWC and Maximum Dry Density Determined from Proposed Method and Modified Proctor Method

RAP Materials	RAP OWC (%)		Maximum Dry Density (kg/m^3)	
	Proposed SGC	Mod. Proctor	Proposed SGC	Mod. Proctor
DF (A)	4.0	6.5	2046.8	2101.1
DM (B)	3.0	3.8	1940.8	1944.6
VM1 (D)	4.0	6.5	1969.3	2018.3
VC1 (E)	3.0	5.3	1899.6	1955.5

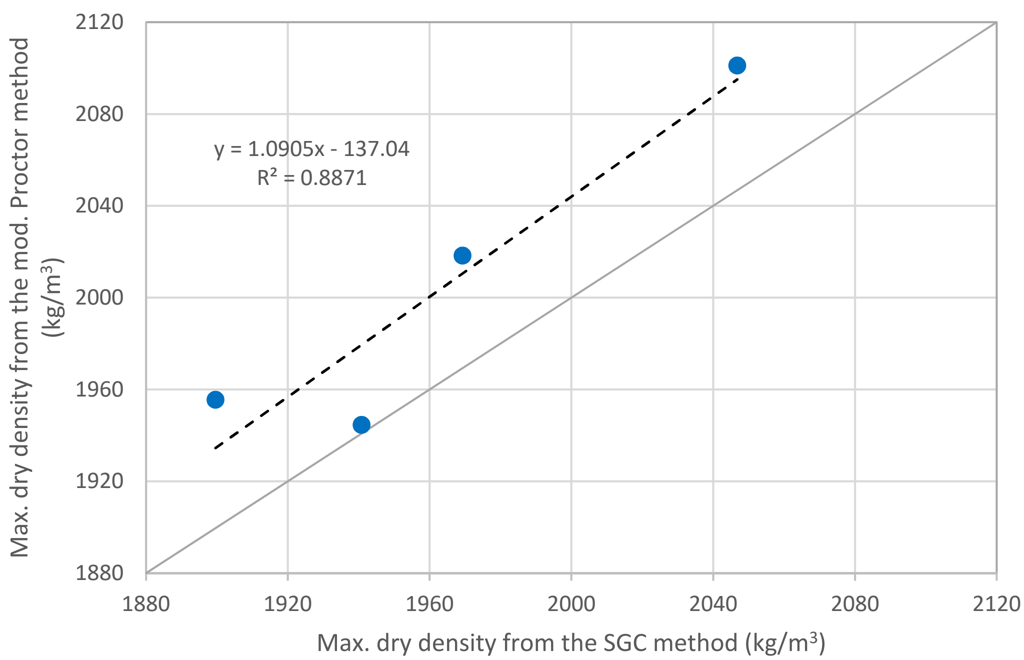


Figure 4.6 Correlation between the Maximum Dry Densities from the SGC Method and the Modified Proctor Test Method

4.3 Improvement on the Optimum Total Water Content Determining Method

4.3.1 Methodology

In cold recycling, water is added to achieve uniform mixing and to facilitate compaction by providing lubrication in the mix (ARRA, 2014). This improvement to the mix design of cold recycled foamed asphalt mixtures was focused on optimizing total water content to achieve better laboratory testing results. The proposed compactive effort and the method to determine the OWC of RAP materials were used in this improvement study. In this section, the experimental plan for the effects of water were discussed first, followed by the modeling method and the validation method.

4.3.1.1 Effects of Water on the Cold Recycled Mixes' Performance

To optimize water content in cold recycled foamed asphalt mixtures, the effects of water during mixing and compacting on mixture's properties in a laboratory environment needed to be investigated. Understanding these effects is essential because it gives the importance of the water to the mixture and answers why this improvement was based on the water content determining method. In most previous studies, water was added only once before mixing. This amount of water participated in both mixing and compacting processes during mix production (Asphalt Academy 2009; Kim and Lee 2006). However, Wirtgen GmbH (2012) suggested adding water to the mixture in two separate steps - adding 75% of the RAP OWC before the foamed asphalt to achieve ideal mixing, then adding the remaining water after the foamed asphalt to facilitate compaction. This improvement was based on the first method in which the total water was added only once. But the two-step method provided a good way for investigating the effect of water on the mixture during the mixing process and compacting process individually. Investigation for the effects included evaluating the effects of water during mixing and compacting, as well as the combined effect of total water (added once) on the cold recycled mixes' density and indirect tensile strength(s).

Effect of Water during Mixing

During mixing, water helps distribute foamed asphalt. This concept was proposed in the 1950s by Csanyi (1957). In this study, the effect was evaluated by examining dry ITS at different mixing water contents while keeping total compaction water the same. The mixing water was added in RAP before foamed asphalt and the remaining water was added afterwards. Wet ITS was not tested due to a lack of available specimens. Table 4.7 summarizes the experimental plan. All the water content levels regarding MWC (mixing water content) and TWC (total water content) was represented by the percentage of RAP OWC. For example, the OWC of type DM(B) RAP was 3.0%. The two levels (50 and 75% of OWC) were 1.5 and 2.3% by weight of dry RAP material. The same mixture (No.2) was tested for density and dry ITS at two mixing water contents (MWC) at each of three total water contents (TWC). The Analysis of variance (ANOVA) was performed in Minitab 18 software to determine the effect of MWC and compare it with the effect of TWC.

Table 4.7 Experimental Plan for Determining the Effect of Mixing Water on Density and the Dry ITS Results

Mix No.	Binder	RAP	FAC (%)	MWC (% of OWC)	TWC (% of OWC)
2	67-22 (A)	DM (B)	3.0	50, 75	75
2	67-22 (A)	DM (B)	3.0	50, 75	100
2	67-22 (A)	DM (B)	3.0	50, 75	125

Effect of Water during Compaction

During compaction, water provides lubrication between particles and facilitates the compaction process. The effect of water content may be observed from the change in compacted density or ITS by keeping mixing water content constant and varying compaction water content. An experiment was designed based on the mix design procedure proposed by Wirtgen GmbH (2012), in which water was introduced in two steps. A total of 50% RAP OWC was added first, and the remaining water was added after the foamed asphalt was introduced. Then, mixing was performed by hand to ensure the remaining water was distributed uniformly in the mixture. All the added water helped compaction. In this case, the effect of total water content on density and dry ITS represents the effect of water during compaction because the mixing water content was kept the same. Table 4.8 shows the experimental plan for determining the effect of compaction water on the density and the dry ITS results at different foamed asphalt contents. Four levels of total water contents (each includes same mixing water content but different compaction water content) in Table 4.8 were used to determine the effect of water during compaction on the density and dry

ITS. These different TWC levels (50, 75, 100, 125% of OWC) were 1.5, 2.3, 3.0, and 3.8% by weight of dry RAP.

Table 4.8 Experimental Plan for Determining the Effect of Compaction Water on Density and Dry ITS Results

Mix	Binder	RAP	TWC (% of OWC)	FAC (%)
2	67-22 (A)	DM (B)	50, 75, 100, 125	1.5
2	67-22 (A)	DM (B)	50, 75, 100, 125	2.0
2	67-22 (A)	DM (B)	50, 75, 100, 125	2.5
2	67-22 (A)	DM (B)	50, 75, 100, 125	3.0

Effect of Total Water Content

The above experimental plans in Table 4.7 and 4.8 were expected to determine the influence of the water during mixing and compacting separately. But this improvement study was based on the design method that total water content was added once into the mixture. Therefore, the same amount of water was used for mixing and compacting. The effect of water on the mixture can be considered as a combined effect. This combined effect of water on density and ITS results were evaluated for six cold recycled mixes. Both dry and wet ITS results were included for evaluation because of more available specimens. Table 4.9 summarizes the experimental plan for determining the effect of total water content on density and strength (dry and wet ITS). Each mixture was tested at two FAC levels. This allowed an ANOVA to determine the effects of TWC as well as FAC.

Table 4.9 Experimental Plan for Determining the Combined Effect of Water on Density and the ITS Results

Mix	Binder	RAP	TWC (%OWC)			FAC (%)
			Effect on Density	Effect on Dry ITS	Effect on Wet ITS	
1	67-22 (A)	DF (A)	50, 75	50, 75	50, 75	2, 3
2	67-22 (A)	DM (B)	75, 100, 125, 150	75, 100, 125, 150	75, 100, 125, 150	2, 3
3	67-22 (A)	DC (C)	100, 125, 150, 175	100, 125, 150, 175	100, 125, 150, 175	2, 3
4	58-34 (B)	DF (A)	50, 75	50, 75	50, 75	2, 3
5	58-34 (B)	DM (B)	75, 100, 125,	75, 100, 125	75, 100, 125	2, 3
6	58-34 (B)	DC (C)	100, 125, 150	100, 125, 150	100, 125, 150	2, 3

4.3.1.2 Modeling

In addition to the total water content (TWC), other factors may also influence the properties of mixtures. The potential factors had to be investigated before conducting modeling to find the relationship between OTWC and the factors. The potential factors affecting mixtures' properties (density, dry and wet ITS) were properties of the RAP and binder. Before investigating these properties, the OTWC needed to be validated because optimizing one result could affect other results.

Cold recycled mixtures must have adequate strength to support traffic loads. Current mix design methods set the foamed asphalt content based on minimum dry and wet ITS. Higher foamed asphalt content typically improves the ITS. Water helps mixing and compaction of the cold recycled mixtures. But unlike the asphalt content, there is an optimum total water content corresponding to a peak ITS. Either too much or too little water reduces the strength. Also, if the water content is not at the optimum level, asphalt may not be properly distributed and peak density may not be achieved. If so, more foamed asphalt may be needed to achieve the minimum strength. Because strength is the most important mix design target, the optimum total water content (OTWC) was determined at the peak dry ITS. Wet ITS was not used because it was insensitive to the change of total water content. Therefore, this study selected the dry ITS results to determine the OTWC. The correlation between dry ITS and density, and the correlation between dry ITS and wet ITS were examined. Then, a regression analysis on the OTWC was conducted.

Validation of dry ITS to determine the OTWC

Results of density, the dry ITS, and the wet ITS obtained from the six cold recycled mixes (No. 1 - 6) were used for correlation. A total of 167 data points was available to correlate the dry ITS with density. A total of 184 data points was available to correlate the dry ITS with the wet ITS. Correlation results are summarized in Section 4.3.2. Dry ITS correlated well with the other two properties. Therefore, using the dry ITS to determine the OTWC was validated.

Regression Analysis

Three materials properties (RAP material's OWC, binder type, and FAC) were selected as predictors to build the regression model for the OTWC. First, the RAP material, binder, and FAC were found to have significant effects on the dry ITS. Therefore, they may also influence the OTWC. The effect of RAP material's OWC and binder on the dry ITS were analyzed and summarized in Section 6.2.1. The significant effect of FAC on the dry ITS was confirmed during evaluating the effect of total water content. The analysis result is discussed in Section 4.3.2.1.

Second, the most commonly used method to determine the OTWC of mixture was only based on the relationship between the OTWC and the RAP material's OWC (Mohammad et al. 2003, Asphalt Academy 2009, Wirtgen GmbH 2012). This relationship has been widely used because RAP material can be used up to 98% of total materials (by mass) in the cold recycled foamed asphalt mixture. Although small amount foamed asphalt was added, the OTWC of mixture may still be highly dependent on the OWC. Also, slight gradation change was observed in this study which was because the foamed asphalt tended to bond the fine RAP particles to form the mastic. This little gradation change caused by the interaction between foamed asphalt and fine RAP particles may contribute to the difference between the OTWC and the OWC. The distribution of foamed asphalt may also affect the differences between two optimum water contents. The temperature of foamed asphalt mixture after mixing dropped quickly to about 70 - 80 °F but the mastic was still much softer than the RAP particle. The soft mastic can reduce the friction during compaction and increase the density and strength of the compacted mixture. Therefore, mixture with more uniform distribution of foamed asphalt would be easier to compact and require less water. Different binders and FAC affecting distribution may lead to the different OTWC.

To build a model for determining OTWC, the measured OTWCs for different mixtures were obtained first. The measured OTWC for each mixture was determined by fitting quadratic regression model (2^{nd} order polynomial) for the tested dry ITS results at different total water contents (TWCs) at each FAC. The quadratic multiple regression was selected because it was simple and had sufficient fitting quality to explain most of the variability of the data. The measured OTWC corresponded to the maximum dry ITS on the regression curve built. Then, the measured OTWC and three determining predictors were used for modeling. In this regression model, the binder was treated as a categorical factor. The OWC of RAP materials in the model was used as a continuous factor. Finally, the regression model was built by correlating the measured OTWC with one categorical factor (binder) and two continuous factors (FAC and OWC). Each binder would have one regression model.

4.3.1.3 Validation

The proposed OTWC determining model was validated using different mixtures from the six mixtures used for model development. Also, dry ITS at a predicted OTWC based on this proposed model was compared to the dry ITS at two other OTWC levels determined based on typical design methods.

Validation Using Different Mixtures

Table 4.10 shows the mixtures used for validating the models (No. 7 – 12). These mixtures were different from the mixtures –for developing the model (No. 1 – 6) regarding FAC, binder, and RAP materials. Mixtures 11 and 12 did not contain cement so the robustness of the model to the change of cement content could be evaluated. These mixtures were tested for dry ITS results at different TWC levels. The measured OTWC was determined by the quadratic regression.

Table 4.10 Experimental Plan for Model Validation Using Different Mixtures

Mix	Binder	RAP	FAC (%)
7	58-34 (B)	VM1 (D)	2.5
8	58-34 (B)	VC1 (E)	2.5
9	67-22 (C)	VM1 (D)	2.5
10	67-22 (C)	VC1 (E)	2.5
[1]11	58-34 (B)	DF (A)	3.0
[1]12	58-34 (B)	DM (B)	3.0

Note: 1. No cement was added in mixes 11 and 12.

Compare with Typical OTWC Determining Methods

The proposed OTWC determining method needs to be compared with a typical method to evaluate its effectiveness for improving the dry ITS. Wirtgen GmbH (2008) suggests using the modified Proctor test to determine RAP material's OWC but it suggested a reduction factor to calculate the OTWC from RAP's OWC. In this study, the dry ITS results of mixtures at different TWCs were available to be used for comparing the two OTWC determining methods regarding their corresponding ITS results. The dry ITS results of six mixtures were available to use for comparison. Other mix design procedure from ARRA, Asphalt Academy, or VDOT recommended a range of RAP material's OWC to be used in the mixture. There was no single value of OTWC can be used for comparison with the proposed OTWC in this study.

4.3.2 Results and Discussions

4.3.2.1 Effects of Water on the Cold recycled mixes' Performances

Effect of water during mixing

Table 4.11 summarizes the effect of mixing water on density and dry ITS. Mixing water content (MWC) has a significant effect on density but not on dry ITS. The interaction of mixing water and total water had an insignificant effect on either density or dry ITS. The effect on wet ITS was not analyzed because only three specimens in dry condition were prepared at each water content. With the same total water content and foamed asphalt content during compaction, different mixing water contents caused significant differences in the compacted density. This indicated that both total water content and the foamed asphalt distribution governed by mixing water content affect compaction and strength.

Table 4.11 Effect of Water during Mixing by ANOVA

Factors	Effect on Density		Effect on dry ITS	
	P-value	Significance	P-value	Significance
[1]MWC	0.016	Y	0.144	N
TWC	0.034	Y	0.007	Y
Interaction	0.880	N	0.987	N

Note: 1. MWC stands for mixing water content.

Effect of Water during Compaction

The TWC in Table 4.12 represents the effect of water during compaction. As discussed in Section 4.3.2.1, the total water content (TWC) composed of a fixed mixing water content (MWC) and a varying compaction water content. The densities at different TWC were significantly different. TWC had more influence on density than the foamed asphalt content (FAC) or the interaction. Because the mixing water content was fixed, the effect of total water content on density or dry ITS was caused by the change of added water for compaction. First, the results indicated that the added water during compaction had a more significant effect on density than the effect of mixing water content shown in Table 4.11. Second, the added compacted water or the corresponding change in total water content together with foamed asphalt content had a significant effect on the dry ITS.

Table 4.12 Effect of Water during Compaction by ANOVA

Factors	Effect on Density		Effect on dry ITS	
	P-value	Significance	P-value	Significance
TWC	<0.001	Y	0.479	N
FAC	0.615	N	<0.001	Y
Interaction	0.303	N	0.008	Y

Effect of Total Water Content

To analyze the water during mixing and compacting individually, the water was added in two steps (before and after adding foamed asphalt). However, in CCPR and CIR, water is added only once. Therefore, this improvement for determining OTWC was based on the one-step method. This section evaluates the effect of total water content using six mixtures. The water facilitates both mixing and compacting process. This combined effect can be evaluated through the change in compacted density, dry ITS, and wet ITS.

Table 4.13 summarizes the effect of TWC on density, the dry ITS, and the wet ITS for the six mixtures. The effect of FAC was also evaluated to compare with the TWC factor. Interaction between the two factors was included in analysis. The p-value results from ANOVA are summarized to show the significance of each effect. A p-value less than 0.05 was considered significant. The exact number of p-less than 0.001 was not shown in the Minitab's analysis result due to its default decimal setting. As shown, the TWC factor had significant effects on density of three mixtures (3, 5, and 6), the dry ITS for three mixtures (2, 4, and 6), and the wet ITS for three mixtures (2, 5, and 6). For mix 3, the effect of TWC on either dry or wet ITS result was not significant, but the effect became more significant (p-value = 0.053 for dry ITS, and 0.060 for wet ITS) when the interval of TWC levels doubled (from 25 to 50% of OWC). The FAC had significant effects on the dry and wet ITS results for each of the six mixtures, as well as the densities of four mixtures (1, 3, 5, and 6). The effect of FAC was more significant than the effect of TWC. Overall, Table 4.13 shows that the effect of water on density was similar to the effect of FAC. This was expected because both water and foamed asphalt provide lubrication during compaction. Table 4.13 also shows that the effect of water on dry or wet ITS was significant on three of six mixtures, but not as significant as FAC. This was not expected because water provides no bonding between RAP particles. The results indicate that water indirectly affects the strengths through its effect on density.

Higher density of compacted mixture improves the bonding between RAP particles. Therefore, TWC needs to be optimized in the cold recycled foamed asphalt mix design.

Table 4.13 Summary of Total Water Content Effect on Density and ITS Results

Mix No.	Factors	Effect on Density		Effect on Dry ITS		Effect on Wet ITS	
		P-value	[1]Sig.	P-value	[1]Sig.	P-value	[1]Sig.
1	TWC	0.270	N	0.506	N	0.134	N
	FAC	0.019	Y	<0.001	Y	0.001	Y
	Interaction	0.156	N	0.153	N	0.576	N
2	TWC	0.149	N	0.007	Y	<0.001	Y
	FAC	0.115	N	<0.001	Y	<0.001	Y
	Interaction	0.152	N	0.509	N	0.184	N
3	[2]TWC	0.001	Y	0.194	N	0.122	N
	FAC	<0.001	Y	<0.001	Y	<0.001	Y
	Interaction	0.552	N	0.207	N	0.695	N
4	TWC	0.547	N	0.048	Y	0.997	N
	FAC	0.680	N	0.001	Y	0.007	Y
	Interaction	0.394	N	<0.001	Y	0.106	N
5	TWC	<0.001	Y	0.405	N	0.001	Y
	FAC	<0.001	Y	<0.001	Y	<0.001	Y
	Interaction	0.018	Y	0.492	N	0.018	Y
6	TWC	<0.001	Y	0.001	Y	0.041	Y
	FAC	<0.001	Y	<0.001	Y	<0.001	Y
	Interaction	0.260	N	0.185	N	0.550	N

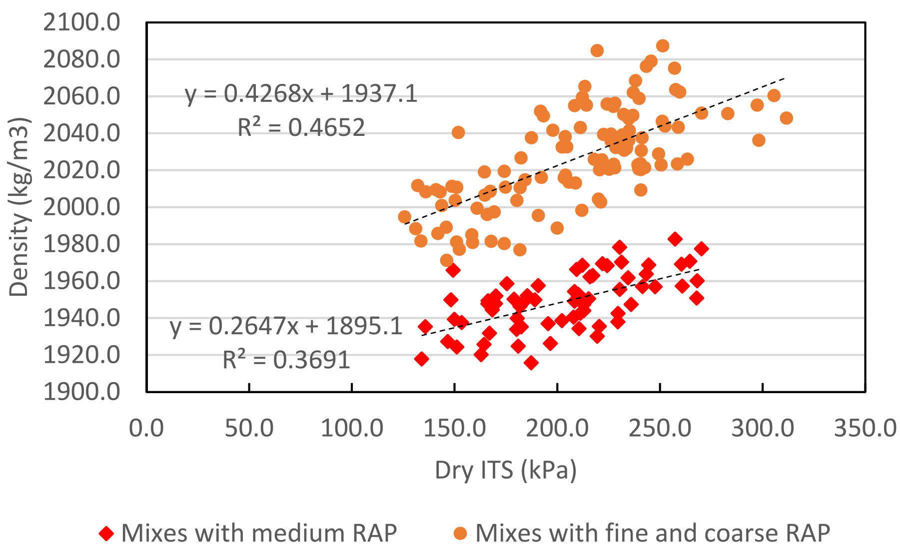
Notes:

1. "Sig." stands for significance.
2. P-value of TWC factor became close to 0.05 as interval doubled.

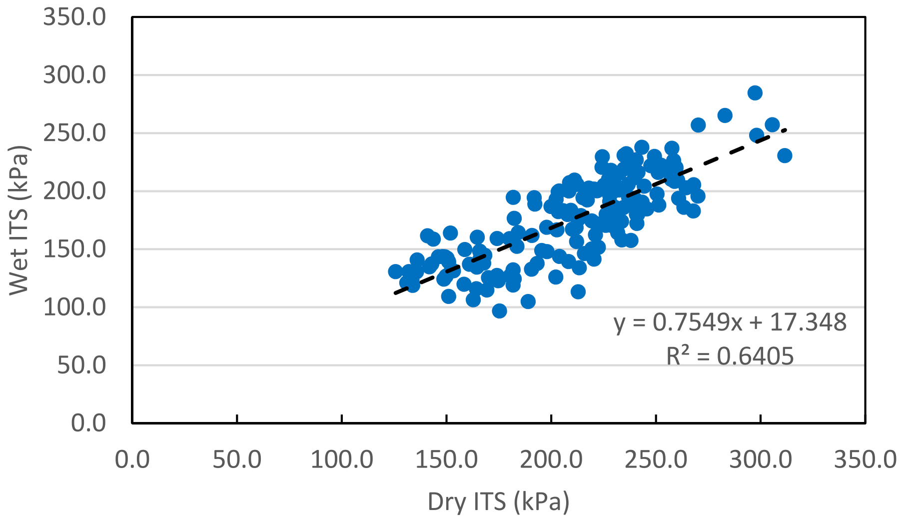
4.3.2.2 Modeling

Selection of dry ITS to determine the OTWC

Correlation between the dry ITS result and other two results (density and the wet ITS result) were evaluated. If their correlations were strong, selection of the dry ITS to determine the optimum total water content (OTWC) was validated. Figure 4.7 shows these two correlations. Both correlations were positive, indicating these results tended to increase together. Figure 4.7(a) divided all the data points into two groups based on the RAP gradation for regression analysis because the mixtures with medium RAP (No. 2 and 5) had relatively lower densities than other mixtures. If only one regression model was fitted, the gap between two groups would reduce the degree of correlation indicated by the R-squared value and may lead to misleading conclusion for the correlation. Therefore, the two groups of densities were correlated to the dry ITS individually. Comparing Figure 4.7(a) and 4.7(b), the dry ITS result correlated better with wet ITS than with the density. Each group of data points in Figure 4.7(a) were more scattered than those in Figure 4(b). These positive correlations were sufficient to validate the use of dry ITS in determining the OTWC.



(a) Correlation between the dry ITS result and the density



(b) Correlation between the dry and wet ITS results

Figure 4.7 Correlations between the Dry ITS Results and Other Two Performances

Regression Analysis

To conduct the regression analysis, the measured OTWC and predictors were needed. As discussed before, the measured OTWC corresponded to the maximum dry ITS was determined by quadratic regression. Figure 4.8 shows a typical regression model built based on the dry ITS results and total water contents. The measured OTWC was determined at the peak ITS. The same method was used to determine the measured OTWC of the six mixtures. Results were listed in the 6th column of Table 4.14 along with the calculated OTWC.

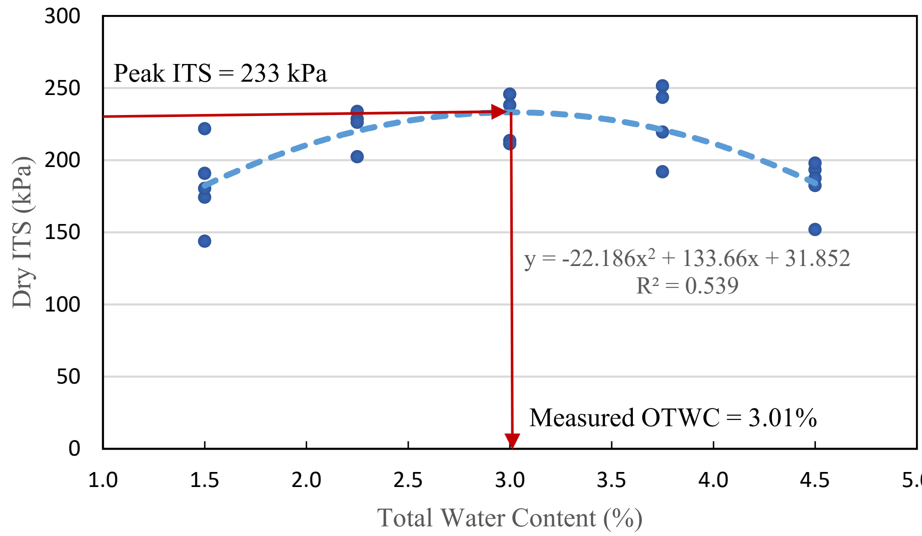


Figure 4.8 Regression Curve Built to Determine the Measured OTWC (Mix 6)

After the measured OTWC was determined using the quadratic regression, it was used with three predictors selected (RAP material's OWC, binder, and FAC) to build models by multiple linear regression. The predictors were summarized in the 2nd, 4th, and 5th column of Table 4.14. The models built were shown in Equation 4.2 and 4.3. Their adjusted R-squared are 68.2% and 89.8%, respectively. This indicated both models had a good fit. The interaction term "OWC×FAC" was included in the model 2 for binder PG 58-34 because its effect was significant.

Model 1: For mixtures with PG 67-22 (A):

$$\mathrm{OTWC} = 0.143 \times \mathrm{FAC} - 1.383 \times \mathrm{OWC} + 7.68 \tag{4.2}$$

Model 2: For mixtures with PG 58-34 (B):

$$\mathrm{OTWC} = 3.520 \times \mathrm{FAC} + 2.750 \times \mathrm{OWC} - 1.260 \times \mathrm{OWC} \times \mathrm{FAC} - 4.37 \tag{4.3}$$

The calculated OTWC in the last column of Table 4.14 was determined from the fitted models. For all six mixtures, the measured OTWC was ranged from 2.07 to 4.51% and the calculated OTWC was between 2.07 and 3.96%, slightly lower than the range of the measured OTWC.

Table 4.14 Summary of the OTWC for Each Mixture

Mix No.	Binder	RAP	OWC of RAP (%)	FAC (%)	Measured OTWC of Mix (%)	Calculated OTWC of Mix (%)
1	67-22 (A)	DF (A)	4	2	2.76	2.43
	67-22 (A)	DF (A)	4	3	2.26	2.58
2	67-22 (A)	DM (B)	3	2	3.69	3.82
	67-22 (A)	DM (B)	3	3	4.51	3.96
3	67-22 (A)	DC (C)	3	2	3.63	3.82
	67-22 (A)	DC (C)	3	3	3.74	3.96
4	58-34 (B)	DF (A)	4	2	3.59	3.59
	58-34 (B)	DF (A)	4	3	2.07	2.07
5	58-34 (B)	DM (B)	3	2	3.21	3.36
	58-34 (B)	DM (B)	3	3	3.19	3.10
6	58-34 (B)	DC (C)	3	2	3.51	3.36
	58-34 (B)	DC (C)	3	3	3.01	3.10

Figure 4.9 shows the correlation between the measured and the calculated OTWC. The data points were near the line of equality. The calculated OTWC was about 86% of the measured OTWC. A linear regression model was obtained with a good fitting (R-squared = 0.867). However, these two models were built based on the limited type of materials (RAP and Binder). Further study using more different mixtures is needed for calibration.

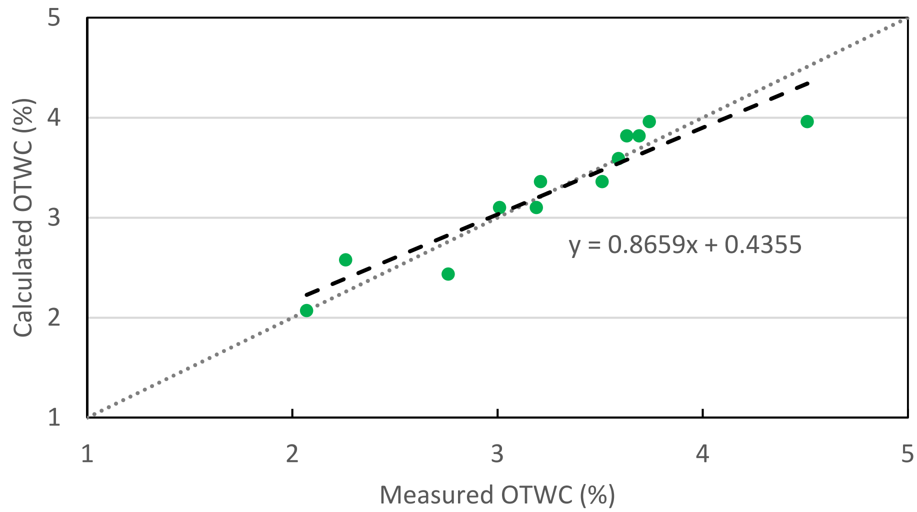


Figure 4.9 Correlation between the Measured and the Calculated OTWC

4.3.2.3 Validation

Validation Using Different Mixture Types

Table 4.15 summarizes the mixtures used for validation (No. 7 - 12). The models (Equation 4.2 and 4.3) were applied to predict the OTWC of each of these mixtures. Results of the measured OTWC, the predicted OTWC, and their differences are shown in the table. The measured OTWC was ranged from 2.74 to 5.01%, while the predicted OTWC was between 2.07 and 3.89%. The built models under-predicted the OTWC for these mixtures but the differences were less than 0.9% except for the two mixtures with binder PG 67-22(C). Differences between the measured and the predicted OTWC for both mix 9 and 10 were higher than 1.0% while mix 9 had 0.65% more. This binder had the same PG as the type A binder but from a different source. The foaming properties of type C binder such as expansion ratio (ER) and half-life (HL) were all less than type A binder. Therefore, this foamed asphalt may need more water to distribute in the mixture. Also, the models could predict the OTWC for the mixture without cement because removing cement did not significantly affect the predicting accuracy for the OTWC of mixes 11 and 12.

Table 4.15 Summary of Model Validation Results using Different Mixtures

Mix No.	Binder	RAP	FAC	Measured OTWC (%)	Predicted OTWC (%)	Difference (%)
7	58-34 (B)	VM1 (D)	2.5	3.33	2.83	0.50
8	58-34 (B)	VC1 (E)	2.5	3.43	3.23	0.20
9	67-22 (C)	VM1 (D)	2.5	4.28	2.51	1.77
10	67-22 (C)	VC1 (E)	2.5	5.01	3.89	1.12
[1]11	58-34 (B)	[1]DF (A)	3.0	2.74	2.07	0.67
[1]12	58-34 (B)	[1]DM (B)	3.0	3.98	3.10	0.88

Note: 1. No cement was added in mix 11 and 12.

The measured and predicted OTWC of the mixtures for validation are plotted in Figure 4.10. Most data points were parallel with the line of equality but were offset from it. The correlation between the measured and the predicted OTWC was not strong (R-squared = 0.533). But the regression improved considerably (R-squared = 0.853) when mix 9 was excluded. The under-prediction for two mixes (No. 9 and 10) indicate that the models may not be accurate to predict the OTWC of mixture with binder of different sources even though with the same PG. This suggests that the binder source may also affect the OTWC of mix. A factor representing the binder's foaming properties may be considered for further research. Because the proposed models generally under predicted the OTWC, an additional validation work for predicting the OTWC (after determining the optimum asphalt content) is needed.

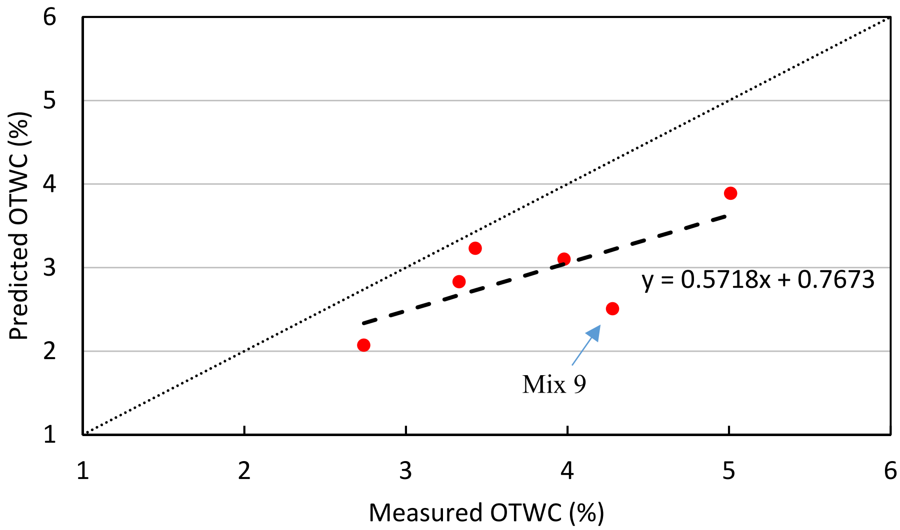


Figure 4.10 Correlation between the Measured and the Predicted OTWC

Comparison with Typical OTWC Determining Methods

Table 4.16 summarizes the predicted OTWC and the actual water content closest to the predicted OTWC for three cold recycled mixes (Mixes 5, 10, and 13). The ITS results were obtained at the actual water content closest to the predicted OTWC within ±0.6% of the predicted OTWC. Only the three mixtures had OTWC in the tested range of water content when comparing with the OTWC determining methods by ARRA (2016) and Wirtgen (2012). Both methods recommended using 100% of RAP material's OWC as OTWC, while ARRA also stated the 75% OWC may be used if excessive water was extruded out of SGC mold. The dry ITS results obtained at 75% OWC, 100% OWC, and the model-predicted OTWC were compared to validate the effectiveness of the proposed method to improve the dry ITS. The wet ITS results were also compared to confirm that the proposed OTWC determining method did not negative affect the wet ITS.

Table 4.16 Comparing the Proposed OTWC with the Method of ARRA (2016) and Wirtgen (2012)

Mix No.	Binder	RAP	FAC	Predicted OTWC (%)			Closest Actual Water Content (%)		
				75% OWC[1]	100% OWC[1, 2]	Proposed	75% OWC	100% OWC	Proposed
5	58-34(B)	DM(B)	3.0	2.9	3.8	3.1	3.0	3.8	3.0
10	67-22(C)	VC1(E)	2.5	4.0	5.3	3.9	4.1	4.9	4.1
13	67-22(C)	DF(A)	2.5	4.9	6.5	2.5	5.0	7.0	2.7

Note:

1. The 75% and 100% OWC were suggested by ARRA (2016) as the OTWC for the cold recycled mix.
2. The 100% OWC was recommended by Wirtgen GmbH (2012) as the OTWC for the cold recycled mix.

Figure 4.11 shows the comparison of the dry ITS results at OTWC levels determined by different methods. The percentage of the dry ITS change showed that the mixtures at the proposed OTWC had equal or higher ITS than the 75% OWC method suggested by ARRA (2016). Also, the dry ITS results of the mixtures at the proposed OTWC were equivalent to those of the mixtures with 100% OWC. The improvement was not evident because dry ITS of mix 10 had 19% decrease. However, t-test results showed the dry ITS results of the mixtures with the proposed OTWC were statistically the same as those with either 75% or 100% OWC (p-value = 0.426 and 0.970, respectively). Figure 4.12 shows the comparison for the wet ITS results. The comparison showed the three different methods tended to have similar wet ITS with slight differences less than 10%. T-test results also showed the wet ITS results of the mixtures with the proposed OTWC were statistically the same as other two wet ITS results (p-value = 0.812 and 0.920, respectively). This confirmed the proposed OTWC method did not negatively affect the wet ITS.

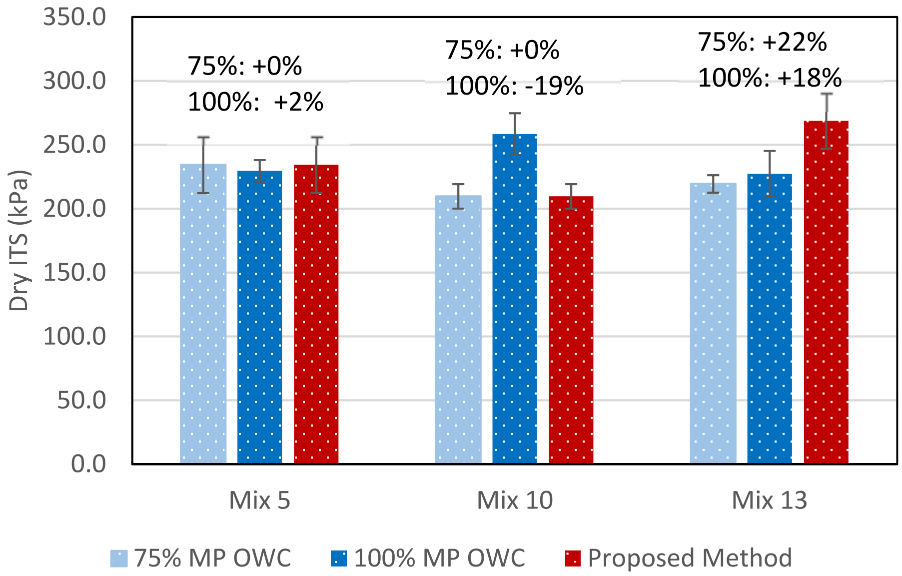


Figure 4.11 Comparison of the Dry ITS Results at the OTWC Determined by ARRA (2016), Wirtgen (2012), and the Proposed Method

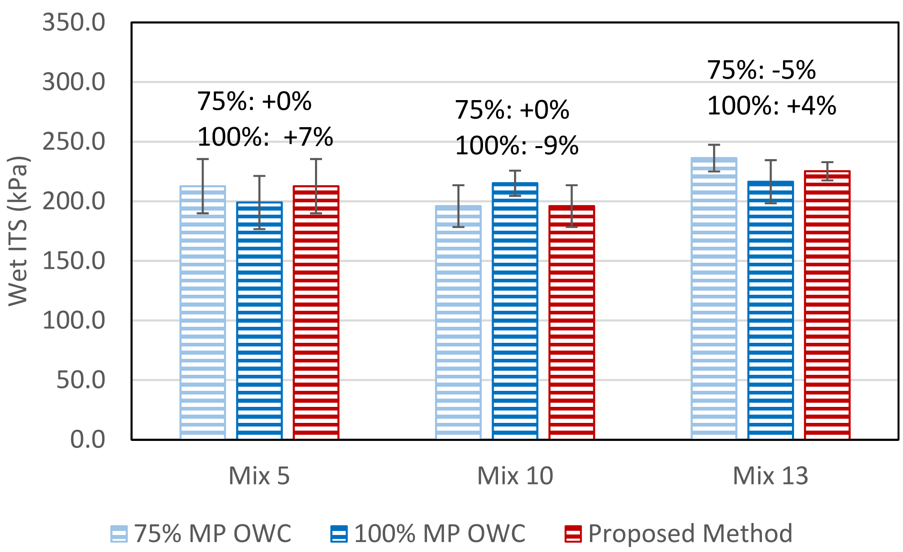


Figure 4.12 Comparison of the Wet ITS Results at the OTWC determined by ARRA (2016), Wirtgen (2012), and the Proposed Method

Table 4.17 summarizes the predicted OTWC and the actual water content closest to the predicted OTWC for six cold recycled mixes (mix 2 with two FAC, 8, 9, 10, and 13). Similarly, the ITS results were obtained at the actual water content closest to the predicted OTWC within ±0.6% of the predicted OTWC. The test results of six mixtures were used to evaluate the effectiveness of proposed OTWC determining method to improve the dry ITS when comparing with the method suggested by Wirtgen GmbH (2008).

Table 4.17 Comparing the Proposed OTWC with the Wirtgen's Method (2008)

Mix No.	Binder	RAP	FAC	Predicted OTWC (%)		Closest Actual Water Content (%)	
				Wirtgen (2008)	Proposed	Wirtgen (2008)	Proposed
2	67-22(A)	DM(B)	2.0	3.3	3.8	3.0	3.8
2	67-22(A)	DM(B)	3.0	3.3	4.0	3.0	3.8
8	58-34(B)	VC1(E)	2.5	4.3	3.2	4.3	3.5
9	67-22(C)	VF(H)	2.5	5.2	2.5	5.7	2.7
10	67-22(C)	VC1(E)	2.5	4.3	3.9	4.1	4.1
13	67-22(C)	DF(A)	2.5	5.2	2.5	5.0	2.7

Figure 4.13 show the dry ITS results of the mixtures with the proposed OTWC and those with the OTWC determined by Wirtgen GmbH (2008). The changes of dry ITS results were summarized above columns. All the six mixtures had the same or higher dry ITS at the proposed OTWC while t-test did not show significant difference between these dry ITS results at the Wirtgen's and the proposed OTWC. In addition, although the proposed models underestimated the OTWC of mix 9 and 10 more than 1.0%, their dry ITS results at the proposed OTWC was still equal to those at the OTWC determined by Wirtgen GmbH (2008).

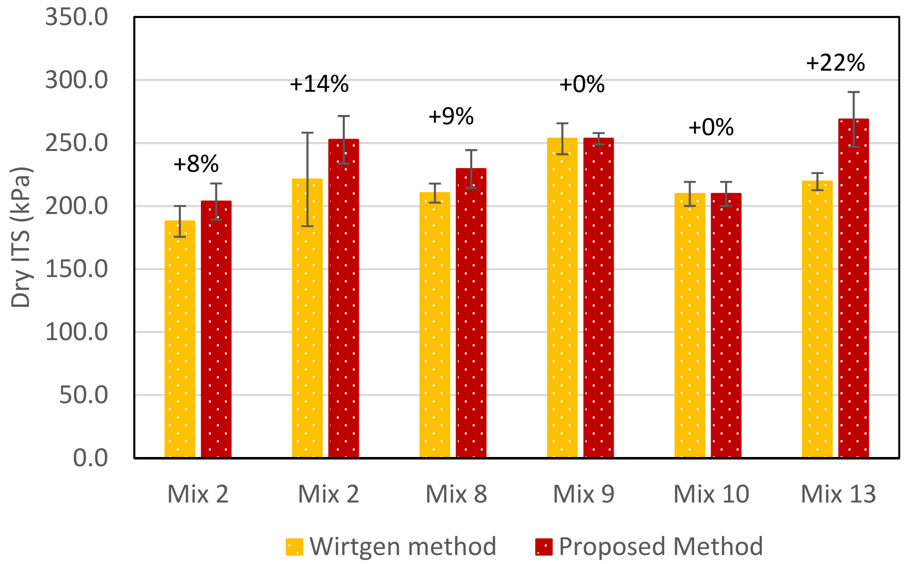


Figure 4.13 Comparison of the Dry ITS Results at OTWC determined by Wirtgen (2008) and the Proposed Method

Figure 4.14 showed the comparison of the wet ITS results at the proposed OTWC and the OTWC determined by Wirtgen GmbH (2008). Three mixtures (mix 2, 8, and 9) showed decreases at the proposed OTWC. Two mixtures (mix 2 and 13) showed increases at the proposed OTWC. One mixture (mix 10) had equal wet ITS results at the two OTWC levels. Overall, a little more decrease occurred than the increase at the proposed OTWC but the ITS results were statistically the same at two OTWC (p-value = 0.715). Therefore, this comparison showed that the OTWC did not necessarily increase the wet ITS, even though evident improvements on the dry ITS results were observed in Figure 4.13. This may be because the OTWC was proposed to reach the maximum dry ITS in this study.

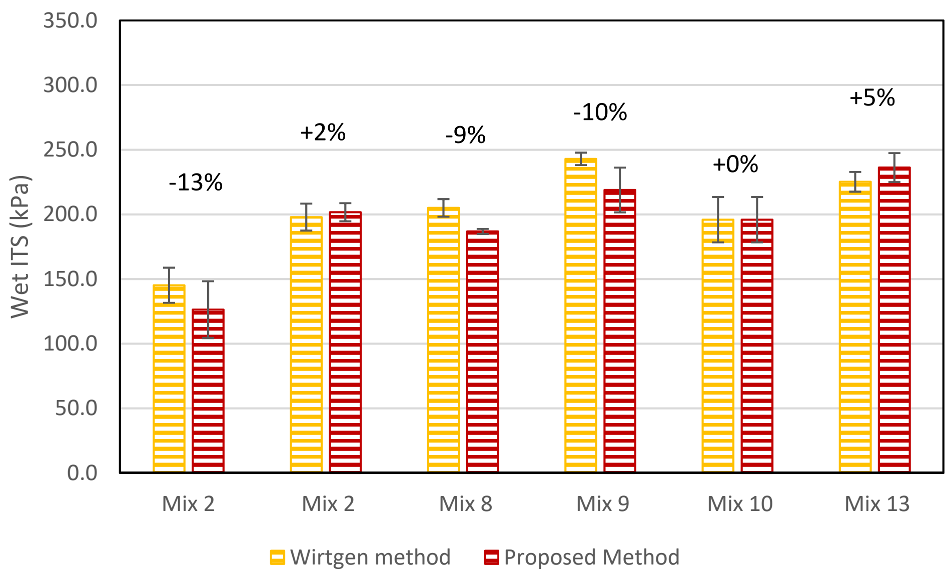


Figure 4.14 Comparison of the Wet ITS Results at OTWC determined by Wirtgen (2008) and the Proposed Method

4.4 Proposed Mix Design Method

The basic steps of the mix design method after the proposed improvements are plotted in Figure 4.15. The first and second improvements were ready to be implemented but the third improvement to the OTWC determining method may not be ready to use. The objective of the third improvement was to determine the OTWC of mix directly using the proposed models. However, the models built in this study still need calibration using different binder types and RAP materials. Therefore, an additional step was suggested to verify the model-determined OTWC. The ITS result at the final OTWC is expected to be highest. Discussion of each step in the flowchart follows.

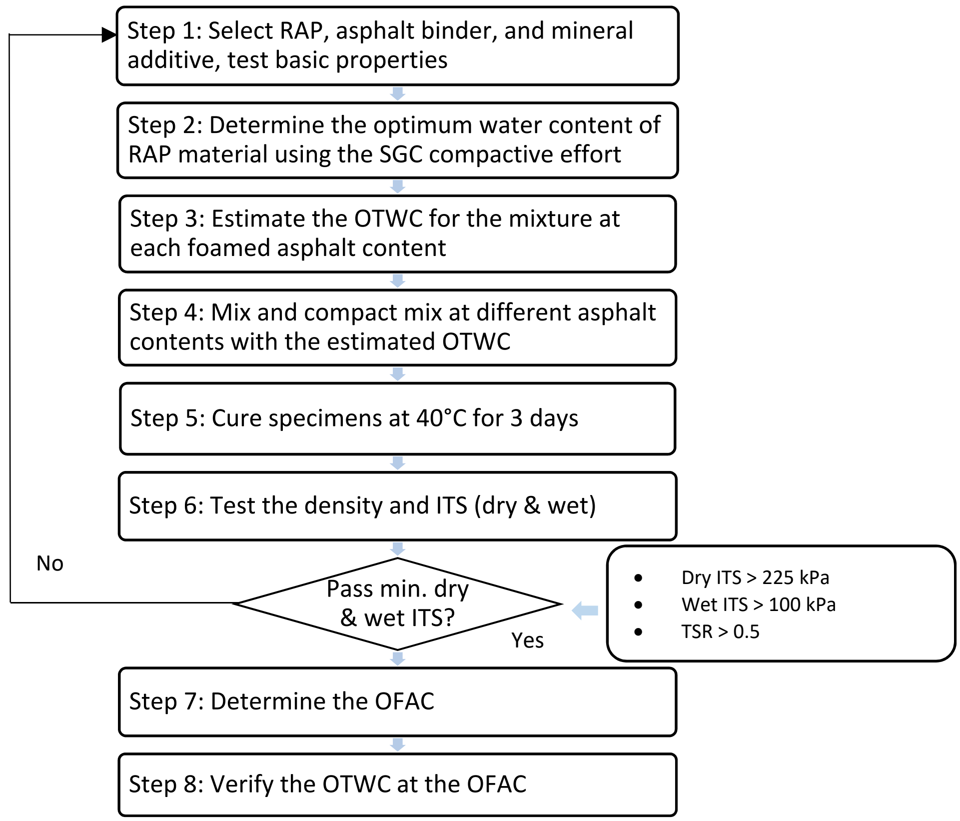


Figure 4.15 Flowchart of the Mix Design Procedures after Improvements

Step 1: Select RAP, asphalt binder, and mineral additive; test the basic properties.

RAP materials were sampled from stockpiles, which had been milled, crushed, and screened before stockpiling. After sampling, the RAP was stored in buckets and then dried in pans in front of a fan for a week before mixing. A mechanical splitter was used to reduce RAP to testing size following AASHTO R47. Gradation, binder content, and binder performance grade of the RAP were also tested. The RAP gradation range recommended by Wirtgen (2012) may be used as a reference to select RAP materials. But the available RAP materials with gradation slightly out of the range may be still good to use in mix design.

After RAP materials are prepared, virgin asphalt was heated and poured into 1-gallon cans for ease of handling for foaming properties testing. The expansion ratio (ER) and half-life (HL) were tested using the Wirtgen WLB10S laboratory plant (as shown in Figure 4.16) based on the method suggested by Wirtgen GmbH (2012). Also, the optimum foaming water content selected to produce foamed asphalt was determined to maximize the two foaming qualities (ER and HL). The temperature used to heat the binder for foaming should not exceed 180°C. Use of relatively lower temperature is preferred once the criteria of foaming qualities were satisfied for it is easier to achieve during the construction. The acceptable asphalt to produce foamed asphalt need to have at least 8 times expansion ratio and 6 seconds' half-life.

Figure 4.16 Laboratory Pug Mill (Left) and Foaming Plant (Right)

Step 2: Determine the optimum water content of RAP material using the SGC compactive effort

Using the method similar to the modified Proctor test (AASHTO T180), RAP material was prepared and compacted using 30 gyrations by the Superpave gyratory compactor. The dry density at different water content was determined. A compaction curve was obtained to determine the optimum water content (OWC) for the RAP corresponding to its peak dry density.

Step 3: Estimate the OTWC for the Mixture at each foamed asphalt content

The OTWC can be predicted by the models in Equation 4.2 and 4.3 based on the OWC of RAP material determined in step 2, binder PG, and FAC. The predicted OTWC corresponded to the FAC levels to be used in step 4. Because the models still need calibration using more binders, the calculated OTWC is only an estimate based on limited binder types. The estimated result may be used as an initial water content to prepare specimens when selecting the optimum foamed asphalt content (OFAC) in step 7. Literature review showed that the typical binders used to produce foamed asphalt in cold recycling had a narrow range of high-temp PG (i.e., 52, 58, and 64). The first model based on the PG 67-22 binder may be used to estimate the OTWC of the mixes containing binder with high-temperature PG equal to or higher than 64. The second model based on the PG 58-34 binder may be used for the mixes containing binder with high-temperature PG equal to or lower than 58. The calculated OTWC from this step needs to be verified in step 8 after the OFAC is determined.

Step 4: Mixing and Compacting

RAP was mixed with cement, water, and foamed asphalt in a laboratory pug mill as shown in Figure 4.16. RAP material was added first, then mineral additive, followed by water and foamed asphalt at last. Added water helps to disperse foamed asphalt in the RAP and aid compaction of the mixture (ARRA 2014). The twin-shaft pug mill has 10 - 30 kg mixing capacity. At each time, 10 - 15 kg RAP was added for mixing. Mixing speed and mixing time are adjustable. This study used a medium driving speed (about 72 rpm) and a 60-second mixing time for each mixture. Wirtgen GmbH (2008) recommended a 30-second mixing time for aggregate/RAP blending with cement and water. Therefore, a 15-second pre-mixing time was used for both cement and water to prevent non-uniformity of these two non-asphalt materials in the mixture.

Different laboratory compactive efforts have been suggested for cold recycled foamed asphalt mixture as discussed in the literature. The most common method in the U.S. and Europe is compacting samples with 75 blows of a Marshall Hammer on each side. Some researchers also recommended using the modified Proctor compactive effort (ASTM D1557). But its effort was found to have lower result in densities than field compaction (Schwartz and Khosravifar, 2013). The Superpave Gyratory Compactor (SGC) became much more widely available in laboratories. ARRA (2016) had suggested using SGC 30 gyrations as compactive effort in the mix design process but this effort was originally determined for cold recycled asphalt emulsion mixtures. This study verified this compactive effort for the foam mixes that specimens compacted by this method had similar density with the field compacted density. The determination process will be discussed later in this dissertation. Specimens used for mix design were compacted to 100-mm diameter and 63.5 ± 2.5 mm height. This compactive effort is being validated in an ongoing study at NCAT that will determine if the compacted dry density produced by the SGC and the Marshall method are equivalent.

Step 5: Cure specimens at 40°C for 3 days

The curing procedure used in this study followed the Wirtgen cold recycling manual (Wirtgen GmbH 2012) and the ARRA guidelines (2016). Compacted specimens were cured in oven at 40°C immediately after compaction. Curing time was 3 days for all specimens. This curing method helps specimens develop strength at a faster rate than ambient-temperature curing. Therefore, it was considered accelerated curing.

Step 6: Test Density and Indirect Tensile Strengths

After curing, specimens were cooled at room temperature overnight before taking any measurements or conducting any tests. Then, the specimens were measured and tested for dry density and the ITS.

Density Test

Two methods were used for density test. The first method estimated the density by the dry mass and dimensions following Equation 4.4. Although it is a quick way to determine the density for each specimen, the uneven surface of the specimen may cause biased measurements which may influence the estimated density and the ITS result. The Superpave gyratory compactor (SGC) may automatically record specimen heights during compaction, but the recorded measurements from SGC were found 1 - 2-mm smaller than the measurement using caliper after curing. Therefore, the caliper measurement of height (average of four different points around the specimen) was recommended for determining density in Equation 4.4. This density was used as an indicator to check the uniformity of the compacted specimens and to group the specimens for the dry and wet ITS test. This allowed each group to have same level of variability.

$$\rho_d = \frac{M_d}{\pi d^2 h/4} \tag{4.4}$$

Where:

ρ_d= dry density of compacted specimen (kg/m^3);

M_d= dry mass of compacted specimen (kg);

d= diameter (m);

h= height (m).

The second method used an automatic vacuum sealing system (as shown in Figure 4.14) to seal the specimen tightly and measure its bulk density using the water displacement method (AASHTO T331). Compared to the typical Saturated Surface-Dry method (AASHTO T166), this method avoided potential moisture damage to the specimen and saved specimen drying time.

(a) Bulk specific gravity measurement (G_{mb}) (b) Theoretical maximum specific gravity (G_{mm})

Figure 4.17 Density Measurements Using the Automatic Sealing Device

The theoretical maximum specific gravity (G_{mm}) was not directly used in the mix design procedure for selecting the suitable materials, determining the OTWC or the optimum foamed asphalt content. But this would be useful to estimate the volumetric properties of specimens such as VMA and VFA for laboratory testing. The value of G_{mm} may be measured following AASHTO T209 standard where loose mix was vacuumed and agitated before the water-displacement procedure. During the test, there might be some fine particles floating on the surface of water after agitation. But a little droplet of surfactant (e.g., aerosol OT solution) can be used to help the sedimentation these fines. However, some RAP may contain porous aggregates and require additional dry-back procedure when absorption was higher than 1.5%. This dry-back procedure was time consuming because continuous weight tracking was required until the sample became dry. Instead, this study tested the G_{mm} using the automatic vacuum sealing method following ASTM D6857. It allowed each test to be completed in 6 - 8 minutes without concern about the aggregate absorption under vacuum condition in the typical method (i.e., AASHTO T209).

Indirect Tensile Strength Test

The ITS test was conducted for each mixture in both dry and wet conditions. A simple load machine was used to apply monotonic load at a speed of 50 mm/min along the diameter of specimens as shown in Figure 4.15. The peak load was recorded to determine ITS. Before testing, the dry specimens were cured at 25°C for one hour and the wet specimens were cured in a water bath at 25°C for 24 ± 2 hours. The wet ITS testing method was recommended by Wirtgen GmbH (2012).

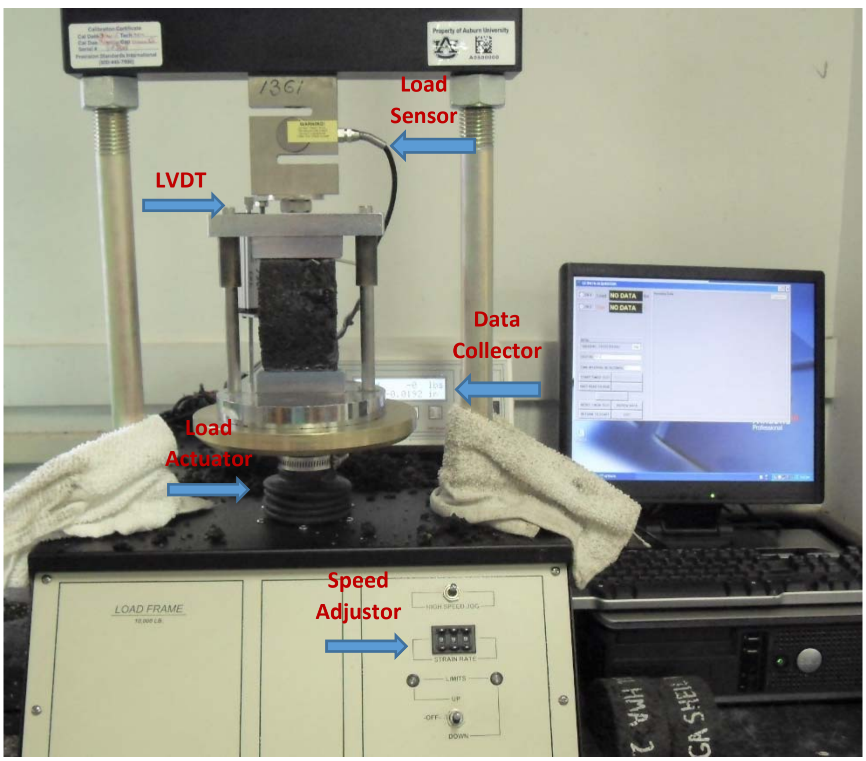


Figure 4.18 Indirect Tensile Strength Test Setup

Step 7: Determine the Optimum Foamed Asphalt Content

Based on the tested dry and wet ITS results at different foamed asphalt contents. The minimum foamed asphalt content able to achieve the ITS criteria was selected as the optimum foamed asphalt content. These criteria (dry ITS > 225 kPa; wet ITS > 100 kPa; TSR > 0.5) suggested by Wirtgen GmbH (2012) were adopted even though they were lower than those recommended by VDOT (2015) and ARRA (2016) (min. dry ITS > 310 kPa; TSR > 0.7). The available data from this study have shown that the dry ITS criterion of 310 kPa was difficult to achieve for most of the design mixes (No. 1 - 6) based on the proposed compactive effort when typical FAC was used (2 - 3%). The ITS criteria of Wirtgen can be achieved for these mixes when FAC was in the same range.

Step 8: Verify the OTWC at the Selected OFAC

The proposed OTWC determining models tended to underestimate the actual OTWC. Once the OFAC is determined from step 7. Additional specimens at other four water contents (OTWC ± 1% & ±2%) may be used to determine the final OTWC with the highest dry ITS result.

4.5 Summary

This chapter discussed three improvements to the mix design process for the cold recycled foamed asphalt mixtures. The improvements were focused on laboratory compactive effort, RAP material's OWC determining method, and the mixture's OTWC determining method. Based on the results obtained from this study, the findings and conclusions were summarized below.

- The compactive effort of 30 gyrations using Superpave gyratory compactor was validated as it was close to the field compacted density. Since there was a lack of field compacted and laboratory SGC-compacted densities build correlation, the Marshall compactive effort was used to relate with both field and SGC-compacted densities, through which the relationship between field density and the SGC-compacted density could be correlated.
- A different method to determine the OWC of RAP material was proposed based on the validated compactive effort using SGC. This method allows designers to use same compactive effort for RAP and the mixture. Also, the proposed SGC-based method can save cost to purchase the mechanical modified Proctor testing equipment because it may not be available in typical asphalt laboratories. Use of SGC-based method can also save labor or testing time if the manual version of the testing equipment is used. Compared to the modified Proctor test method following AASHTO T180), the

OWC of RAP material determined by the proposed method was lower and the determined maximum dry densities determined by the proposed method were also slightly lower.

- Two models for determining the OTWC of the cold recycled foamed asphalt mixtures were built based on six cold recycled mixes with two different virgin binders and three RAP materials. The models were built by multiple linear regression for the measured OTWC results and three predictors including RAP material's OWC, virgin binder PG, and FAC. The predictive capability of the models was validated using six different mixtures. The predicted OTWC were lower than the measured OTWC. Two of the six mixtures under-predicted the OTWC more than 1.0% due to the use of binder from another source. This indicated that the current way to distinguish foamed asphalt based on the virgin binder PG was not enough. A single factor to indicate the foaming properties of binder may be used to calibrate the models in the future. In addition, comparisons were conducted for the dry ITS between the proposed OTWC and the OTWC determined from a typical design method used by Wirtgen GmbH (2008). All the mixtures had the same or improved dry ITS results at the proposed OTWC.

CHAPTER 5 MINERAL ADDITIVE STUDY

The primary objective of this study was to determine the effects of different mineral additives on the ITS results of cold recycled foamed asphalt mixtures in laboratory and field curing conditions. Effects of additive type, additive content, and curing moisture were studied based on test results after laboratory curing conditions. A secondary objective was to evaluate the effects of curing time, top load and confinement based on the results after simulated field curing.

5.1 Methodology

Specimens were divided into five curing schemes including three laboratory curing conditions and two simulated field curing conditions, as shown in Table 5.1. The ITS results of three replicates were determined for each mixture after each curing scheme. Cement and hydrated lime were added at three levels (by weight of dry RAP). A maximum of 2.0% was used for these two additives because excessive content may be detrimental to the mixture's flexibility (Wirtgen GmbH 2012). Mixtures with fly ash and baghouse fines may only gain limited strengths during curing, thus these two additives were added up to 5.0 percent to explore the effect of additive content assuming theses higher contents would have little influence on flexibility. Additives were added uniformly and mixed sufficiently with RAP in a laboratory twin-shaft pug mill.

Scheme 1 is the most commonly used laboratory curing method for cold recycled foamed asphalt mixtures. It consisted of curing specimens in a convection oven at 40°C for 3 days. Effects of additive type and additive content were determined for this scheme. Effect of curing moisture was determined by comparing the dry ITS of specimens cured in scheme 2 and 3. Each scheme consisted of an initial curing (40°C for 3 days) and an extended curing in either dry or moisture room. After initial curing, specimens were cured in a dry laboratory room (about 50% relative humidity) without moisture under scheme 2. For scheme 3, specimens were cured from the same batch in a moisture room where water vapor was constantly provided (around 98% relative humidity). The temperature in both dry and moisture rooms were similar (around 22°C). This initial curing dried specimens and helped developing initial strengths to avoid damage during handling. However, in this step, moisture in the specimens evaporates quickly limiting the reaction between water and additives. Cardone et al. (2015) suggests that these incomplete reactions may not adequately represent the material's strength. In scheme 4 and 5, specimens were placed outdoor on a pallet with sufficient sunlight and wind to simulate field curing. The top load and confinement were used to simulate an asphalt overlay on the recycled pavement. Figure 5.1 shows the two schemes in the field curing condition.

Table 5.1 Experiment Plan for Mineral Additive Study

<table>
<tr><th rowspan="3">Additive</th><th rowspan="3">Content (%)</th><th colspan="5">Curing Schemes</th></tr>
<tr><th colspan="3">Lab[1]</th><th colspan="2">Field[2, 3, 4]</th></tr>
<tr><th>1</th><th>2</th><th>3</th><th>4</th><th>5</th></tr>
<tr><td>Control (CTRL)</td><td>—</td><td rowspan="5">3 days at 40°C</td><td rowspan="5">3 days at 40°C + 30 days in dry room</td><td rowspan="5">3 days at 40°C + 30 days in moist room</td><td rowspan="5">3, 14, 30, 100, (120) days outdoor</td><td rowspan="5">100 days outdoor with top load and confinement</td></tr>
<tr><td>Cement (CEM)</td><td>1.0, 1.5, 2.0</td></tr>
<tr><td>Hydrated Lime (HL)</td><td>1.0, 1.5, 2.0</td></tr>
<tr><td>Fly Ash (FA)</td><td>1.0, 1.5, 2.0, 5.0</td></tr>
<tr><td>Baghouse Fine (BHF)</td><td>1.0, 1.5, 2.0, 5.0</td></tr>
</table>

Notes:

1. After curing following scheme 3, the specimens were dried before testing ITS.
2. Scheme 4 is an outdoor environment in Auburn, Alabama from November 2016 to February 2017; it only applies for the mixtures with 1.0% additive. The specimens were covered on rainy days.
3. Specimens cured for 120 days in scheme 4 were tested for wet ITS only to evaluate the moisture susceptibility.
4. Wired mesh was tied around each specimen by hands in a consistent manner to provide confinement.

(a) Specimens without top load and confinement (b) Specimens with top load and confinement

Figure 5.1 Field Curing Condition

5.2 Results and Discussions

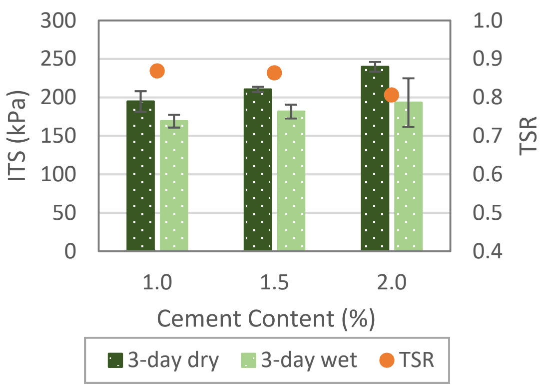
5.2.1 Laboratory-Curing

5.2.1.1 Effects of Additive Type and Additive Content

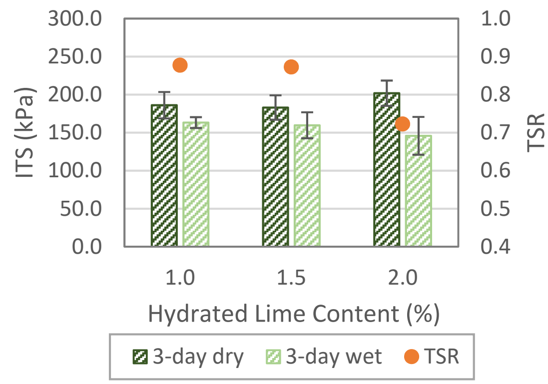
Figure 5.2 shows the average dry and wet ITS results of specimens cured at 40 °C for 3 days (scheme 1). Whiskers represent one standard deviation above and below the average. Average dry ITS results ranged from 149.5 kPa to 239.7 kPa. The wet ITS results ranged from 107.7 kPa to 193.2 kPa. TSR values were between 0.50 and 0.93. The dry ITS were higher than the wet ITS in each group of mixtures.

Figure 5.2(a) shows the results of mixtures with cement at 1.0, 1.5, and 2.0%. Dry and wet ITS increased as more cement was added. The analysis of variance (ANOVA) showed that both cement content and test condition (dry vs. wet) were significant effects on ITS (p-values 0.007 and 0.001, respectively). A significance level (α) of 0.05 was used (Montgomery 2009). Figure 5.2(b) shows results of mixtures with hydrated lime. Dry ITS results were a little higher at 2.0% than at other contents. Wet ITS slightly decreased as more hydrated lime was added. ANOVA results indicated that the effect of hydrated lime content was not significant but the testing condition was (p-values 0.945 and 0.001, respectively). Figure 5.2(c) shows ITS results of specimens with baghouse fines. Dry ITS increased with baghouse fines content from 1.0% to 2.0%, but decreased at 5.0% content. The TSR results ranged from 0.50 to 0.69, below that of cement (0.81 – 0.93) and hydrated lime (0.72 – 0.88). ANOVA on the results of three baghouse fines contents (1.0 - 2.0%) showed that baghouse fines content was not a significant factor but the testing condition was (p-values 0.079 and < 0.001, respectively). Figure 5.2(d) illustrates the ITS results of mixtures with fly ash. Either dry ITS or wet ITS result with no more than 2.0% fly ash was lower than that at 5.0% content. Dry ITS decreased at 1.5% and increased at 2.0%. Wet ITS changed a little in the range of 1.0 to 2.0%. The TSR result was higher than that of baghouse fines. ANOVA on the results of three fly ash contents (1.0 - 2.0%) showed the effect of the content was not significant on the dry and wet ITS but the effect of testing condition was (p-values 0.237 and < 0.001, respectively). Figure 5.2(e) shows the TSR value of the control mixture was only 0.60, indicating the recycled mixture with no additive was moisture susceptible. T-test result verified the significant difference between dry and wet ITS of the control mixture (p-value 0.005) (Montgomery 2009).

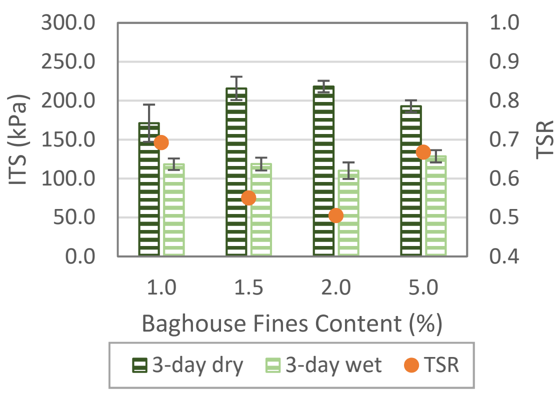
The relative effects of the mineral additives follow expected trends. Cement reacts with water (hydrates)to form hydration products that bind together and give the strength to the material. Class-C fly ash, a non-engineered by-product of burning coal, has a similar hydration reaction as cement, but the reaction products are weaker and strength gains are much less. Hydrated lime has a “asphalt-lime reaction” where lime particles adsorb polar components of asphalt (Little and Epps 2001). Baghouse fines have no chemical reaction and (inert filler) that may only provide strength by stiffening the foamed asphalt binder. For the reactive mineral additives, higher contents were expected to improve dry and wet ITS results of the cold recycled mixtures. For dry ITS, this trend of cement, hydrated lime, fly ash was apparent, even though only the effect of cement was statistically significant. The decrease of dry ITS at 5.0% baghouse fines may be due to non-uniform distribution of fines during mixing. For wet ITS, only the mix with cement increased the wet ITS. No improvement in wet ITS was evident for the other additives. The specimens were tested after soaking for 24 hours. Only cement was observed to effectively reduce air voids within the specimen because water only infiltrated into the outer layer of the specimen. Specimens with other additives were completely wet after soaking. Therefore, their wet ITS results were not improved and not depend on the change of additive content.



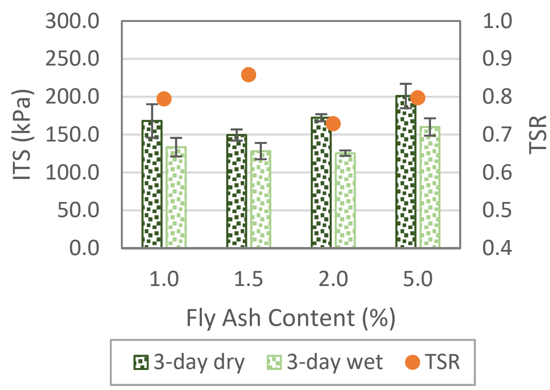
(a) Cement



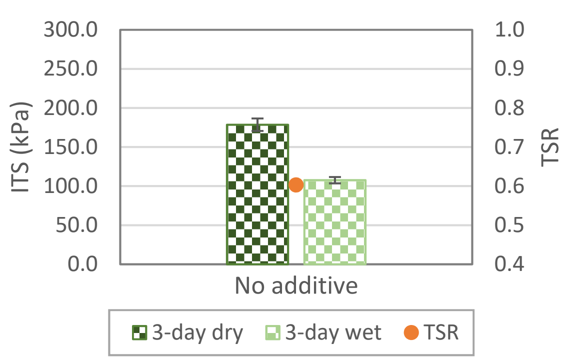
(b) Hydrated Lime



(c) Baghouse Fines



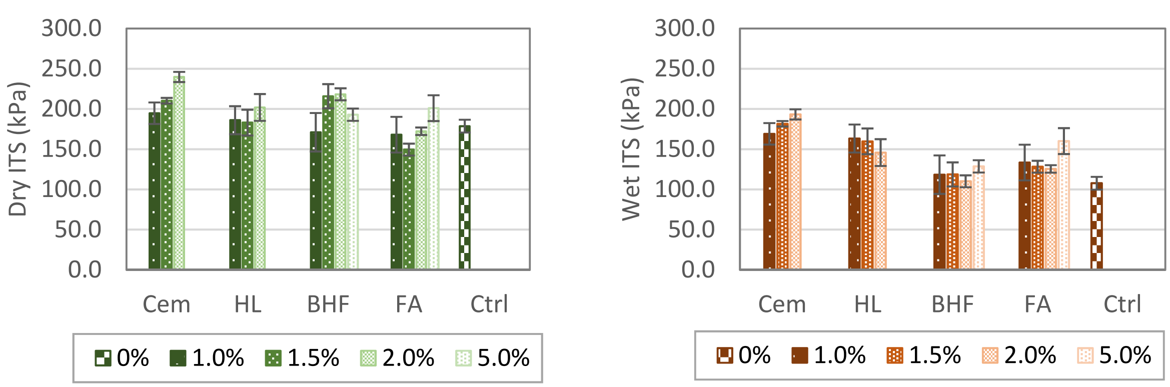
(d) Fly Ash



(e) Control

Figure 5.2 Comparison of the ITS of Each Mixture (Scheme 1)

To compare the effects of different additives further, the dry and wet ITS results are shown separately in Figure 5.3. Figure 5.3(a) summarizes the dry ITS of the five mixtures' groups. Mixtures with cement (CEM), hydrated lime (HL), and baghouse fines (BHF) had higher dry ITS than those with fly ash (FA) and the control mixture (CTRL). At 1.0% additive, the mixture with cement had the highest ITS, followed by mixtures with hydrated lime, baghouse fines, and fly ash. At 1.5% additive, the mixture with baghouse fines had the highest ITS. At 2.0% additive, the mixture with cement was again the highest. In addition, the increase of baghouse fines content to 5.0% did not improve dry ITS, but the increase of fly ash to 5.0% did. The dry ITS of the control mixture was close to dry ITS of the mixtures with 1.0% baghouse fines and fly ash. Figure 5.3(b) shows that the wet ITS results differed more among the five mixtures than those of dry ITS results. Mixtures with cement had the highest wet ITS at all three contents, followed by the mixtures with hydrated lime, fly ash, and baghouse fines. The rankings of wet ITS results at 1.0, 1.5, and 2.0% additive contents were the same, indicating adding more than 1.0% did not significantly improve the moisture damage resistance of the cold recycled mixture. However, all of the additives had higher wet ITS than the control mix, indicating the benefit of adding at least 1.0% additive to improve moisture damage resistance. Overall, mixtures with cement performed best in both dry and wet conditions, followed by hydrated lime, fly ash, and baghouse fines. Mixtures with baghouse fines may have comparable dry ITS but did not provide resistance much to moisture damage. The rankings of mixtures' dry and wet ITS with different additives were summarized in Table B.2 in Appendix B.



(a) Dry ITS

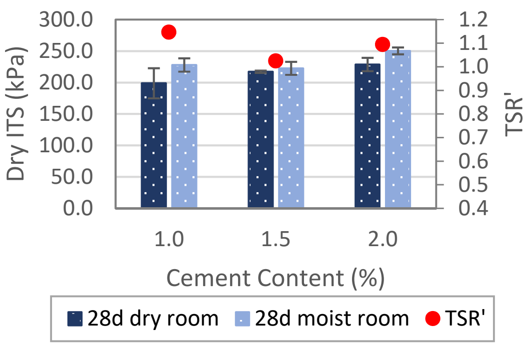
(b) Wet ITS

Figure 5.3 Comparing the Dry and Wet ITS of Mixtures Separately (Scheme 1)

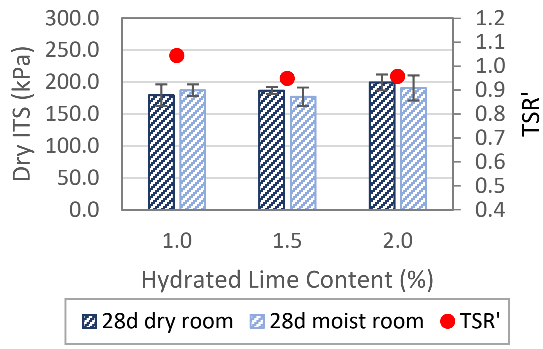
5.2.1.2 Effect of Curing Moisture

Figure 5.4 shows the comparison of ITS results of specimens cured in dry and moisture rooms, as well as the tensile strength ratio calculated as the ratio of the dry ITS results from the two curing conditions (hereafter referred to as TSR'). After moisture room curing, specimens with cement have higher ITS than other mixtures. The values of TSR' for the mixture with baghouse fines are lower than those of other mixtures suggesting the potential problem of using baghouse fines when mixtures are exposed to excessive moisture.

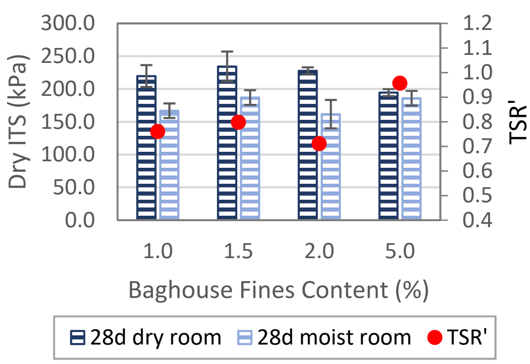
Figure 5.4(a) shows the dry ITS results of mixtures with cement after curing in dry and moisture rooms. The dry ITS increased as more cement was added. ANOVA results showed that both cement content and curing moisture have significant effects on ITS (both effects had the same p-value equals to 0.004). It means the specimens with cement cured in moisture room had statistically higher ITS than those cured in dry room. This result indicated that part of the un-hydrated cement remaining after initial curing reacted and caused the dry ITS increase during moisture room curing. Figure 5.4(b) shows only 1.0% hydrated lime specimens cured in a moisture room had slightly higher dry ITS than those specimens cured in dry room. ANOVA showed the effects of both hydrated lime content and curing moisture were not significant (p-values 0.235 and 0.599, respectively). For baghouse fines, the comparisons of dry ITS for dry and moist curing is shown in Figure 5.4(c). Moisture room curing had lower dry ITS at all four additive contents. Figure 5.4(d) shows results for mixtures with fly ash. Statistical analysis indicated that the type of curing was not significant (p-value 0.908). Figure 5.4(e) shows that moist curing for the control mix also resulted in lower dry ITS. However, a t-test showed that the ITS results for the two curing methods were not statistically different (p-value 0.198).



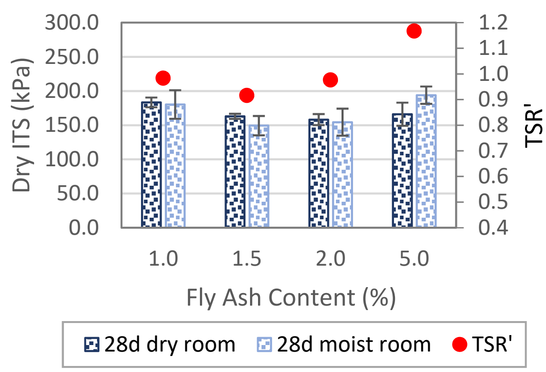
(a) Cement



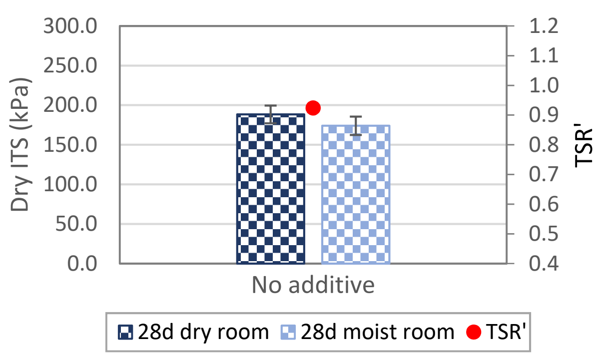
(b) Hydrated Lime



(c) Baghouse Fines



(d) Fly Ash



(e) Control

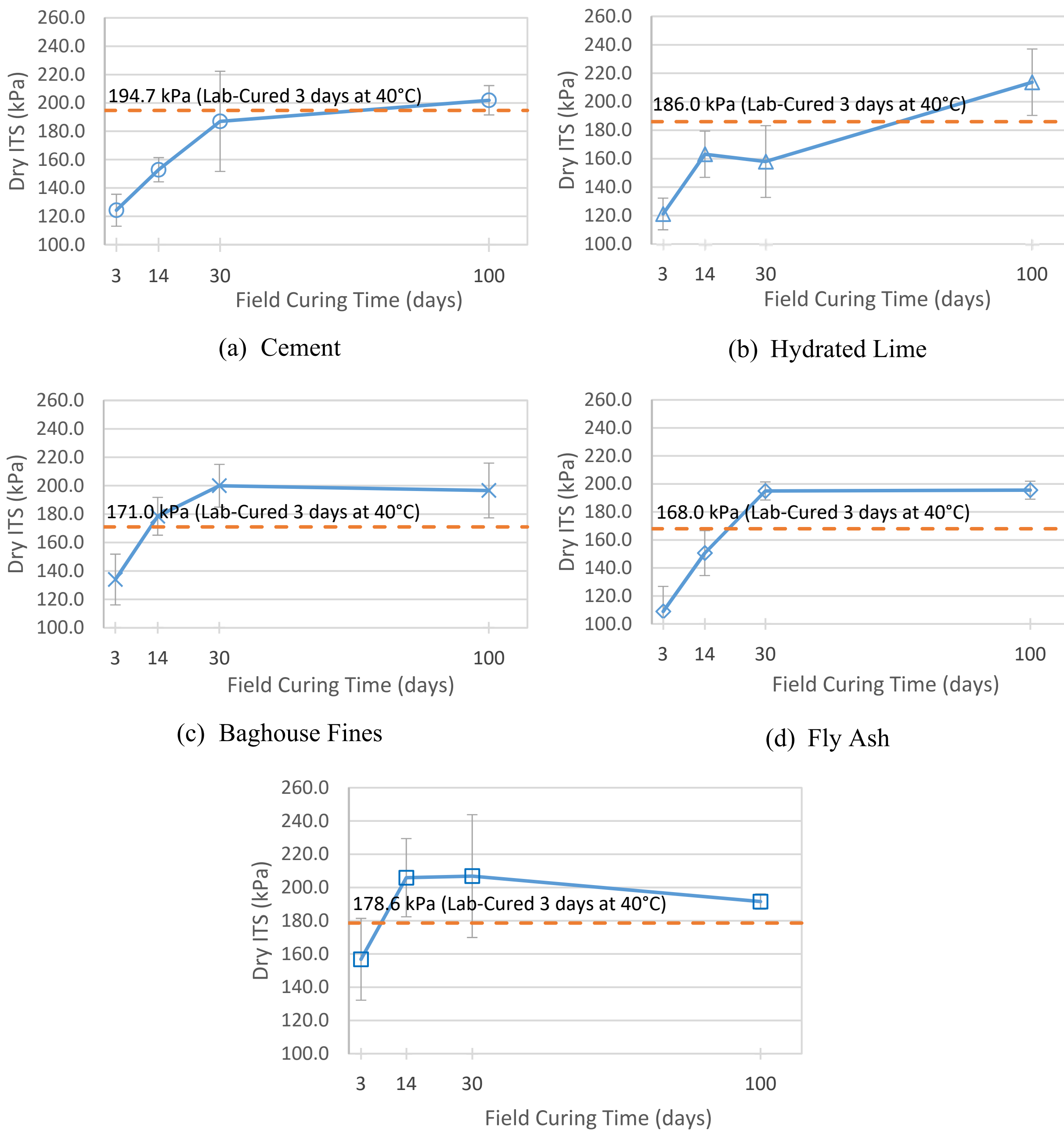
Figure 5.4 Comparison of Dry ITS of the Mixtures Cured in Dry and Moisture Rooms (Schemes 2 and 3)

5.2.2 Field-Curing

5.2.2.1 Effect of Curing Time

The effect of field curing time was evaluated by measuring the dry ITS of specimens cured for different periods of time (3, 14, 30, and 100 days) and wet ITS after 120 days. For specimens cured longer than 14 days, the exact testing date was controlled within ±3 days of the plan. For those cured for 100 days, testing date was controlled within ±5 days of the plan. Specimens with same additive were produced from one batch. Figure 5.5 shows the development of cured dry ITS for the five mixtures. The coefficients of variance of the dry ITS results at different testing dates were between 6.3% to 12.7%. The dashed horizontal line in each plot indicates dry ITS tested after curing scheme 1 (3 days at 40°C). The error bars show that results of the control mixture had higher variability than the results of the mixtures with mineral additives.

Figure 5.5(a) shows the dry ITS of the mixture with cement. Approximately 92% of the 100-day dry ITS occurred during the first 30 days. The dry ITS after curing at 40°C for 3 days was between the 30-day and 100-day field curing strength. Figure 5.5(b) shows that the dry ITS of mixture with hydrated lime increases sharply from 3 to 14 days, and then decreased slightly after 30 days. This unexpected lower dry ITS may be due to a sampling error because the specimen preparing procedures (mixing and compaction) and curing conditions were the same. A greater increase in the dry ITS occurred between 30 days and 100 days than did in the first two weeks. The laboratory-cured dry ITS was between the 30-day and the 100-day field-cured strengths. Figure 5.5(c) shows the dry ITS of mixtures with baghouse fines increased rapidly within first two weeks, then followed by a lesser increase until 30 days. The laboratory-cured specimens had dry ITS value close to the 14-day field-cured specimens. The dry ITS of the mixtures with fly ash is shown in Figure 5.5(d). The dry ITS increased at a higher rate in the first month and then was consistent after the first month. Laboratory-cured specimens had dry ITS value slightly higher than 14-day field-cured specimens. Figure 5.5(e) shows that the dry ITS of the control mixture increased in the first two weeks, stayed constant after until 30 days, and decreased a little afterwards. However, comparison between 30- and 100-day dry ITS results by t-test showed these two did not have a statistically significant difference (p-value 0.550). The unexpected decrease from 30 days to 100 days may have been due to material variability. For the control mixture, laboratory-cured dry ITS was equivalent to the field-cured strength between 3 days and 14 days.



(a) Cement

(b) Hydrated Lime

(c) Baghouse Fines

(d) Fly Ash

(e) Control

Figure 5.5 Dry ITS of Mixtures after Different Periods of Field Curing (Scheme 4)

Furthermore, Figure 5.6 shows dry ITS of these five mixtures after field curing for convenience of comparing. Each line represents development of dry ITS for a mixture with one additive. All of the mixtures have sharp increase of dry ITS from 3 to 14 days in similar rates. Then, most mixtures (except the mixture with hydrated lime) had slower strength increase from 14 to 30 days. Mixtures with cement and fly ash continued the rapid strength gain until 30 days. The mixture with baghouse fines had a lower rate of increase from 14 to 30 days. The control mixture reached its ultimate strength by 14 days. At 30-days, the strengths for all of the mixtures were similar except for hydrated lime which had unexpectedly low ITS results at 30 days. After 100 days, all five mixtures had similar dry ITS results in the range of 192 - 202 kPa. All the mixtures with additives had higher dry ITS than the control mixture. Figure 5.6 generally shows that strength gain of cold recycled mixtures is slow with most of the ultimate strength achieved around 30 days. Slower strength gain in the field may be due to temperatures during field-curing which were lower than for the consistent 40°C lab-curing condition. All physical and chemical reactions to form internal bonding happened slow. Control mix had better short-term strength than those with additives. It may be because mineral additives coated RAP surface initially, reducing the bonding of foamed asphalt between RAP particles.

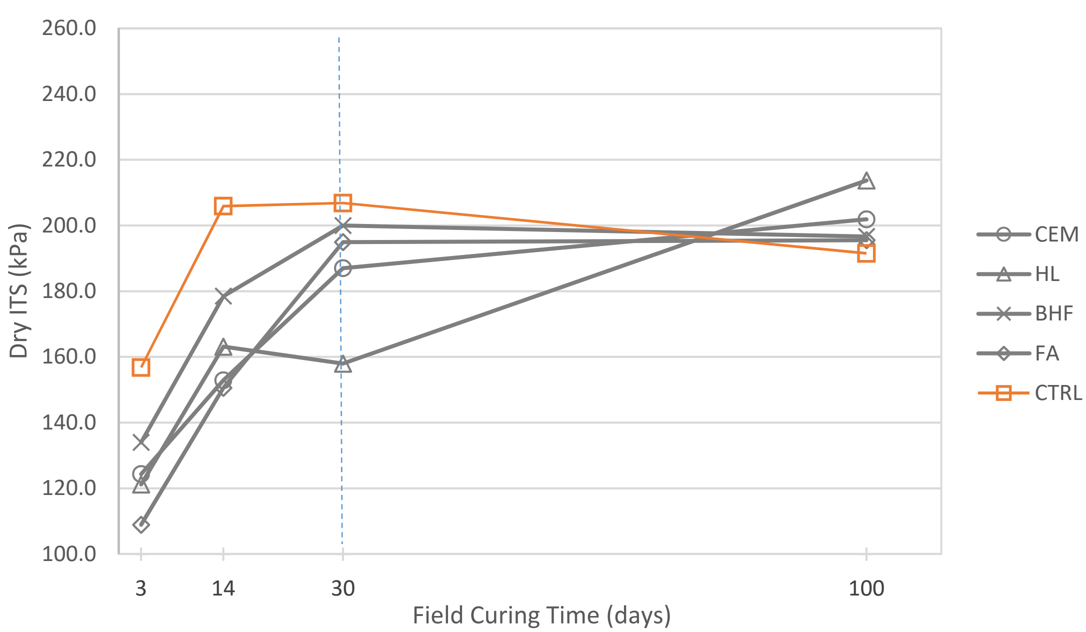


Figure 5.6 Comparison of Dry ITS for Mixtures with Different Additives after Field Curing (Scheme 4)

The five mixtures were ranked numerically and compared pairwise based on dry ITS results after every field curing period. The ranking and comparison results from Tukey's test (Montgomery 2017) are summarized in Table 5.2. Considerable differences in numerical rankings exist for the mixtures with hydrated lime (from 5th to 1st), control mixture (from 1st to 5th), and cement (from 4th to 2nd). The 100-day ranking of these mixtures is close to the numerical ranking of laboratory-cured mixtures (1.0% additive) (shown in Table B.2 of Appendices). Numerically, mixtures with cement and hydrated lime have higher dry ITS than those with baghouse fines and fly ash; while the control mixture was ranked differently between 100-day field curing and laboratory curing. The Spearman's rank correlation coefficient (ρ) equals 0.60 indicating good positive correlation (Royal Geographical Society 2017). The two similar rankings suggest that the dry ITS results for mixtures cured for 3 days at 40°C can be used to rank the relative strengths of long-term cured mixtures (longer than 100 days). Tukey's test results show that the dry ITS of the five mixtures were grouped into only two categories at early ages (3 days and 14 days) after which there was no statistical difference in the dry ITS value for field-cured mixtures. The change in rankings was expected because the strengths of mixes with additives may be reduced in the beginning but would eventually developed as the hydration, lime-asphalt, stiffening, or cementitious reactions continued. Tukey's test grouping results matched the results showed in Figure 5.6 that mixtures tended to have similar strengths after long-term field curing.

Table 5.2 Numerical Ranking and Tukey's Test Grouping Based on Dry ITS after Field Curing

Additive in the Mixture	Numerical Ranking				Tukey's Test Grouping			
	3-Day	14-Day	30-Day	100-Day	3-Day	14-Day	30-Day	100-Day
Cement	3rd	4th	4th	2nd	A B	B	A	A
Hydrated lime	4th	3rd	5th	1st	A B	A B	A	A
Baghouse fines	2nd	2nd	2nd	3rd	A B	A B	A	A
Fly ash	5th	5th	3rd	4th	B	B	A	A
Control	1st	1st	1st	5th	A	A	A	A

However, examining only dry ITS results would miss the potential for damage from exposure to moisture. To compare these mixtures' moisture susceptibilities, additional specimens (three replicates for each mixture) were cured for 120 days before conducting wet ITS tests. Figure 5.7 shows the 120-day wet

ITS of the mixtures compared to the dry ITS results tested after 100 days of field curing. The ratio of 120-days wet ITS to the 100-day dry ITS (hereafter referred to as TSR) is also shown as dots. The cement, hydrated lime, and fly ash mixtures showed slight reductions in wet ITS values (TSR were all greater than 0.87) while more significant reductions were evident in the mixture with baghouse fines (TSR = 0.67) and the control mixture (TSR = 0.72). The three mixtures using cementing additives were those that had the highest tensile strength ratio. Rankings of these mixtures (with 1% additive) by wet ITS were similar to the rankings of laboratory-cured mixtures (ρ = 0.80) (as shown in Table B.2 of Appendices). This suggests that ITS results for mixtures cured for 3 days at 40°C may be used to rank the relative strengths of long-term cured mixtures (100 days).

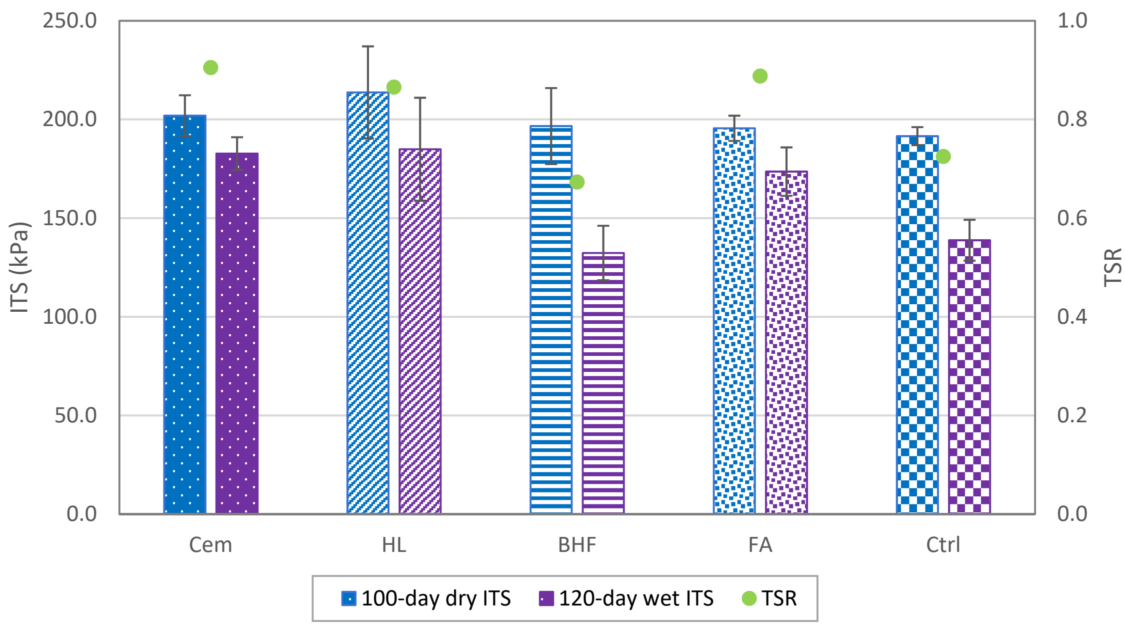


Figure 5.7 Comparison of Dry and Wet ITS after Long-Term Field Curing (Scheme 4)

5.2.2.2 Effect of Top Load and Confinement

Figure 5.8 shows the comparison of dry ITS for specimens with and without top load and confinement. All the mixtures with additives showed a decrease in dry ITS because of the top load and confinement (the ratio of these two dry ITS results were referred to as TSR’’). T-tests showed that the effect of top load and confinement was only significant for the mixture with cement (p-value 0.020). The top loaded and confined specimens would have been shaded more which reduced the temperature and slowed down the development of strength.

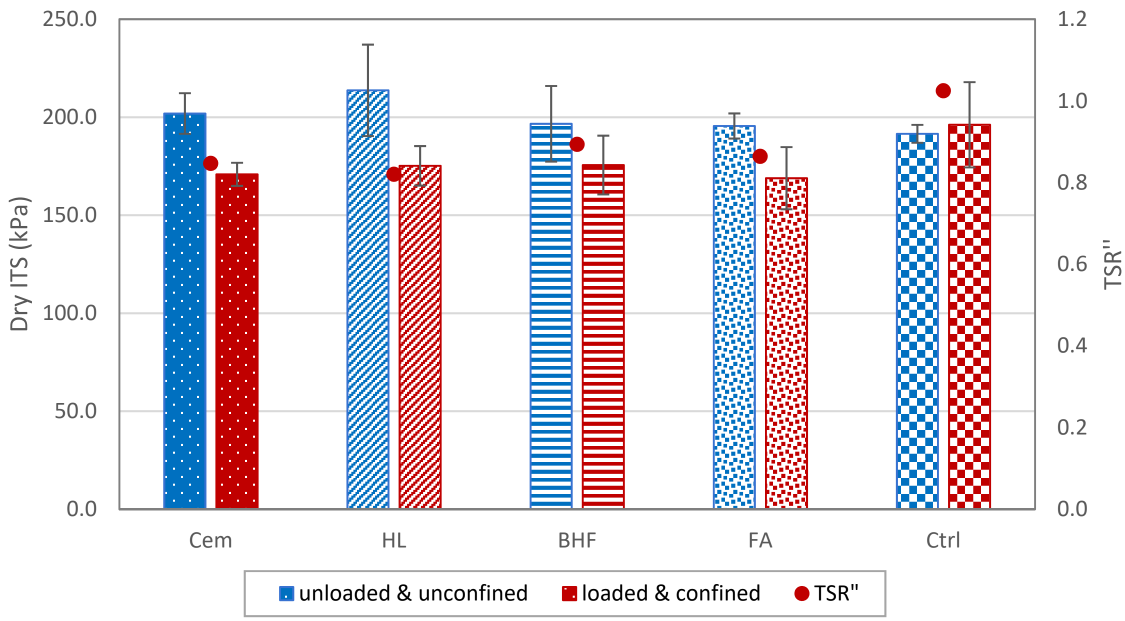


Figure 5.8 Comparison of the Dry ITS for Specimens with and without Top Load and Confinement after 100 Days' Field Curing (Scheme 4 and 5)

5.3 Summary

The study examined the effects of different mineral additives on the indirect tensile strengths of cold recycled foamed asphalt mixtures subjected to laboratory and field curing conditions. Based on the test results presented above, the following findings and conclusions can be made:

- After the commonly used laboratory curing condition (3 days at 40°C), mixtures with cement and baghouse fines had higher dry ITS than mixtures with hydrated lime, fly ash, and no additive. Also, mixtures with cement and hydrated lime had higher wet ITS. Cement was the best overall mineral additive considering improvements to both dry and wet ITS. Cement and hydrated lime were expected to have higher ITS results (dry and wet) than fly ash and baghouse fines. The wet ITS results basically showed this trend but the dry ITS results did not. The unexpected high dry ITS of baghouse fines indicated its effect on strength may not only be due to stiffening. It may contain some limestone fines that can increase ITS.

- Higher additive contents do not necessarily improve dry or wet ITS of mixtures. Only the mixture with cement had increased strength as more cement was added. There may be optimum contents for other additives to be used in cold recycled mixtures to improve ITS.
- Curing moisture had a significant effect on dry ITS of mixtures with cement or baghouse fines. As expected, sufficient moisture helps increase hydration of the cement. However, the current laboratory curing method (3 days at 40°C) may not allow the cement to fully hydrate and could result in a lower strength. Curing moisture significantly reduced the dry ITS of the mixture with baghouse fines. This indicates that using baghouse fines in cold recycled mixture could result in a strength loss if water infiltrates the material in the field.
- Dry ITS of all mixtures with and without mineral additives increased with field curing time. After 30 days of field curing, changes in dry ITS were minor for mixtures with cement, baghouse fines, fly ash, and the control mixture. Results for the hydrated lime did not follow the expected trend with time and probably indicates a materials sampling error with those results. The slower strength gain in the field compared to the laboratory curing was expected because the curing temperature was lower than the lab-curing condition (40°C) and not consistent. This caused specimens to have longer drying times which slowed physical and chemical reactions. The strength of control mixture was unexpectedly higher than those with additives in the first 30-days' field curing. This may be because mineral additives coated RAP surface, initially reducing the bonding of foamed asphalt between RAP particles before all physical and chemical reactions forming the bonding.
- The top load and confinement in field curing were used to simulate the weight of an asphalt overlay on the cold recycled pavement layer. This was expected to make cold recycled mixture denser and increase dry ITS. However, it significantly reduced the dry ITS of mixture with cement and had negligible effect on other mixtures. This may be because the top load and confinement shaded the specimens and reduced the wind effect on the specimens, slowing the drying process and strength development.
- Comparing laboratory and field curing results, test results of mixtures cured for 3 days at 40°C provided similar ranking as the simulated long-term field curing (at least 100 days). Based on the wet ITS results from the simulated long-term field curing, cold recycled foamed asphalt mixtures using

cement, hydrated lime, and fly ash are better for foamed asphalt mixtures than baghouse fines or no additive when adequate wet ITS and/or TSR criteria are required in the mix design.

CHAPTER 6 LABORATORY TESTING STUDY

6.1 Methodology

Laboratory testing is an efficient way to evaluate mixture properties. Three categories of properties, including viscoelastic properties, permanent deformation, and cracking resistance, are evaluated for cold recycled foamed asphalt mixtures. Viscoelastic properties govern the asphalt mixture response under loading. Permanent deformation and cracking tests provide quick assessments of a mixture's resistance to critical distresses. Laboratory test methods are proposed and their repeatability are evaluated. Six cold recycled foamed asphalt mixtures (composed of three RAP materials and two binder types) were tested using different laboratory testing methods. Because only the gradation-related property (i.e., fineness modulus) rather than RAP binder content or F-E particles' percentage was found to strongly correlate with the indirect tensile strength of cold recycled mixes, the RAP gradation was considered as the dominant property to characterize the three RAP materials (R-squared equals 0.71 for fineness modulus, comparing to 0.41 for RAP binder content and 0.02 for F-E particles' percentage). Therefore, the effects of RAP gradation and virgin binder PG on the laboratory testing results can be evaluated based on testing results of the six cold recycled mixes.

6.1.1 Indirect Tensile Strength

The ITS test is used in the mix design procedure for cold recycled foamed asphalt mixtures. Specimens were conditioned at 25°C and tested to failure on a load frame where a diametrical load applied at a rate of 50.8 mm/minute. The peak load was recorded to calculate tensile strength based on Equation 6.1 (ASTM D6931). The six cold recycled mixes from the mix design study were used to evaluate the effects of RAP and virgin binder. Because the wet ITS was less sensitive as the dry ITS to the change of virgin binder PG and RAP gradation, only the dry ITS results are evaluated herein.

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

Where:

ITS = Indirect tensile strength (kPa);

P = Peak load (N);

t = Average height of specimen (mm);

D = Diameter of the specimen (mm).

6.1.2 Dynamic Modulus

Mechanistic-empirical design methods (e.g., Pavement M-E) typically use dynamic modulus as a primary input for asphalt concrete material (Pierce et al. 2014). Dynamic modulus has also been recommended to characterize the viscoelastic properties of cold recycled foamed asphalt mixtures (Kim et al. 2009, Diefenderfer and Link 2014). It allows researchers to capture the response of an asphalt mixture to different temperatures and loading frequencies.

The current procedure for dynamic modulus in an AMPT (AASHTO TP 79-15) requires fabrication of a tall specimen (minimum height of 160 mm) and coring and cutting to obtain a specimen with 100-mm diameter and 150-mm height. However, non-uniformity of density/air void distribution may exist between the upper and lower parts of cold recycled foamed asphalt mixtures (Apeagyei and Diefenderfer 2012, Loizos 2007). The current specimen fabrication procedure removes both ends and the outside part of a tall specimen. After coring and cutting, differences in density between the upper and lower parts may still exist. Instead, small-scale specimens were used to perform the dynamic modulus test in this study. Figure 6.1 shows the small-scale specimen fabricated from a big sample and the test setup in the AMPT. These small specimens have a 50-mm diameter and a 110-mm height. These dimensions were proposed for the convenience of obtaining test specimens from HMA field cores (Bowers et al. 2015). It has been used to evaluate several cold recycled pavement projects across the U.S. (Diefenderfer et al. 2016). In this study, use of small-scale specimens avoided the potential problem of non-uniform air void/density specimens were cored horizontally from SGC-compacted samples. The dimensions of the compacted SGC samples were 150-mm diameter and 115-mm height. The compactive effort applied was 30 gyrations. Statistical analysis (t-test) showed the SGC-compacted samples (150-mm diameter and 115-mm height) had statistically the same density as the mix design samples (100-mm diameter and 63.5-mm height) for two cold recycled foamed asphalt mixtures used in the US Hwy 280 pavement preservation project. The p-values were equal to 0.205 and 0.101 for the CCPR and CIR foam mixes, respectively (Ma et al. 2018).

(a) Specimen fabrication

(b) Test setup in the AMPT

Figure 6.1 Small-Scale Dynamic Modulus Test

Specimens were tested in the AMPT in accordance with AASHTO TP79 with some modifications. Tests were conducted at three temperatures (4, 20, and 40°C) and four frequencies (0.01, 0.1, 1, and 10 Hz). Master curves were built using the sigmoidal function proposed in the Mechanistic-Empirical Pavement Design Guide based on the principle of time-temperature superposition (ARA Inc. 2004). One of the factors in the sigmoidal model is the limiting maximum modulus calculated using the voids in mineral aggregates (VMA) and voids filled with asphalt (VFA) based on the Hirsch model (Christensen 2003) for HMA. For cold recycled mixes, VMA was expected to be higher and the VFA was expected lower than those of typical HMA. The correct calculation for VMA is given by Equation 6.2 (Asphalt Institute, 2015). Although the bulk specific gravity of the compacted mixture (G_{mb}) and percent stone (P_s) can be tested for cold recycled mixtures, the bulk specific gravity of total aggregate (G_{sb}) is unknown for the cold recycled mixes. Therefore, this study estimated VMA, using the effective specific gravity of RAP (G_{se}) instead of $G_{sb.}$ Equation 6.3 shows the formula to calculate G_{se}. Because absorbed asphalt is not counted when calculating G_{se}, the value of G_{se} would be slightly higher than G_{sb}, and therefore the approximate value of VMA may be a little higher, if G_{se} is used instead of G_{sb}. Finally, the approximate VMA can be determined using Equation 6.4. After estimating VMA, the value of VFA can be determined using V_a and VMA.

$$VMA = 100 - \frac{G_{mb} \times P_s}{G_{sb}} \tag{6.2}$$

$$G_{sb} \approx G_{se} = \frac{100 - P_b}{\frac{100}{G_{mm}} - \frac{P_b}{G_b}} \tag{6.3}$$

$$VMA \approx 100 - \frac{G_{mb} \times P_s}{G_{se}} \tag{6.4}$$

Where:

VMA = voids in RAP particles (percent of bulk volume);

G_{mb} = bulk specific gravity of compacted mixture;

G_{sb}= bulk specific gravity of total aggregate;

P_s = RAP content in mixture (percent by weight);

G_{se} = effective specific gravity of RAP;

G_{mm}= theoretical maximum specific gravity of mixture;

P_b= asphalt content in mixture;

G_b= specific gravity of asphalt (assume 1.03).

6.1.3 Resistance to Permanent Deformation

The Flow Number (FN) test, also known as the repeated-load permanent deformation test, was proposed in the National Cooperative Highway Research Program (NCHRP) 9-19 as a performance test to evaluate permanent deformation in HMA (Witczak 2005). Previous studies have also used this method to characterize cold recycled mixtures for rutting susceptibility (Kim et al. 2009, Diefenderfer and Apeagyei 2014).

Small-scale specimens were also used for the Flow Number test in this study. Although no literature was found using small-scale specimen to perform Flow Number test for cold recycled mixes, given the successful application of the small-scale dynamic modulus test to evaluate cold recycled mixes and the fact that small scale specimens have more uniform density, this approach was extended to the

Flow Number test. Figure 6.2 shows the Flow Number test setup and specimen confined by membrane in the AMPT. Specimens used for the dynamic modulus tests were also used for the Flow Number tests.

(a) Top and base plates (b) Test setup in the AMPT

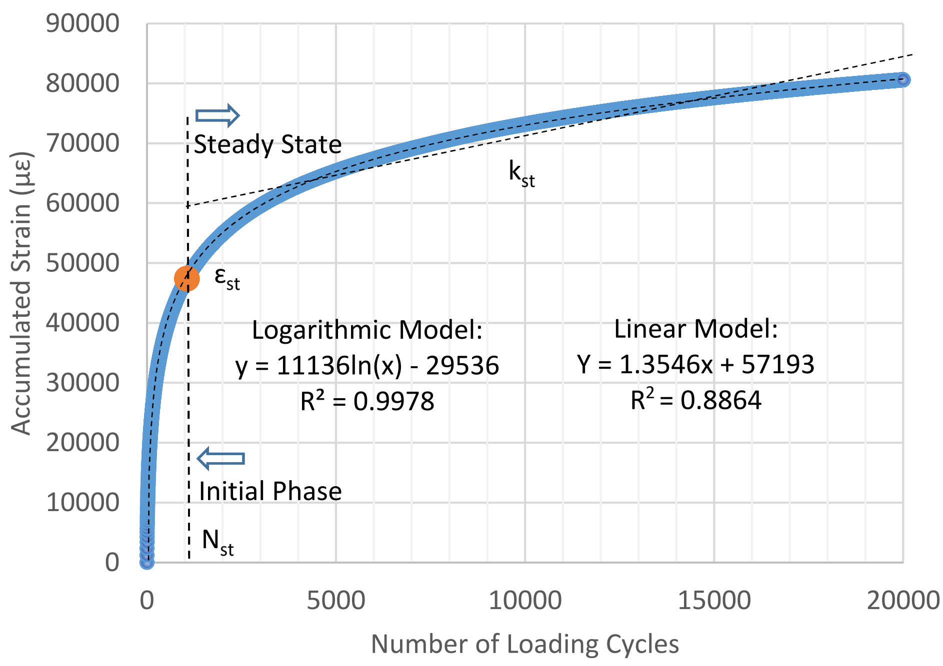
Figure 6.2 Small-Scale Flow Number test

Flow Number testing was conducted with a 483 kPa (70 psi) deviator stress and a 69 kPa confining stress in accordance with the recommendation by Von Quitus (2012). These parameters were found to be within the intermediate range of Flow Number test parameters in an FHWA study based on 15 field projects and 13 laboratory studies (Dongré et al. 2009). Cold recycled foamed asphalt mixtures are typically used in a pavement structure at depth of 100 - 150 mm. The test temperature of 54.5 °C was selected to represent a critical temperature of cold recycled mixtures in the field. This temperature was calculated using the LTPPBind 3.1 at 95-mm (3.75 inches) depth of the Test Track S8 section (at 50% reliability).

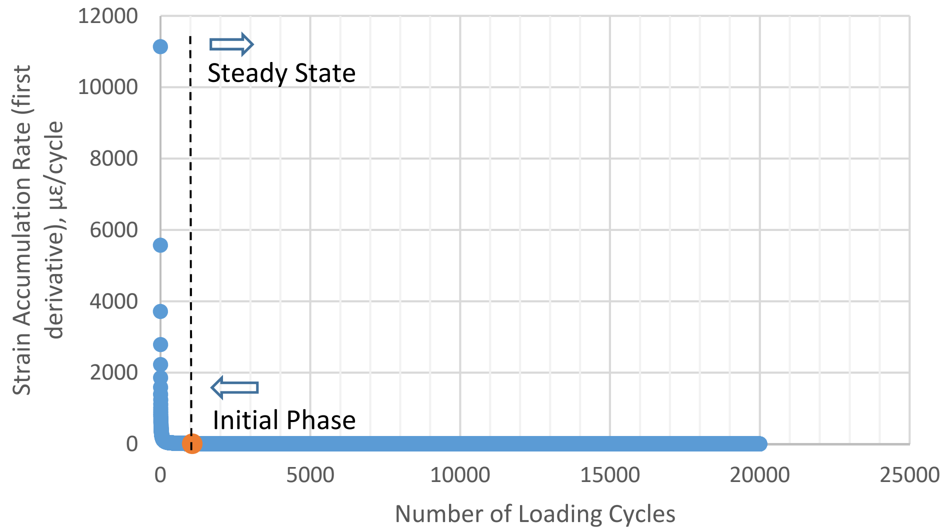
During the FN test, the specimens are subjected to repeated compressive loading until the accumulated strain reaches 100,000 microstrain or the number of loading cycles reaches 20,000 cycles, whichever comes first. Accumulated strain is monitored while during loading, and its development is used for evaluating rutting susceptibility. Ideally, the plot of accumulated strain versus cycles exhibits three phases during testing. The initial phase of deformation represents densification and seating under loading indicated by volume decrease. Permanent strain accumulates fast in this phase. In the second phase, permanent strain develops at a lower and constant rate. In the third phase (typically called tertiary flow), shear failure occurs and accumulated strain rate increases. The Flow Number is defined as the number of loading cycle where tertiary flow begins, indicating the start of shear deformation under constant volume (Witczak 2007). However, recent studies by the University of Maryland (Khosravifar et al. 2015) and NCAT (Ma et al. 2017) reported that cold recycled foamed asphalt mixtures did not exhibit tertiary flow. This lack of tertiary flow may be due to the confinement (West et al. 2013).

Therefore, this study evaluated FN data using two approaches similar to those described in NCHRP report 752 (West et al. 2013). The first approach examined the strain accumulation rate in the steady state phase of strain development. Dongre et al. (2009) defined the steady state in which the second derivative of the fitted regression model (strain vs. cycle) has an absolute value less than 0.025. In this study, the steady state was defined as the second derivative of fitted regression model less than 0.01 (absolute value). Because there was no tertiary flow, this state would continue until the end of the test. The logarithmic model (base e) was used to fit the strain vs. cycle data. The fitting quality was good with R-square values greater than 0.97. Only the model-predict strains in the first 100 cycles did not match the measured strain, however, the model accuracy was not affected since the steady state typically started between 800 and 1300 cycles. Once the number of cycles where the second derivative of the model became less than 0.01 (N_{st}) was determined, all the data after this loading cycle until the end of test was fitted by linear regression model to determine the slope. Ten of the twenty-four specimens in total terminated earlier than 20,000 cycles, including five that reached the maximum strain limit (100,000 microstrain) and five had punctures on the membrane.

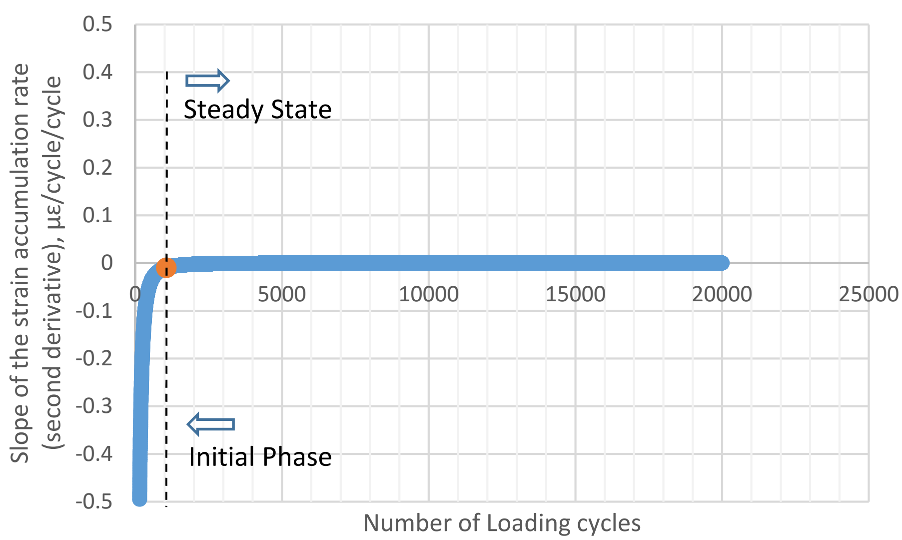
The second FN result obtained from the test data was the accumulated strain at the beginning of the steady state, which was determined at the cycle N_{st}. NCHRP report 752 used the accumulated strain at the end of testing (at 20,000th cycle) to characterize the resistance of HMA to permanent deformation (West et al. 2013). However, in this study, nine tests terminated before 20,000 cycles. Therefore, the accumulated strain at the beginning of the steady state was used instead. Typically, about 40 – 84% of the total permanent strain accumulated in the initial phase of deformation (West et al. 2013). Results from this study showed the initial phase of deformation typically occurred up to 800 - 1,300 loading cycles, where the steady state started and continued to the end of test. Figure 6.3 shows the development of strain, the rate of the strain accumulation (the first derivative of strain vs. cycle regression model), and the slope of the strain accumulation rate (the second derivative of strain vs. cycle regression model) against loading cycles. This data if from a cold recycled mixture specimen with PG 58-34 binder and fine RAP. Figure 6.3(a) shows the development of accumulated strain during the test. A logarithmic regression model was used to fit the data with good quality (R-squared = 0.9978). The end of the initial phase and the beginning of the steady state phase was defined as the cycle N_{st} where the second derivative of the logarithmic model dropped below 0.01 (absolute value). The first result of the FN test is the accumulated strain at N_{st}. A linear regression model was used to fit the data after N_{st}. The slope of this linear model was used as the second result of the FN test. Figure 6.3(b) shows the strain accumulation rate calculated from the first derivative of the logarithmic model. The second derivative of the logarithmic model at each cycle is shown in Figure 6.3(c). The initial data points are negative because the strain accumulation rate is decreasing. The plot plateaus about 1,000 loading cycles and becomes close to zero. This data was typical of all the cold recycled mixes tested in this study.



(a) Accumulated strain



(b) Strain accumulation rate (first derivative of the strain vs. cycle regression model)



(c) Slope of strain accumulation rate (second derivative of the strain vs. cycle regression model)

Figure 6.3 Typical Accumulated Strain, Strain Accumulation Rate, and Slope of the Accumulation Rate Against Loading Cycles in Flow Number test (Mixture with PG 58-34 and Fine RAP)

6.1.4 Resistance to Cracking

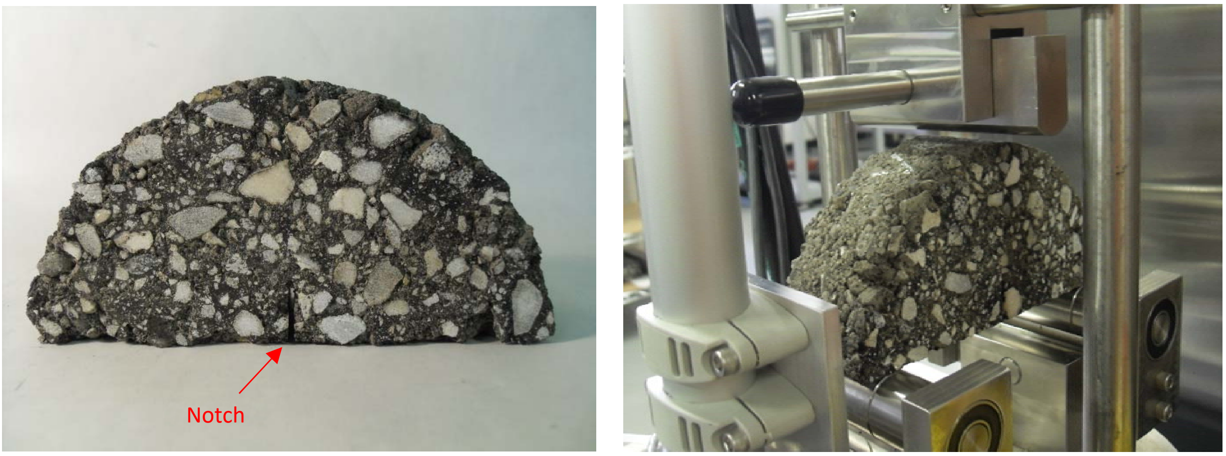
Cracking is another potential concern for pavements using cold recycled foamed asphalt mixtures. No standards have been established for evaluating the cracking resistance of cold recycled mixtures. For this study, two methods were used to assess cracking resistance: one was the I-FIT method; the other was an ITS-based method proposed as the fracture work factor (FWF).

I-FIT Method

The I-FIT test method was developed at the University of Illinois at Urbana-Champaign for characterizing the cracking potential of asphalt mixtures. The Flexibility Index (FI) determined from the test has a coefficient of variation (COV) of 10-20 percent for both laboratory- and plant-produced HMA (Al-Qadi et al. 2015). The FI result not only correlated well with fatigue cracking at FHWA's accelerated loading facility, but also correlated well with field cracking distress in 35 pavement sections in Illinois (Al-Qadi et al. 2015). Based on the correlation between FI values and field performance on FHWA's ALF, mixtures with FI results above 6.0 were considered to have better cracking resistance (Al-Qadi et al. 2015, Ozer et al. 2016). However, I-FIT has not been used to characterize cold recycled foamed asphalt mixtures. The FI threshold 6.0 may not be suitable for cold recycled mixes.

A similar method called SCB FIVE has been proposed to characterize CIR mixtures (Wegman and Sabouri, 2016). Specimens were tested by controlling the rate of crack mouth opening displacement (CMOD). A fracture energy index was used to evaluate cracking resistance. However, repeatability of the SCB FIVE was not reported and correlation with field cracking performance was not investigated.

I-FIT was selected to evaluate the cracking resistance of cold recycled foamed asphalt mixtures in this study because this method has a more convenient setup, shorter testing time, lower variability, and simpler analysis method, compared to other cracking test methods such as DC(T) and S-VECD test. The test procedure follows Illinois test procedure 405, in which a single-notched SCB specimen is loaded at a rate of 50 mm per minute at 25°C. The FI result is determined from the load-displacement curve. Figure 6.4 shows the I-FIT specimen and test setup.



(a) Cold recycled mix I-FIT specimen (b) I-FIT test setup

Figure 6.4 The I-FIT Test for Cold Recycled Foamed Asphalt Mixtures

Fracture Work Factor Method

The second method used to evaluate cracking resistance in this study was based on the ITS test. This method was inspired by the I-FIT method but uses a different specimen geometry and analysis procedure. The ITS Fracture Work Factor (FWF) is based on small ITS specimens (100-mm diameter and 63.5-mm height). FWF was calculated from the area under the load-displacement curve up to the post-peak inflection point. Only specimens tested in the dry condition were analyzed because the wet conditioned specimens were insensitive to changes in component materials. Equation 6.5 shows the formula for determining the FWF based on the ITS load-displacement data.

$$\text{Fracture Work Factor} = \frac{W_F}{k_{inf}} \times C \tag{6.5}$$

Where:

W_F = partial fracture work (up to the inflection point);

k_{inf} = absolute value of the post-peak slope at the inflection point;

C = unit conversion factor (the value equals 1).

A polynomial regression is used to fit the load-displacement relationship. Figure 6.5 shows a typical load-displacement curve from an ITS test. The displacement at the inflection point ($u = u_i$) was determined at the first zero point of the regression model's second derivative. W_F was determined by taking the integral of the regression model from the first non-zero displacement up to the displacement at the inflection point. Slope at the inflection point (k_{inf}) was determined by establishing displacement $u = u_i$ in the first derivative of the polynomial regression model.

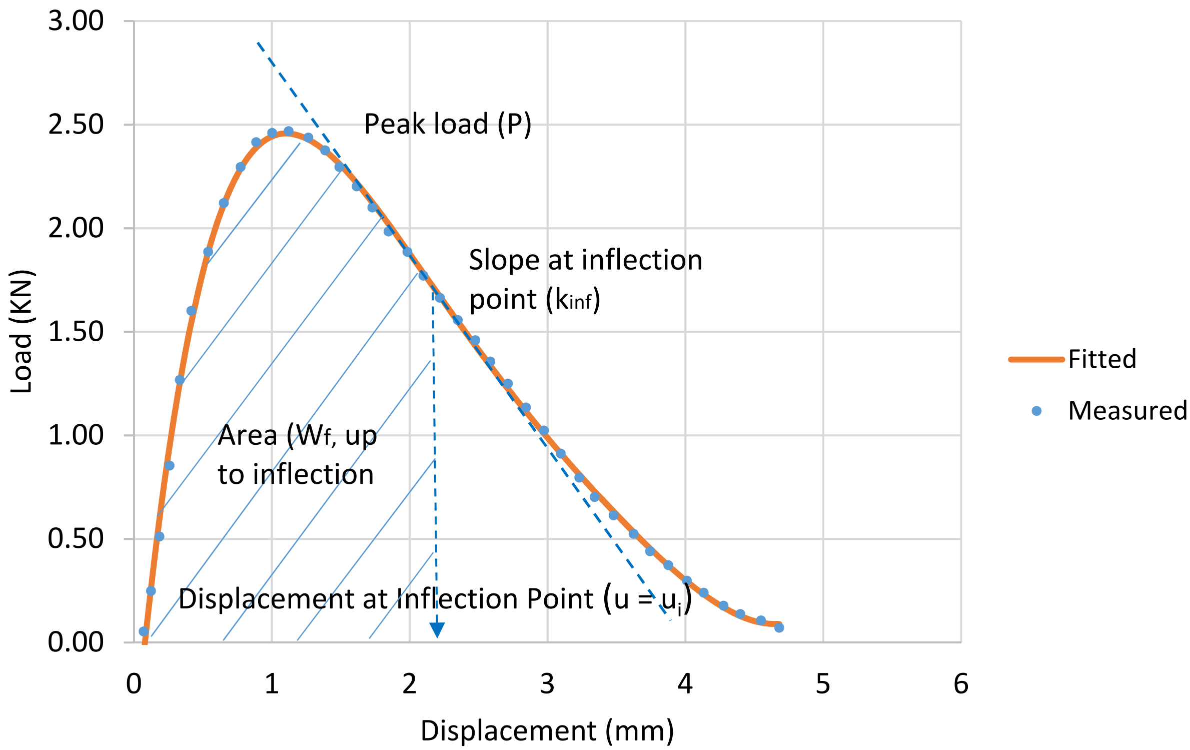


Figure 6.5 Typical Polynomial Regression for the Load-Displacement Curve from the ITS Test

6.1.5 Summary of the Test Methods

Table 6.1 summarizes the objectives and test parameters for the laboratory testing methods used to characterize cold recycled foamed asphalt mixtures.

Table 6.1 Summary of Performance Testing Methods for Cold Recycled Foamed Asphalt Mixtures

Performance Test	Testing Parameters	Test Objectives
Indirect Tensile Strength (ASTM D6931)	• Specimen dimensions: 100-mm Ø and 63.5-mm height • Rate of loading: 50 mm/min. • Test Temperature: 25 °C (Bags containing the dry specimens were put in the 25 °C water-bath for conditioning before testing) • Test result: Indirect Tensile Strength • No. of replicates: 4	• Effects of RAP and virgin binder on the ITS
Dynamic modulus (AASHTO TP79)	• Specimen dimensions: 50-mm Ø and 110-mm height (small-scale) 100-mm Ø and 150-mm height (directly compacted full-scale) • Frequency: 0.01(only used at 40 °C), 0.1, 1, 10 • Test Temperature: 4, 20, 40 °C • Test results: dynamic modulus at combinations of temperature and frequency • No. of replicates: 4	• Investigate effects of RAP and virgin binder on $\|E^*\|$ • Evaluate possibility to use $\|E^*\|$ in pavement M-E design
Flow Number (AASHTO TP79)	• Specimen dimensions: 50-mm Ø and 110-mm height • Deviator stress: 483 kPa (70 psi) Confining stress: 69 kPa (10 psi) Contact stress: 10 kPa (1.45 psi) • Termination strain: 100,000 με • Test temperature: 54.5 °C • Test results: strain accumulation rate (k_{st}) and the accumulated strain at N_{st} (ε_{st}) • No. of replicates: 4	• Investigate effects of RAP and virgin binder on permanent deformation resistance (represented by k_{st} and ε_{st})
Illinois Flexibility Index Test (Illinois Test procedure 405)	• Specimen dimensions: semi-circular150-mm Ø and 50-mm height with a 15-mm notch depth • Loading rate: 50 mm/minute • Test Temperature: 25 °C • Test result: Flexibility Index • No. of replicates: 6	• Effects of RAP and virgin binder on cracking resistance (indicated by FI)
Fracture Work Factor (ASTM D6931 and proposed analysis method)	• Test method: same as ITS test • Test result: Fracture Work Factor • No. of replicates: 4	• Effects of RAP and virgin binder on cracking resistance (indicated by FWF) • Correlation with I-FIT result

6.2 Results and Discussions

The results of six cold recycled mixes from the five laboratory testing methods are summarized in this section. The dry ITS results are discussed first, followed by dynamic modulus results, Flow Number test results, I-FIT results, and the FWF results. All the mixtures tested contained 3% foamed asphalt, except for additional mixes containing 2% foamed asphalt content used for evaluating the FWF method. The optimum total water content (OTWC) was used to facilitate mixing and compacting of specimens. This study only evaluated the effects of RAP and virgin binder on the test results of cold recycled mixes. As discussed before, the effects were mainly caused by different gradations and different virgin binder PG.

6.2.1 Indirect Tensile Strength Test

The dry ITS test results of the cold recycled mixes are summarized in Table 6.2. The average tensile strengths ranged from 227.2 to 296.4 kPa (33 to 43 psi). All the COV results were less than 10%. The highest COV was 9.5% for the mixture with 58-34 binder and medium RAP. The lowest COV is 0.4% for the mixture with 58-34 binder and fine RAP. The bulk densities of these ITS test specimens are summarized in Appendix C. The ITS results were not significantly affected by the densities as evident by the weak correlation (R-squared = 0.068).

Table 6.2 The Dry ITS Results of the Mixtures

Mix	Virgin Binder	RAP	FAC (%)	Dry ITS of Replicates (kPa)				Average (kPa)	Std. Dev. (kPa)	COV (%)
				1	2	3	4			
1	PG 67-22	Fine	2	252.5	251.2	234.9	230.1	242.2	11.4	4.7
			3	305.6	270.3	311.6	298.2	296.4	18.3	6.2
2		Medium	2	215.9	212.2	183.7	202.2	203.5	14.4	7.1
			3	257.4	247.7	260.9	241.5	251.9	8.9	3.5
3		Coarse	2	203.5	184.4	161.0	167.8	179.2	18.9	10.6
			3	235.1	239.4	241.0	200.0	229.0	19.4	8.5
4	PG 58-34	Fine	2	231.1	249.4	239.8	228.0	237.1	9.6	4.1
			3	257.7	257.2	259.7	259.0	258.4	1.2	0.4
5		Medium	2	180.2	166.2	149.9	148.2	161.1	15.1	9.4
			3	243.3	224.3	260.5	209.4	234.4	22.2	9.5
6		Coarse	2	164.6	150.4	151.1	141.0	151.8	9.7	6.4
			3	213.5	238.2	245.8	211.3	227.3	17.4	7.6

Figure 6.6 shows the dry ITS results of the six cold recycled mixes. The mixtures with 58-34 binder tended to have lower dry ITS. The mixtures with fine RAP had higher dry ITS than mixes with medium and coarse RAP. This may be because the foamed asphalt was distributed more uniformly in finer RAP, and therefore, the internal bonding provided by the asphalt increased. Also, mixtures with finer gradations had better compactibility and the asphalt mastic bonding between RAP particles may be stronger because of better compaction. Because the results at two FAC levels did not meet the assumption equal variance for the ANOVA test, the effects of virgin binder and RAP gradation on the dry ITS were evaluated at each level of FAC separately. For the results at 2% FAC, the effects of binder and RAP material were both significant (p-values < 0.001). For the results at 3% FAC, the ANOVA results also showed these effects were significant (p-value 0.006, < 0.001, respectively). Due to the violated assumption of equal variance between the results at two FAC levels, the difference between 2 and 3% FAC was not determined using ANOVA or t-test. As observed in Figure 6.6, mixtures with higher FAC tended to have higher dry ITS results. Based on the limited data, finer RAP gradations and higher PG

binder with adequate foaming properties (ER and HL), should result in cold recycled foamed asphalt mixtures with higher ITS.

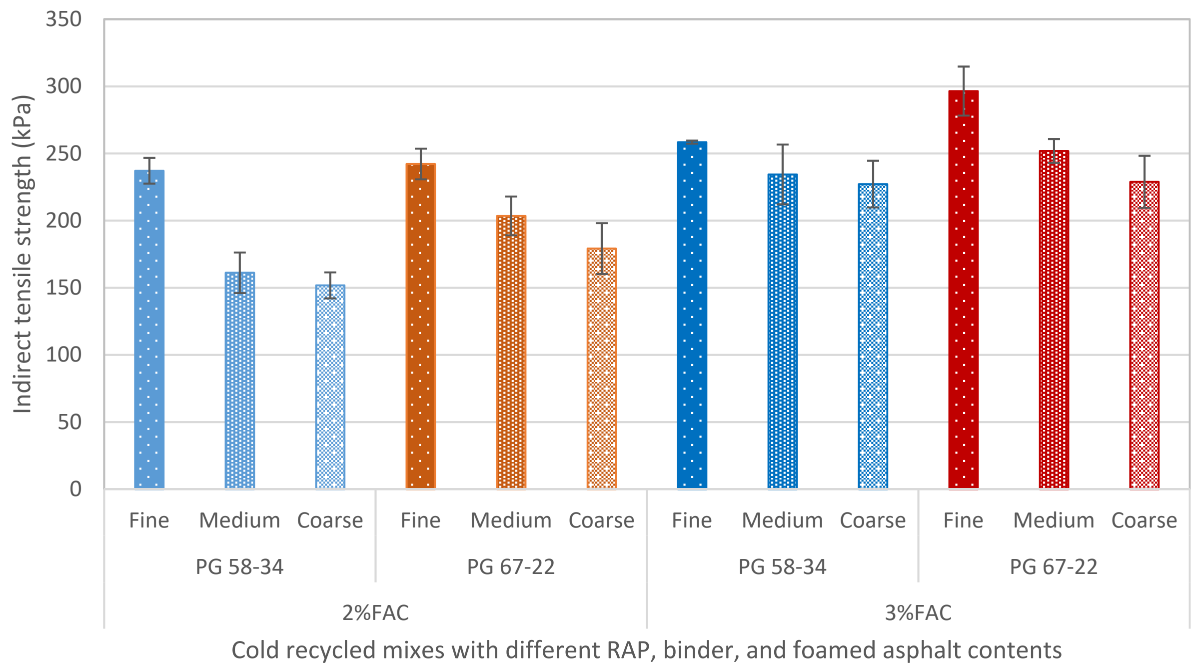


Figure 6.6 The Dry ITS Results of the Cold Recycled Mixtures

6.2.2 Dynamic Modulus Test

The dynamic modulus test was used to determine the viscoelastic properties of cold recycled foamed asphalt mixtures. Small-scale specimens were cored from the Superpave gyratory-compacted samples for all the mixtures except mix 1 because it was very tender and tended to fall apart during coring, despite using several different procedures. Instead, full-scale specimens were prepared and tested for this mixture. Bowers et al. (2015) found this small-scale dynamic modulus test results were about 10 – 20% higher than the full-scale test results for HMA. However, another on-going study at NCAT showed the small-scale test results were only about 5% lower than the full-scale test results for a cold recycled foamed asphalt mixture placed on US Highway 280.

Table 6.3 shows the volumetric properties of the cold recycled mixes, including the bulk density, air void content, and calculated VMA and VFA used in the Hirsch model for determining the dynamic moduli. These mixtures had similar volumetric results except for mix 2 and 3. These two mixes had lower densities and VFAs. Their air void contents and VFAs were higher. This may have been because the virgin binder PG 67-22 was relatively stiff and had lower foaming properties (ER and HL). For the medium and coarse RAP, the foamed asphalt may not have been distributed uniformly in the mixtures. Therefore, these mixes were more difficult to compact compared to the fine RAP mixture. The PG 58-34 virgin binder had lower stiffness and higher foaming properties, which may have facilitated its distribution in the mix and offset the negative effect caused by medium and coarse RAP. Mixtures with stiffer binder (PG 67-22) had higher V_a, VMA, and VFA results than the mixtures with softer binder (PG 58-34). The stiffer binder may not have been distributed as well as the softer binder during mixing. This could have reduced the compactibility of the mixtures with stiffer binder and increased V_a, VMA, and VFA. The densities of the $|E^*|$ specimens were similar to the densities of ITS specimens (used in mix design) with about 5% difference. Unlike the dynamic modulus tests for HMA or WMA, a specific air void content was not targeted when preparing the specimens.

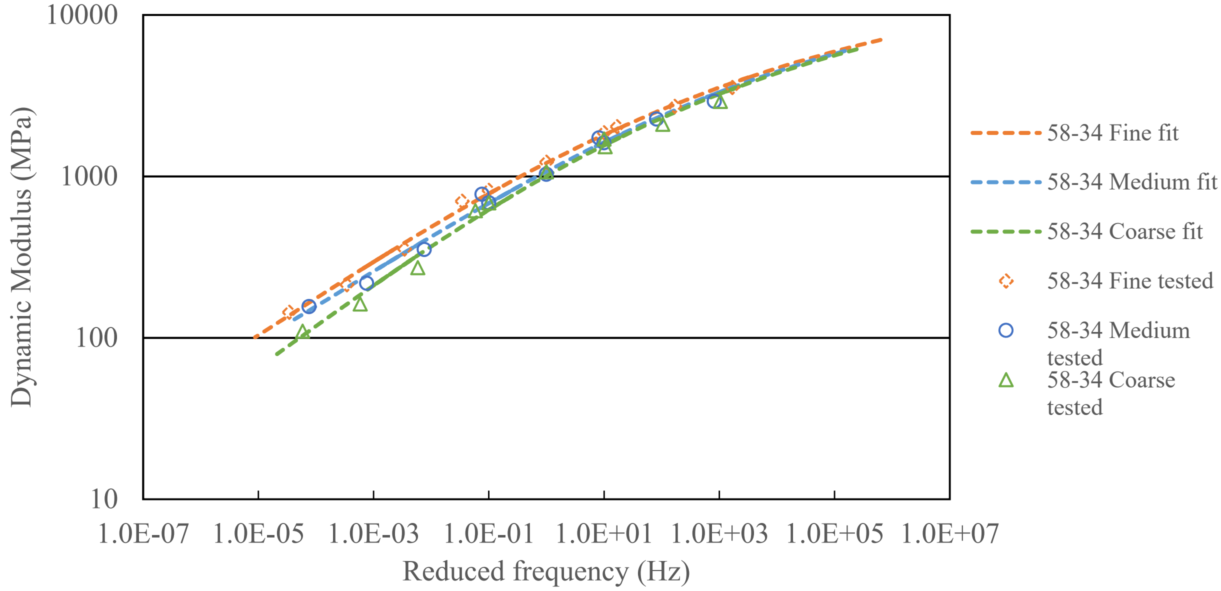
Table 6.3 Average Volumetric Properties of Dynamic Modulus Test Specimens

Mixture	Virgin Binder	RAP	Avg. Bulk Density (kg/m^3)	Avg. V_a (%)	Avg. VMA (%)	Avg. VFA (%)
1	PG 67-22	Fine	2131.3	8.3	24.9	66.6
2		Medium	1920.0	18.5	33.0	43.9
3		Coarse	2109.7	14.2	28.9	50.9
4	PG 58-34	Fine	2146.5	7.9	24.6	67.8
5		Medium	2086.2	9.3	25.4	63.2
6		Coarse	2158.9	9.6	24.6	61.3

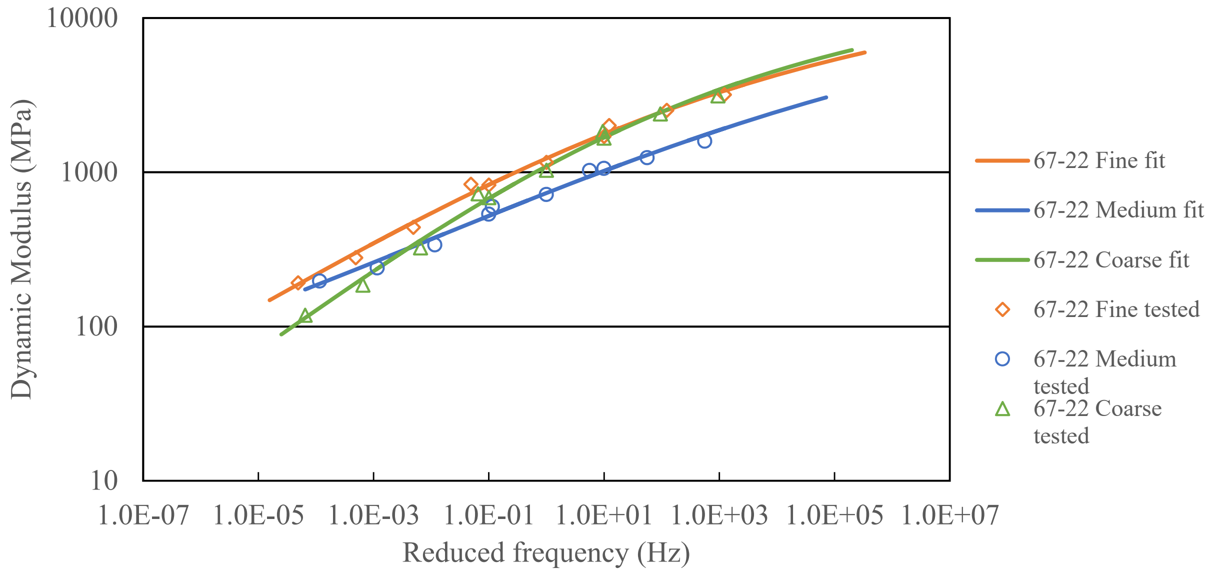
The four quality indicators (load standard error, deformation standard error, deformation uniformity, and phase uniformity) were also obtained from the test. AASHTO TP79 requires the load standard error and the deformation standard error below 10%, the deformation uniformity below 30%, and the phase uniformity less than 3°. The three quality indicators except the phase uniformity were used by Li and Gibson (2015) to compare the data qualities of small- and full-scale dynamic modulus tests for HMA with NMAS no more than 19 mm. The small-scale test had some results with indicators out of the recommended ranges in AASHTO TP79 while the full-scale test only had few. This condition was also found by Diefenderfer et al. (2016) while performing small-scale dynamic modulus tests for cold recycled mixes cored from 24 projects across U.S. and Canada. These recommended ranges were considered not suitable for cold recycled mixes because they were developed based on the HMA test results. In this study, the small-scale test results of the cold recycled mixes had a portion of the quality indicators (about 8 – 25%) out of the recommended ranges. These indicators were used to select three specimens out of four to build master curve for each mixture. The specimen with the most quality indicators out of range was not included. This process helped improving the data quality and decreasing the COV.

Figure 6.7 shows the dynamic modulus master curves of the cold recycled foamed asphalt mixtures. The moduli were plotted against the frequency in a log-log scale. All the mixtures followed the time-temperature superposition principle for linear viscoelastic materials: a master curve at the reference temperature can be built by shifting the moduli from different testing temperatures. Figure 6.7(a) shows the master curves of three mixtures with PG 58-34 binder. The three curves had similar trends while their differences were more evident at lower frequencies. The mixture with fine RAP had the highest modulus, followed by the mixture with medium RAP and coarse RAP. Cold recycled mixtures with finer gradations had better compactibility and lower air voids content. Asphalt mixtures with lower air void content tend to have higher moduli (Steiner et al. 2016). Therefore, the dynamic modulus of the cold recycled mixture with fine RAP was the highest, followed by those with medium and coarse RAP. Figure 6.7(b) shows the master curves for the mixtures with PG 67-22 binder. These master curves were in the same range as the curves showed in Figure 6.7(a) except for the mixture with medium RAP. This mixture had a lower modulus than other mixtures from intermediate to high frequencies. When fabricating the test specimens, this mix tended to fall apart during coring and cutting procedures. Only one (out of twelve small-scale specimens) was available for testing from two batches of mixtures (six big samples can be made from one batch). This specimen was tested to compare with the average modulus of the full-scale specimens so the influence caused by the different scales may be roughly estimated. The linear correlation between the moduli from two specimen scales was strong (R-squared = 0.998). The average modulus of full-scale test was about 72% of the modulus from the small-scale tests. If the tested moduli were plotted on Figure 6.7(b), the master curve of the “67-22 medium” mixture in high frequency range would be slightly higher but the gaps between this master curve and the other two still exist. This indicated the lower moduli of the

"67-22 medium" mixture was only partially due to the difference between the two specimen scales. The high air void content might be the main reason causing the low modulus of this mixture.



(a) Mixtures with PG 58-34 Binder



(b) Mixtures with PG 67-22 Binder

Figure 6.7 Dynamic Modulus Master Curve for the Cold Recycled Mixtures

As shown in Table 6.3, the density of the 67-22 medium mixture was lower than the densities of other mixtures. This was not due to the different testing scales since the densities of the small- and full-scale specimens were similar for the 67-22 medium mixture. A similar trend was observed for the effect of air void content (V_a). Correlations between density and dynamic moduli, and air voids and dynamic moduli in each testing condition (combination of three temperatures and four frequencies) were evaluated and the results are summarized in Table 6.4. The R-squared values were ranged from 0.002 to 0.664. Density and V_a had relative stronger correlations with the moduli at 4°C and 20°C than at 40°C. Density tended to have better correlations than V_a**.** Density was moderately correlated with the moduli (R-squared > 0.6) at two testing conditions (4°C, 10Hz and 20°C, 10Hz) while other correlations at 4 and 20°C were weaker (R-squared ranged from 0.359 to 0.595). Air voids also had better correlations at the two testing conditions (4°C, 10Hz and 20°C, 10Hz) than at other conditions but none of the correlations were as strong as those with density. Since all the mixtures in this study had the same compactive effort (30 gyrations), density was used instead of air void content to explain the trend of dynamic modulus master curves of the mixtures.

Stronger correlations between density and dynamic moduli occurred in the intermediate to high reduced frequency range. In this range, density may be the major cause of the difference between the 67-22 medium RAP master curve and the other two curves. The lower densities of the mixtures with medium RAP may be due to the RAP gradation. The Type-B had less fine particles than the Type-A RAP and Type-C RAP. As seen in Table 3.2, the Type-B RAP only contained 11.2% passing the 1.18-mm sieve and the percent passing on every smaller size (0.6, 0.3, 0.15, and 0.075 mm) was less than the other two RAP materials. Mixtures with insufficient fine RAP particles may have difficulty during compaction and consequently cause low density and strength (Wirtgen GmbH 2012, Asphalt Academy 2009). However, the lower moduli only occurred for the mixture with PG 67-22 binder rather than the mixture with PG 58-34 binder. This difference may be because the PG 58-34 binder had better foaming properties (higher ER and HL) and was distributed more uniformly in the mix. Therefore, a better foaming binder may facilitate the compaction process and offset the negative effect of less fine particles in the RAP. However, this hypothesis needs to be tested using more experiments in the future.

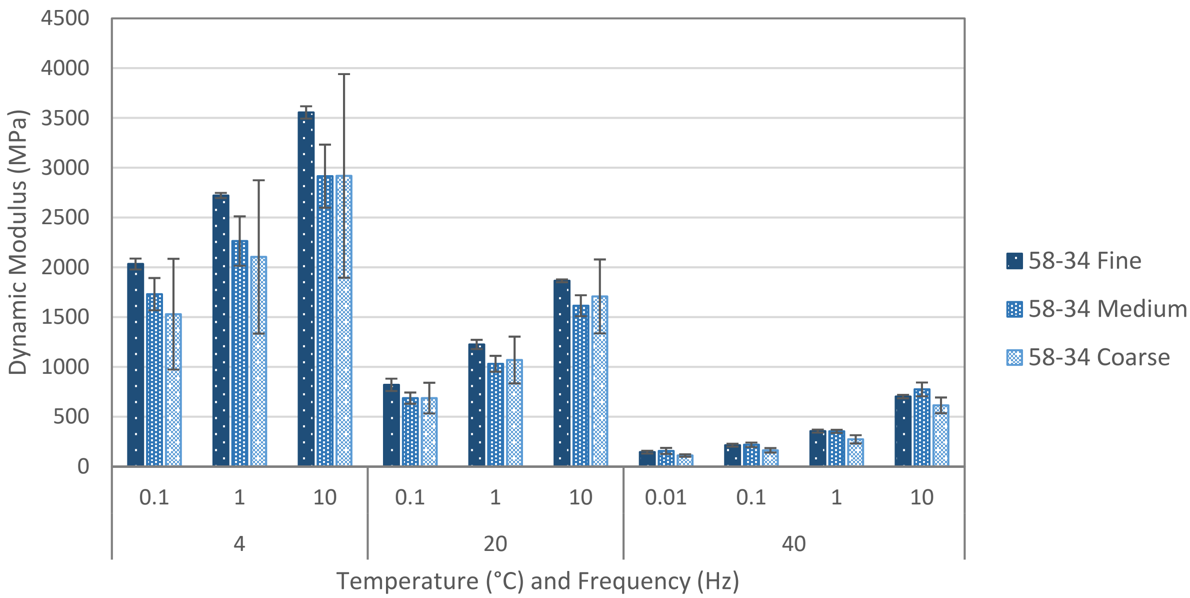
Because the specimen scale appears to affect the dynamic moduli of the cold recycled mixes, the statistical analysis based on the test results may also be influenced. The effect of specimen scale also had to be considered in the discussion of the statistical results in the following paragraphs. However, the effect of density was not evaluated in the statistical analysis because it was a consequence of the materials and would be reflected in the effects of RAP, binder, and their interactions.

Table 6.4 Correlation between Dynamic Modulus and Density (or Air Voids Content)

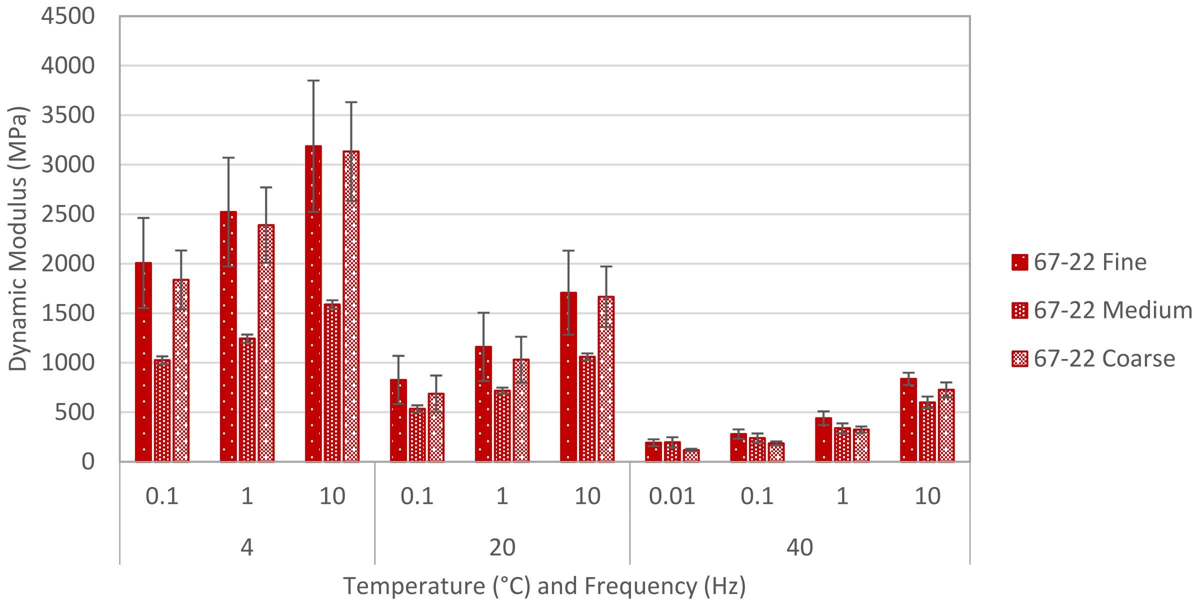
Volumetric Property	R-Squared of Linear Correlations									
	4°C			20°C			40°C			
	0.1 Hz	1 Hz	10 Hz	0.1 Hz	1 Hz	10 Hz	0.01 Hz	0.1 Hz	1 Hz	10 Hz
Density	0.508	0.595	0.664	0.359	0.523	0.662	0.230	0.055	0.002	0.170
Air Void Content (V_a)	0.471	0.529	0.552	0.405	0.514	0.554	0.037	0.001	0.067	0.256

The master curves can be used to compare the cold recycled mixes for their sensitivities to frequency but their results at specific testing condition cannot be compared. Instead, Figure 6.8 plots the average dynamic moduli in clustered columns at three temperatures and four frequencies. Differences between the mixtures with same binder but different RAP) at can be observed. Moduli of the mixture with coarse RAP had higher variability than other mixtures. Results at lower temperatures had more variability than those at higher temperatures. The ANOVA was conducted to evaluate the effects of RAP (gradation) and binder (PG). The results summarized in Table 6.5 can verify the observations on the potentially significant effects shown in Figure 6.8. Figure 6.8 shows that mixtures with different RAP have evident differences in most testing conditions. These differences were verified by the ANOVA results because the effect of RAP was found significant in 7 out of 10 conditions (except for the conditions at 20°C). The mixtures with two binders seemed to have similar ranges of dynamic moduli. This observation was also verified by the ANOVA since the binder effect was not statistically significant in most testing conditions.

The significant effect of RAP on the moduli was examined in the mixtures with PG 58-34 binder and the mixtures with PG 67-22 binder separately. Figure 6.8(a) shows that the significant difference among the mixtures with different RAP may be because the mixture with fine RAP were much stiffer than other mixtures at low and intermediate temperatures (4 and 20°C). However, when PG 67-22 binder was used, the mixture with fine and coarse RAP had similar moduli and both were much higher than moduli of the mixture with medium RAP, as shown in Figure 6.8(b). Tukey's test was used to compare the moduli of mixtures with different RAP. Mixtures with two binders were evaluated separately. The grouping results are summarized in Table 6.6 and 6.7. Table 6.6 shows the three mixtures with PG 58-34 binder were categorized in one group in most of the testing conditions indicating their mean dynamic moduli were statistically the same. At high testing temperature (40°C), mixtures with coarse RAP were categorized in another group because its mean dynamic modulus was lower. This corresponded with the observation in Figure 6.7(a) where two master curves had bigger gap at lower reduced frequencies. Table 6.7 shows the mixtures with medium RAP were categorized in a separate group because its low dynamic moduli in three low-temperature conditions and one high-temperature condition. Since the low-temperature conditions corresponded to the highest reduced frequencies, this Tukey's results reflected the gaps between medium RAP mixture and other mixtures at the right end of the master curves in Figure 6.7(b).



(a) Mixtures with PG 58-34 Binder



(b) Mixtures with PG 67-22 Binder

Figure 6.8 Summary of Dynamic Modulus Results for the Cold Recycled Mixtures

Table 6.5 ANOVA Results of the Effects on Dynamic Modulus at Different Conditions

Effects	P-value of the Effects									
	4°C			20°C			40°C			
	0.1 Hz	1 Hz	10 Hz	0.1 Hz	1 Hz	10 Hz[1]	0.01 Hz	0.1 Hz	1 Hz	10 Hz
RAP	0.017	0.014	0.013	0.071	0.051	N/A	0.007	0.005	0.005	0.042
Binder	0.374	0.149	0.083	0.485	0.163	N/A	0.039	0.029	0.062	0.439
Interaction	0.054	0.061	0.090	0.579	0.441	N/A	0.489	0.422	0.154	0.002

Note: 1. Assumption of equal variances were not satisfied. ANOVA was not proper to use.

Table 6.6 Tukey's Test Results for the Mixtures with Virgin Binder PG 58-34

Binder	RAP	Grouping Results									
		4°C			20°C			40°C			
		0.1 Hz	1 Hz	10 Hz	0.1 Hz	1 Hz	10 Hz	0.01 Hz	0.1 Hz	1 Hz	10 Hz
PG 58-34	Fine	A	A	A	A	A	A	A	A	A	A B
	Medium	A	A	A	A	A	A	A	A	A	A
	Coarse	A	A	A	A	A	A	A	B	B	B

Table 6.7 Tukey's Test Results for the Mixtures with Virgin Binder PG 67-22

Binder	RAP	Grouping Results									
		4°C			20°C			40°C			
		0.1 Hz	1 Hz	10 Hz	0.1 Hz	1 Hz	10 Hz	0.01 Hz	0.1 Hz	1 Hz	10 Hz
PG 67-22	Fine	A	A	A	A	A	A	A	A	A	A
	Medium	B	B	B	A	A	A	A	A	A	B
	Coarse	A	A	A	A	A	A	A	A	A	A B

The results from dynamic modulus test suggested that the RAP gradation had more influences on the dynamic modulus than the virgin binder PG. The effects of RAP gradation were statistically significant in most of the testing conditions and the effects of virgin binder PG were only significant in few testing conditions. The mixtures with fine RAP had higher dynamic moduli than those with medium and coarse RAP. Difference of modulus were more evident for the mixtures with the virgin binder PG 58-34 at high temperature and the mixtures with the virgin binder PG 67-22 at low temperature. This study found mixtures with higher density tended to have higher moduli. The influence of RAP gradation may be because gradation governs the compactibility of mixture. Finer gradations compacted to higher densities than coarser gradations. Virgin binder with a lower PG had better foaming properties which may have facilitated compaction but this effect was not as significant as the effect of RAP gradation.

Even though the master curves could be built based on the test results, the small-scale dynamic modulus test may not be ready to be implemented in the pavement design to characterize the cold recycled foamed asphalt mixtures. The issue in preparing specimens for low-density mixtures (e.g., mix with PG 67-22 binder and medium RAP) needs to be addressed. The acceptable limits of data quality indicators need to be established for the cold recycled mixes.

6.2.3 Flow Number Test

The two results from FN tests were summarized and discussed in this section. These results were used to evaluate the resistance to permanent deformation for the cold recycled foamed asphalt mixtures. The effects of RAP gradations and virgin binder PG were evaluated.

The strain accumulation rates in the steady state (k_{st}), are summarized in Table 6.8. This parameter indicates the rate of strain development during repeated loading. Smaller rates indicate better resistance to permanent deformation. Average accumulation rates ranged from 1.1 to 6.3 microstrain per cycle. Mixtures with coarse RAP had higher accumulation rates than the mixtures with medium or fine RAP. The COV results were based on all the test results without filtering because no replicate was found to be an outlier. The COV results were also higher for the coarse RAP mixtures. This may be because the distribution of foamed asphalt in coarse RAP was not as uniform as those in fine RAP. Figure 6.9 shows the k_{st} for the six mixtures. Regardless of virgin binder PG (PG 58-34 or PG 67-22), the coarse RAP mixtures had the worst resistance to permanent deformation. Mixtures with fine RAP were slightly better than the mixtures with medium RAP. However, initial evaluation for these results showed that both the normality and equal variance assumptions for statistical analysis (ANOVA or Tukey's test) were violated. Therefore, the statistical analysis methods were not applied to evaluate the effects of RAP and virgin binder because the obtained results would not be valid.

Table 6.8 Summary of Strain Accumulation Rate in the Steady State

Mix	Virgin Binder	RAP	Strain Accumulation Rate in the Steady State (k_{st})						
			1	2	3	4	Avg. (με/cycle)	Std. Dev. (με/cycle)	COV (%)
1	PG 67-22	Fine	0.9	0.9	1.3	1.2	1.1	0.2	16.7
2		Medium	2.3	2.3	2.3	3.2	2.5	0.4	16.5
3		Coarse	6.7	0.9	11.0	N/A[1]	6.2	5.1	82.2
4	PG 58-34	Fine	1.7	1.4	1.7	1.9	1.7	0.2	13.6
5		Medium	1.5	4.7	3.1	0.5	2.4	1.9	76.4
6		Coarse	1.6	8.9	8.5	N/A[1]	6.3	4.1	64.7

Note: 1. Test stopped before the steady state of strain development.

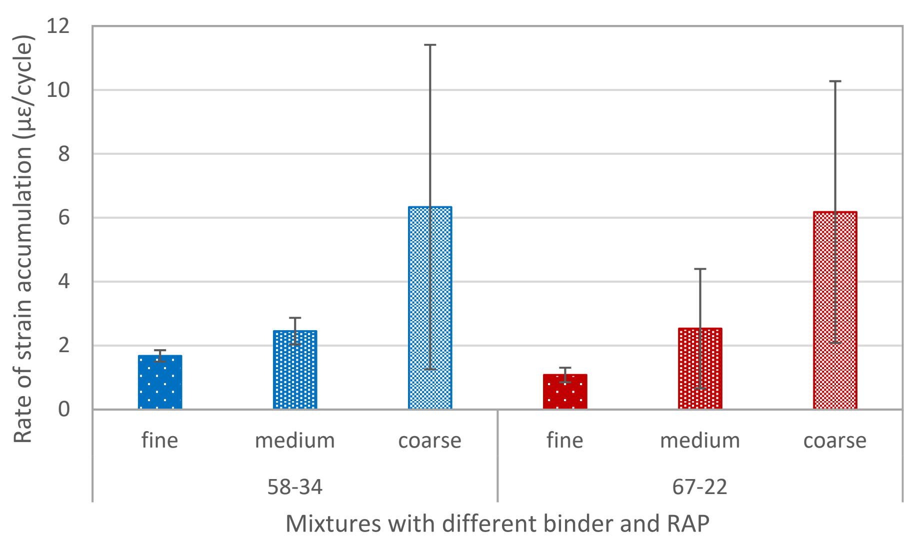


Figure 6.9 Strain Accumulation Rate in the Steady State

Another result from the FN test was the accumulated strain at the beginning of the steady state (ε_{st}). Table 6.9 summarizes the average ε_{st} results for the mixtures. The measured strains were between 21,548 and 70,373 microstrain with COV ranging from 13.6 to 83.4%. The average COV was 31.8%, which was lower than the average COV of the strain accumulation rate in the steady state (k_{st}) (45.0%).

Figure 6.10 shows the average accumulated strain for the cold recycled mixes. Mixtures with PG 67-22 binder had lower strains than the mixtures with PG 58-34 binder except for the mixes with medium RAP. The coarse RAP mixtures had higher strain than mixtures with fine RAP regardless of binder used. The mixture with medium RAP had better resistance to permanent deformation than the fine or coarse RAP mixtures when PG 58-34 binder was used, but it was the worst of the three mixes when binder PG 67-22 was used. The statistical analysis for the effect of binder and RAP was not performed to validate the observation due to the violation of the normality and equal variance assumptions for ANOVA or Tukey's test.

Table 6.9 Summary of the Accumulated Strain at the Beginning of the Steady State

Mix	Virgin Binder	RAP	Accumulated Strain at the Beginning of the Steady State (ε_{st})						
			1	2	3	4	Avg. (με)	Std. Dev. (με)	COV
1	PG 67-22	Fine	18,925	17,360	24,931	24,977	21,548	3,984	18.5
2		Medium	34,108	47,249	43,087	45,145	42,397	5,781	13.6
3		Coarse	23,639	27,503	37,792	N/A[1]	29,64,5	7,316	24.7
4	PG 58-34	Fine	47,170	47,387	45,382	63,077	50,754	8,264	16.3
5		Medium	56,251	14,250	15,066	14,423	24,998	20,839	83.4
6		Coarse	42,514	84,202	84,403	N/A[1]	70,373	24,127	34.3

Note: 1. Test stopped before the steady state of strain development.

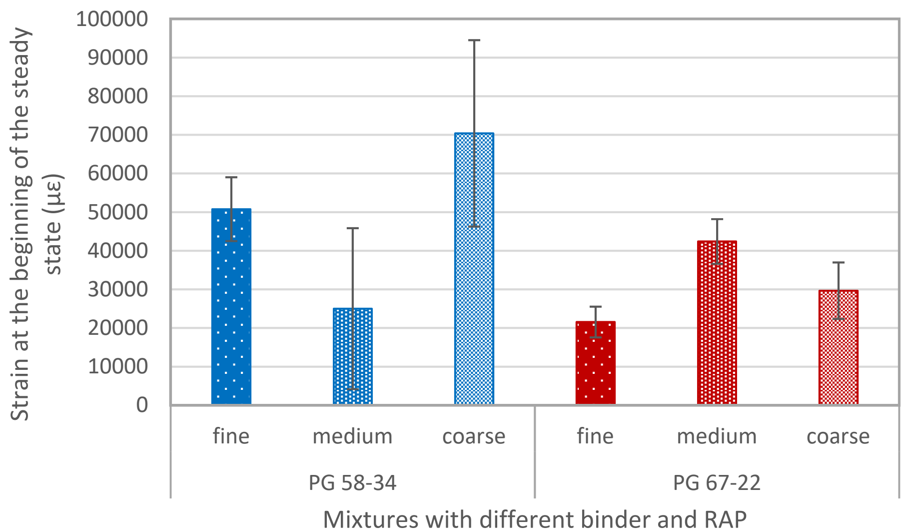


Figure 6.10 Accumulated Strain at the Beginning of the Steady State

Both FN results showed that mixtures with fine RAP had better resistance to permanent deformation than the mixtures with coarse RAP. The effect of binder on the mixtures was not evident based on k_{st}, but seemed to have impact ε_{st} results except for the mixtures with medium RAP. Since ε_{st} had lower variability than k_{st}, it may be better at characterizing cold recycled mixes, especially for the mixtures with coarse RAP. Neither k_{st} nor ε_{st} had strong correlation with density (R-squared < 0.01). The findings for Flow Number test results were consistent with those of Kim et al. (2009). According to their findings, cold recycled mixtures with finer RAP had higher Flow Number results than those with coarse RAP. Although finer RAP had stiffer residual asphalt than the coarse RAP, residual asphalt was assumed to have much less effect than gradation when mixing at room temperature and tested at 40°C.

Mixtures with finer RAP may have better foamed asphalt distribution than for coarse RAP, and therefore, finer mixtures had better integrity than coarser mixtures. During the test, more bonded RAP particles on the cross-section of specimen provide resistance to the applied compressive load. Because same load was applied over the same cross-sections for fine and coarse specimens, number of contact points may be greater for finer graded mixtures. This may cause both Flow Number test results (the strain accumulation rate and accumulated strain at the beginning of steady state) to be lower in finer mixtures.

6.2.4 SCB I-FIT Method

After specimens were prepared, their bulk densities were measured following AASHTO T331. The results of densities are summarized in Table C.8 in Appendix C. The average densities for different mixtures were similar and the COV results was very low. Three parameters obtained from the test were tensile strength, fracture energy, and post-peak slope, as shown Table 6.10. Mixtures with the stiffer binder (PG 67-22) had higher strengths, fracture energies, and post-peak slopes (absolute value). This indicated the area under the load-displacement curve was larger and the specimen failed more quickly after peak load. Also, mixtures with medium RAP had lower tensile strengths and fracture energies than the fine or coarse RAP mixtures. In addition, variability in tensile strengths were smaller than that of fracture energy or post-peak slope. Table 6.10 shows that of the SCB cracking parameter ranked the mixtures differently. Peak tensile strength alone does not capture the full load development. Fracture energy, the area under the load-displacement curve, captured the entire load-displacement relationship. However, a stiffer mixture tends to break earlier (at a smaller displacement) and its curve would be taller and narrower, even though a stiffer and a softer have the same fracture energy. In this case, the post-peak slope also needs to be determined to distinguish the two mixtures. Two mixtures may have the same fracture energy, but one with a higher post-peak slope indicates the mixture is more brittle.

Table 6.10 Summary of Parameters from I-FIT

Mix	Virgin Binder	RAP	Tensile Strength		Fracture Energy		Post-Peak Slope	
			Avg. (kPa)	COV (%)	Avg. (J/m^2)	COV (%)	Avg. (KN/mm)	COV (%)
1	PG 67-22	Fine	126.8	14.6	414.2	25.9	-0.8	39.7
2		Medium	105.2	14.3	354.9	29.1	-1.0	14.2
3		Coarse	124.8	19.8	510.6	26.5	-0.6	44.0
4	PG 58-34	Fine	106.4	15.7	268.9	33.5	-0.7	32.3
5		Medium	99.1	13.3	251.0	28.9	-0.8	24.5
6		Coarse	104.6	10.4	327.0	7.1	-0.6	23.8

The FI results of the cold recycled foamed asphalt mixtures are summarized in Table 6.11. The average FI results ranged from 3.6 to 8.4. Only the coarse RAP mixture with PG 67-22 binder had an FI greater than 6.0. Illinois researchers have suggested asphalt mixtures with FI above 6.0 would have good cracking resistance. However, this preliminary threshold was established for HMA and WMA mixtures, not cold recycled mixtures (Al-Qadi et al. 2015, Ozer et al. 2016). In this study, six replicates were tested and one outlier was removed as recommended in Illinois Test Procedure 405. The result farthest from the average of six replicates was considered an outlier. A previous study conducted by Al-Qadi et al. (2015) found the COV of the FI results from eleven HMA materials ranged from 4 to 20% with an average of 12%. However, the trimmed results for cold recycled mixes had a much higher variability (average COV = 33.5%). Most variability in the cold recycled mix FI results may be due to greater variability in the fabrication of specimens.

Table 6.11 Summary of Flexibility Index from I-FIT

Mix	Virgin Binder	RAP	Flexibility Index					Average	Std. Dev.	COV (%)
1	PG 67-22	Fine	4.7	6.2	8.0	2.4	7.6	5.8	2.3	39.7
2		Medium	2.9	3.7	5.2	5.3	2.0	3.8	1.4	37.9
3		Coarse	10.6	9.3	7.3	9.5	5.4	8.4	2.0	24.1
4	PG 58-34	Fine	3.5	2.0	5.4	4.7	5.7	4.3	1.5	35.4
5		Medium	3.0	5.4	2.6	5.2	2.1	3.6	1.6	44.4
6		Coarse	6.6	3.8	5.2	5.1	5.8	5.3	1.0	19.6

Figure 6.11 shows the FI of the six mixtures. Mixtures with coarse RAP had higher FI values than those with medium and fine RAP. Mixtures with PG 67-22 binder had higher FI results than the mixtures with PG 58-34 binder, even though the difference between the two mixtures with medium RAP was not significant. The results indicate that the higher PG binder and coarse RAP may provide the best cracking resistance of the cold recycled foamed asphalt mixtures.

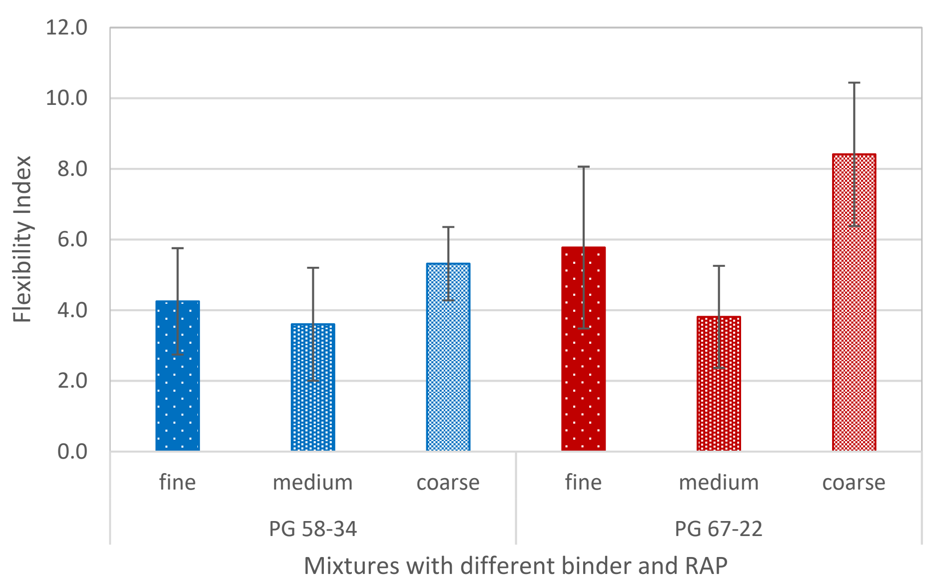
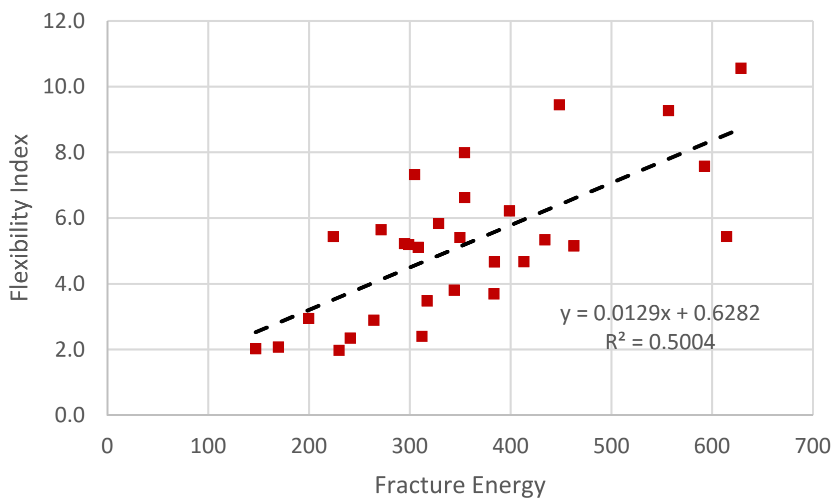


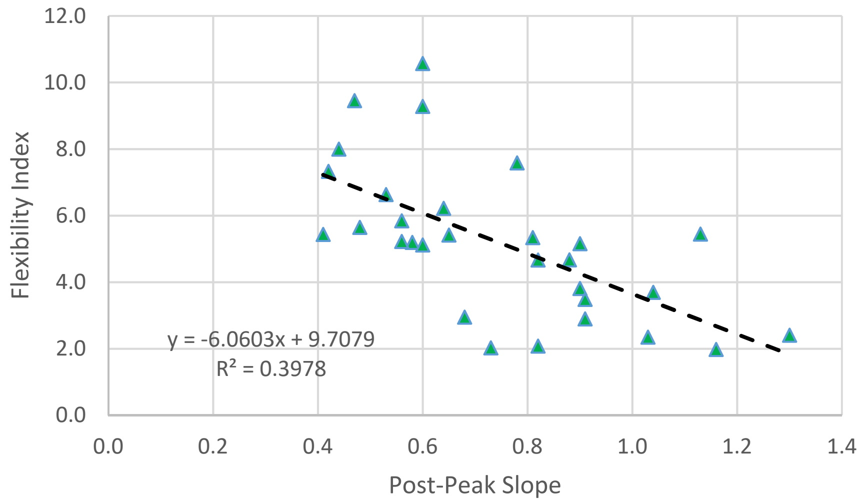
Figure 6.11 Comparison of the FI Results of Six Cold Recycled Mixtures

Effects of binder and RAP were also evaluated by the ANOVA. The results showed RAP had significant effect on FI (p-value = 0.004), which corresponded with the observation in Figure 6.11. The effect of binder was not significant based on the ANOVA (p-value = 0.07). This may be because the high variability reduced the power of F-test in ANOVA. Therefore, more test replicates of cold recycled mixes may be appropriate for the I-FIT test.

Correlation between the FI result and density was weak (R-squared = 0.106) for these cold recycled mixes, although the FI result was found sensitive to the change of density for HMA materials by Al-Qadi et al. (2015). The FI results were also correlated with the fracture energy and the post-peak slope. As shown in Figure 6.12, the correlation between FI and the fracture energy was relative strong and that between FI and the post-peak slope (absolute value) was weaker.



(a) Correlation between the FI result and the Fracture Energy



(b) Correlation between the FI result and the Post-Peak Slope

Figure 6.12 Correlation between the FI Result and the Parameters from I-FIT

The FI result can distinguish the mixtures with different RAP gradations regarding the cracking resistance. This method may also be able to characterize the mixtures with different binder if variability was reduced.

6.2.5 Fracture Work Factor Method

Cracking resistance of the cold recycled foamed asphalt mixtures was also evaluated based a new analysis method for ITS data. This test was simple to conduct and specimens are easy to prepare. The ITS results for the six cold recycled mixes were analyzed to determine the index defined as the Fracture Work Factor (FWF).

Table 6.12 summarizes the intermediate parameters used to calculate FWF. While the peak load was not used in calculation, it had influence on the load-displacement relationship. Therefore, it may affect the FWF result indirectly. The partial fracture work is equal to the area under the load-displacement curve from the starting point (first non-zero displacement) to the inflection point. The area has units of "KN × mm", which is equivalent to the unit of Joules.

Table 6.12 Summary of the Parameters during Result Analysis

Mix	Virgin Binder	RAP	FAC (%)	Peak Load		Partial Fracture Work		Post-Peak Slope	
				Avg. (KN)	COV (%)	Avg. (Joules)	COV (%)	Avg. (KN/mm)	COV (%)
1	PG 67-22	Fine	2	2.5	4.6	4.0	18.2	-0.8	8.5
			3	3.0	6.1	3.5	27.2	-1.4	37.4
2		Medium	2	2.1	7.0	3.3	36.8	-0.8	13.9
			3	2.5	9.6	4.1	18.1	-0.7	15.1
3		Coarse	2	1.8	9.9	3.5	24.1	-0.5	15.9
			3	2.3	7.8	4.8	26.0	-0.6	12.3
4	PG 58-34	Fine	2	2.5	4.1	4.4	6.7	-1.0	8.6
			3	2.6	0.9	5.9	6.6	-0.8	13.1
5		Medium	2	1.7	9.3	2.7	15.9	-0.6	18.2
			3	2.4	9.4	4.0	14.3	-0.9	21.1
6		Coarse	2	1.5	6.1	2.8	8.3	-0.5	17.0
			3	2.3	7.1	4.7	19.3	-0.6	10.6

The FWF for the cold recycled mixtures with 2.0 and 3.0% foamed asphalt contents are summarized in Table 6.13. The average FWF ranged from 3.1 to 8.1. Although the unit of FWF is squared millimeter (mm^2), the result does not represent an actual area on the curve or the testing specimen. Therefore, the result was only considered an index to represent a relative cracking resistance of the tested specimen. The COV results were between 7.7% and 67.7%, with the average of 23.3%. Most mixtures had COV result below 30%. Densities of the test specimens are summarized in Appendix C. The variation of density may not be the major source of variability in FWF results because the COV result of density was very small (0.1 to 1.1%). Correlation between the FWF result and density was also found weak (R-squared = 0.100). The two mixtures with higher variability may be due to greater variability of materials.

Table 6.13 Summary of the Fracture Work Factor Results

Mix	Virgin Binder	RAP Gradation	FAC (%)	Fracture Work Factor				Average	Std. Dev.	COV (%)
1	PG 67-22	Fine	2	6.4	4.3	4.3	4.1	4.8	1.1	22.5
			3	1.3	2.1	2.9	6.0	3.1	2.1	67.1
2		Medium	2	3.1	3.7	3.7	6.9	4.4	1.7	39.6
			3	6.1	4.9	6.1	5.0	5.5	0.7	12.4
3		Coarse	2	8.0	6.9	6.9	6.9	7.2	0.6	7.7
			3	10.0	7.4	7.7	7.1	8.1	1.3	16.5
4	PG 58-34	Fine	2	5.2	4.5	3.9	4.2	4.5	0.5	12.0
			3	6.4	8.3	8.2	5.6	7.1	1.3	18.7
5		Medium	2	6.6	4.6	3.6	5.1	5.0	1.3	25.8
			3	4.4	5.4	3.9	4.5	4.6	0.6	13.1
6		Coarse	2	4.2	6.0	6.4	4.3	5.2	1.1	21.7
			3	8.5	10.4	7.3	6.0	8.1	1.9	23.2

Figure 6.13 shows the comparison for the FWF results of the cold recycled mixes. Mixtures with coarse RAP had higher FWF results than the mixtures with fine and medium RAP gradations. Mixtures with fine and medium RAP had similar results at 2% FAC but the results were much more different at 3% FAC. The effect of binder grade on the FWF results was not significant. in most cases, higher foamed asphalt contents resulted in higher FWF.

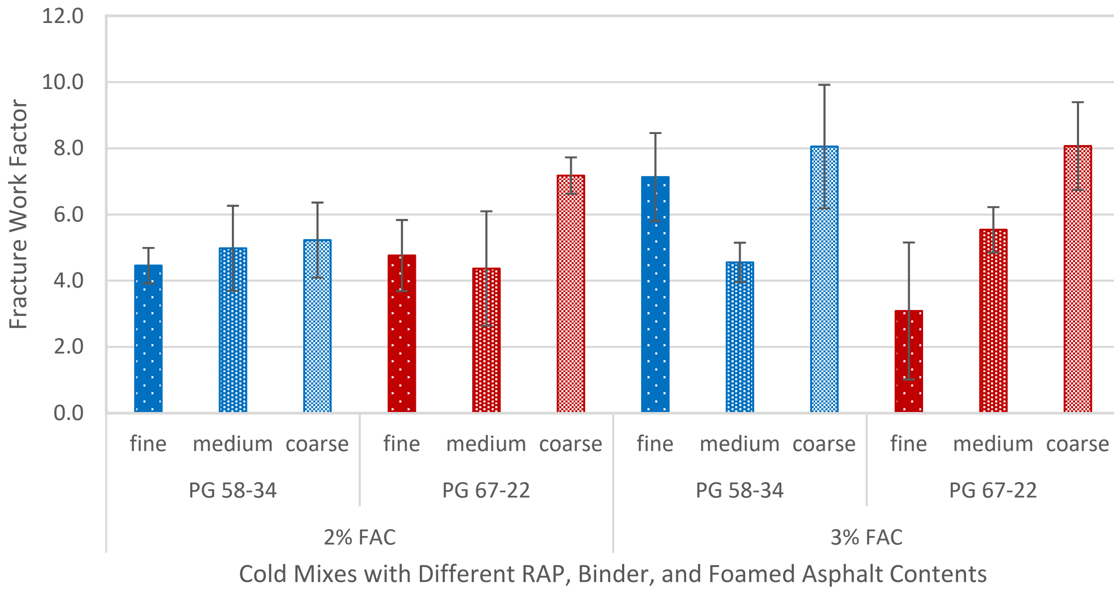


Figure 6.13 Comparison of FWF Results for the Cold Recycled Mixtures

Statistical analysis was conducted to determine the effect of RAP, binder, and FAC. The ANOVA was not performed to evaluate these effects altogether because the normality assumption was violated. Instead, Tukey's test and t-test were used to determine one effect at a time with the assumptions (normality and homogeneity of variance) satisfied. The assumption of independence for these two statistical methods were not examined because each test result was obtained from an independent specimen. Tukey's test was performed to assess the effect of RAP and the results are shown in Table 6.14. Mixtures with coarse RAP were categorized into one group, while mixtures with medium and fine RAP were categorized into another group. This suggested that the mixtures with coarse RAP had significantly higher FWF results than the mixtures with other RAP materials. FWF results of mixtures with fine and medium RAP were not statistically significant. Table 6.15 shows the *t*-test results for evaluating the effects of binder and FAC on the FWF results, respectively. For the comparison of two virgin binder, the assumptions of *t*-test were satisfied only for the results of mixtures at 3% FAC. The results showed the mixtures with two binders were not statistically different. For the comparison of two FAC, the assumptions of *t*-test were satisfied for the mixtures with medium and coarse RAP only. The results showed the difference between 2 and 3% foamed asphalt contents was not statistically significant when medium RAP was used but became significant when the mixture contains coarse RAP. Therefore, the mixtures with coarse RAP had better resistance to cracking than those with finer RAP. Mixtures with the binder PG 58-34 and PG 67-22 may not have significant difference in cracking resistance. In addition, increase of FAC did not necessarily improve the resistance of the mixture to cracking. This may be dependent on the RAP gradation.

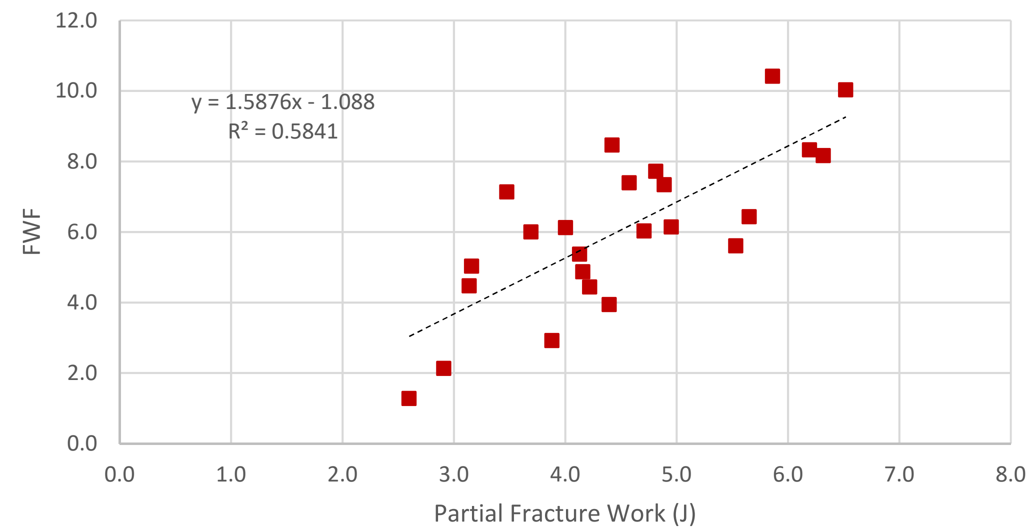
Table 6.14 The Effect of RAP Gradation on the FWF Results Based on Tukey's Test

RAP Gradation	Mean FWF Results	Grouping
Fine	4.9	B
Medium	4.9	B
Coarse	7.2	A

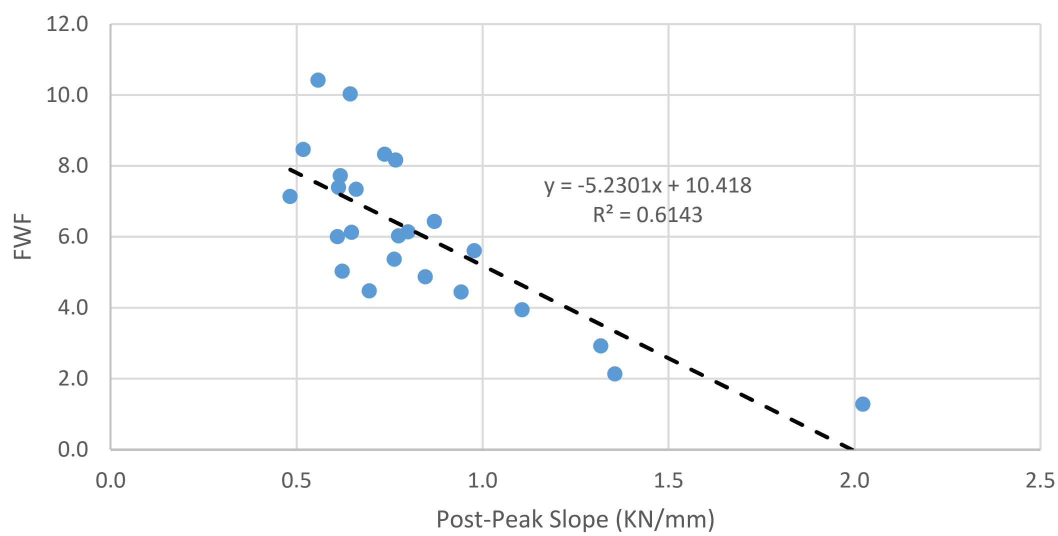
Table 6.15 The Effects of Binder PG and FAC on the FWF Results Based on *t*-Test

Comparison	P-value	Significant (Y/N?)
Binder PG 67-22 vs. PG 58-34 (3% FAC)	0.284	N
FAC 2% vs. 3% (medium RAP)	0.539	N
FAC 2% vs. 3% (coarse RAP)	0.021	Y

The FWF results were correlated with the partial fracture work and the post-peak slope (absolute value). As shown in Figure 6.14, both correlations were good while FWF results were more dependent on the post-peak slope. Compared to the I-FIT method, this method had same level of correlation with the area (fracture energy/partial fracture work) but more sensitive to the post-peak slope. Zhou et al. (2017) found the macro-crack started to initiate and propagate on the test specimens after the peak load. Therefore, the FWF result was more sensitive to the speed of the macro-crack development than the FI result.



(a) Correlation between the FWF results and the partial fracture work



(b) Correlation between the FWF results and the post-peak slope

Figure 6.14 Correlation between the FWF Result and the Two Parameters

Furthermore, the FWF result were correlated to the FI results. Their rankings of mixtures for the cracking resistance were also compared. Correlation between the two results was not strong (R-squared = 0.15) but their rankings were similar as shown in Table 6.14. Only the FWF results at 3% FAC were used for comparison because the FI results were only available for mixtures with 3%FAC. Two methods ranked these mixtures in the same way except for the mixture with PG 67-22 binder and fine RAP (mix 1). All the mixtures other than mix 1 were ranked following the same order. Spearman's correlation coefficient (r_s) was calculated for the two ranking orders. Although the coefficient was equal to 0.43 including mix 1, the coefficient was determined as 1.00 without mix 1. For the remaining cold recycled mixes, mix 3 had the highest ranking, followed by mix 6, mix 4, mix 2, and then mix 5. Rankings of these mixtures suggested it would be better to select coarse RAP to design crack-resist cold recycled mix whether soft or hard binder is used. Using fine RAP seemed to have better cracking resistance than using medium RAP if soft binder is used. Differences between fine RAP and medium RAP were not evident based on both FWF and FI results.

Table 6.16 shows the FWF result ranked mix 1 at the 6th place but the FI value ranked it at the 2nd place. The results of area (fracture energy/partial fracture work) and post-peak slope were examined to investigate possible reasons because these two were the major parameters. For the result of area, both I-FIT and FWF methods gave mix 1 similar results as other mixes. However, for the result of post-peak slope, the FWF method gave mix 1 much lower result than other mixtures while the I-FIT method captured its lowest slope but the slope was not significantly different from other mixes. This may be due to the differences between the ITS test and the SCB test in loading mode and specimen geometry.

Table 6.16 Comparison for Rankings of Mixtures Based on the FWF and the FI Results

Mix	Virgin Binder	RAP	Avg. FWF Results	Ranking by FWF	Avg. FI Value	Ranking by FI
1	PG 67-22	Fine	3.1	6	5.8	2
2		Medium	5.5	4	3.8	5
3		Coarse	8.1	1	8.4	1
4	PG 58-34	Fine	7.2	3	4.3	4
5		Medium	4.6	5	3.6	6
6		Coarse	8.1	2	5.3	3

The results showed two methods ranked the cold recycled mixes in the same order except for the fine RAP mix with the stiffer binder. The COV of FWF results was 25.1%, a little lower than FI result's 33.5%. Further validation based on field cracking performance is necessary to compare the two laboratory testing results.

6.2.6 Evaluation of the Test Result Variability

Suitable test methods for evaluating cold recycled foamed asphalt mixtures should have relatively low variability. Table 6.17 summarizes the variability of the laboratory testing methods and results used to characterize the cold recycled mixes in this study. The COV results of these results were compared with typical COV results of HMA materials. Generally, the cold recycled foamed asphalt mixtures had higher COV than HMA materials. The higher COV value may be due to the greater variability in the cold recycled materials. Among these results, the ITS results had similar COV ranges as HMA. Other test results had much higher COV for cold recycled mixes than HMA materials. Among these results, the two results from FN test had the highest variability than other results. The ITS test had the lowest variability.

Table 6.17 Variability of the Laboratory Testing Results for Evaluating the Cold Recycled Foamed Asphalt Mixtures Tested

Test Method	Test Result			Cold Recycled Mix Average COV (%)	Cold Recycled Mix COV Range (%)	Typical HMA COV Range (%)
Indirect Tensile Strength[1]	ITS (kPa)			6.0	0.4 – 9.5	4.0 – 5.0
Dynamic Modulus[2]	$\|E^*\|$ (MPa)	4°C	0.1 Hz	15.2	2.7 – 26.3	2.5 – 5.8
			1 Hz	14.9	0.9 – 36.6	1.8 – 5.2
			10 Hz	14.5	1.7 – 35.0	2.2 – 5.2
		20°C	0.1 Hz	16.9	6.8 – 29.5	1.3 – 13.4
			1 Hz	15.0	3.8 – 29.7	1.1 – 11.1
			10 Hz	12.6	0.8 – 24.9	2.6 – 12.2
		40°C	0.01 Hz	16.0	9.9 – 25.9	N/A
			0.1 Hz	13.0	6.7 – 19.3	0.7 – 4.8
			1 Hz	10.7	4.3 – 15.6	3.0 – 8.4
			10 Hz	8.7	2.5 – 12.9	2.5 – 13.1
Flow Number[3]	k_{st} (με/cycle)			45.0	13.6 – 82.2	4.3 – 33.3
	ε_{st} (με)			31.8	13.6 – 83.4	1.6 – 29.0
I-FIT[4]	FI			33.5	19.5 – 44.4	10.0 – 20.0
Indirect Tensile Strength[5]	FWF			25.4	12.4 – 67.1	5.6 – 26.0

Notes:

1. The ITS results of HMA material was based on the mixtures compacted to 7% air void content by SGC from more than 18 laboratories. Mixtures included both limestone and sandstone mixtures (Azari et al. 2010).
2. The HMA dynamic modulus results were based on nine HMA materials (9.5 mm and 19 mm NMAS) used in NCHRP Project 9-47A (West et al. 2014).
3. The Flow Number test result for HMA was obtained based on 6 mixes with aggregate's NMAS ranged from 9.5 to 19 mm, and binders' high temperature PG ranged from 58 to 67 (West et al. 2013).
4. Al-Qadi et al. (2015) reported the typical COV range of FI results based on 11 lab-produced mixes and 6 plant-produced mixes.
5. The FWF results of typical HMA were determined using 10 mixes from FHWA's ALF lanes (West et al. 2017).

6.2.7 Evaluation of Performance Tests of Cold Recycled Mixtures

6.2.7.1 Evaluation of the Resistances to the Permanent Deformation and Cracking

This study used five laboratory testing methods to evaluate the cold recycled foamed asphalt mixtures with different RAP materials and binder.

Current mix design methods for the cold recycled mixes rely primarily on the ITS results because the test method is convenient. The test specimens are easy to prepare in the laboratory. However, the ITS results may not be able to characterize the mixture's resistance to permanent deformation and cracking. The accumulated strain at the beginning of the steady state (N_{st}) from the Flow Number test and the fracture work factor results from the ITS test were used to indicate a cold recycled mixes' resistance to permanent deformation and cracking, respectively. These two tests were selected because of their relatively low variabilities. The results of the six cold recycled mixtures were plotted in Figure 6.15. More desirable results would be plotted on the upper left part (low accumulated strain and high Fracture Work Factor) of this graph. However, currently, no criteria have been established for these tests for the cold recycled mixtures.

Figure 6.15 shows that mixtures with coarse RAP tended to have higher FWF and are presumed to have better resistance to cracking. Mixtures with PG 67-22 binder had lower accumulated strain and are presumed to have better resistance to permanent deformation. Among these six cold recycled mixtures, the mixture with coarse RAP and PG 67-22 binder are presumed to have the best overall resistance to cracking and permanent deformation because of its high FWF and low accumulated strain results. Therefore, coarse RAP and higher PG binder with adequate foaming qualities (ER and HL criteria satisfied) are recommended for cold recycled foamed asphalt mixture.

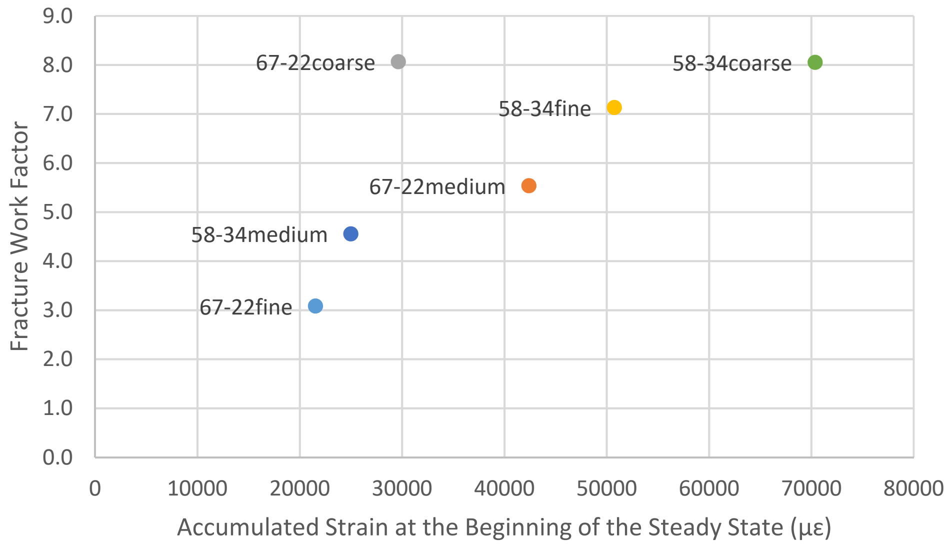


Figure 6.15 The FWF Results vs. the Accumulated Strain at the Beginning of the Steady State (N_{st})

6.2.7.2 Comparison of Cold Recycled Mixes and Typical HMA for the Laboratory Test Results

After conducting the laboratory testing for the cold recycled mixes, the test results were compared to typical HMA results. Table 6.18 summarizes the average results of cold recycled mixes and typical HMA average results. The results showed the HMA results were much better than cold recycled mixes, except for cracking resistance. The average ITS result of HMA was more than three times that of the cold recycled mixes. The average dynamic moduli of HMA were up to six times of the moduli of cold recycled mixes. The differences in moduli were much more significant at lower temperatures and higher frequencies, indicating HMA modulus was more sensitive to the change of temperature and frequency than cold recycled mixes. Only the FI results were similar between cold recycled mixes and HMA materials.

This comparison shows that cold recycled mixtures are a much lower quality material than HMA. Cold recycled mixtures are therefore better suited for use as a base or subbase layer within the pavement structure.

Table 6.18 Comparison of the Cold Recycled Mixtures and Typical HMA for the Laboratory Testing Results

Test Method	Test Result			Cold Recycled Mix Average Result	Typical HMA Average Result	Typical HMA Result Range
Indirect Tensile Strength[1]	ITS (kPa)			250	802	561 – 1106
Dynamic Modulus[2]	$\|E^*\|$ (MPa)	4°C	0.1 Hz	1,694	8,634	4,460 – 11,809
			1 Hz	2,207	12,352	7,614 – 15,014
			10 Hz	2,883	16,235	11,409 – 18,157
		20°C	0.1 Hz	707	2,666	1,074 – 4,177
			1 Hz	1,039	4,954	2,173 – 6,859
			10 Hz	1,603	9,114	4,437 – 17,983
		40°C	0.01 Hz	153	N/A	N/A
			0.1 Hz	216	873	537 – 1,136
			1 Hz	347	1,455	710 – 1,992
			10 Hz	709	2,866	1,186 – 3,842
Flow Number[3]	k_{st} (με/cycle)			3.4	0.7	0.3 – 1.3
	ε_{st} (με)			39,952	12,361	7,944 – 16,777
I-FIT[4]	FI			5.2	5.3	0.3 – 16.0
Indirect Tensile Strength[5]	FWF			5.6	20.8	10.6 – 27.5

Notes:
1. The ITS results of HMA material was based on the mixtures compacted to 7% air void content by SGC from more than 18 laboratories. Mixtures included both limestone and sandstone mixtures (Azari et al. 2010).
2. The HMA dynamic modulus results were based on nine HMA materials (9.5 mm and 19 mm NMAS) used in NCHRP Project 9-47A (West et al. 2014).
3. The Flow Number test result for HMA was obtained based on 6 mixes with aggregate's NMAS ranged from 9.5 to 19 mm, and binders' high temperature PG ranged from 58 to 67 (West et al. 2013).
4. The FI results of HMA were obtained based on 18 mixes consisted of 11 laboratory mixes and 7 plant mixes (Al-Qadi et al. 2015).
5. The FWF results of typical HMA were determined using 10 mixes from FHWA's ALF lanes (West et al. 2017).

6.3 Summary

Five laboratory tests used to evaluate cold recycled mixes in this study were ITS, dynamic modulus, Flow Number, I-FIT, and the FWF. Since there was no standard laboratory testing method for cold recycled mixes, this study obtained typical results for HMA characterization and proposed three new methods to evaluate the cold recycled mixes. Based on the results presented above, the following findings and conclusions were made:

- The ITS results showed that the cold recycled foamed asphalt mixture with finer RAP and binder of higher PG and adequate foaming properties (ER and HL) tended to have higher ITS.
- The results from dynamic modulus test suggested that the RAP gradation had more influence on cold recycled foamed asphalt mixtures than binder grade. However, the small-scale dynamic modulus test may not be ready to be implemented to represent cold recycled foamed asphalt mixtures in the pavement M-E design. The issues in preparing specimens with low-density mixtures and the data quality indicators need to be investigated.
- The steady-state strain development in the Flow Number test was defined. Two parameters were proposed for evaluating the resistance of cold recycled mixes to the permanent deformation. The steady-state strain accumulation rate (k_{st}) had higher variability than the accumulated strain at the beginning of the steady-state phase. Both parameters showed that mixtures with fine RAP had better resistance to permanent deformation than the mixture with coarse RAP. The effect of binder PG on the resistance to permanent deformation was not evidently shown.
- The FI result can distinguish different RAP gradations of cold recycled mixtures and possibly provide an indication of their cracking resistance. This test result may also be suitable to characterize the mixtures with different binder properties if the variability is reduced. In this study, the mixture with coarse RAP had higher FI result than that with fine RAP. Therefore, the mixture with coarse RAP had better resistance to cracking.
- The FWF result was proposed based on the ITS test, which was inspired by the concept of FI result in I-FIT. The two results were found to rank the cold recycled mixes similarly. The variability of FWF result was a little lower than FI. The mixture with coarse RAP had higher FWF result than those with medium and fine RAP. The cracking resistance of mixtures with coarser gradations was better.

CHAPTER 7 CONCLUSIONS AND RECOMMENDATIONS

This study focused on different aspects of the cold recycled foamed asphalt mixtures. Three topics studied included the mix design method, mineral additives effect, and laboratory test results. The conclusions and recommendations from each study were summarized in this chapter.

7.1 Mix Design

One of the objective of this study was to improve the mix design procedure. Therefore, three improvements were proposed to the typical design method regarding the compactive effort, the determining method for RAP material's OWC, and the determining method for the OTWC of mixture. The following findings and conclusions can be made.

- The compactive effort using Superpave gyratory compactor was validated as 30 gyrations for cold recycled foamed asphalt mixtures based on the comparison with field compactive effort.
- A new method to determine the RAP material's OWC using the proposed compactive effort by SGC was proposed. This method allowed designers to use same compactive effort for the RAP and the mixture. Also, it saved time and effort for the designer when the automatic modified Proctor hammer was not available. Compared to the modified Proctor test method, the OWC of RAP material determined by the proposed method was lower.
- The total water content (TWC) had significant effect on the density and strength of cold recycled mixes. The proposed models for determining the OTWC were built based on six cold recycled mixes with two different binders and three RAP materials. The models used three predictors (including RAP material's OWC, binder PG, and FAC) to predict the OTWC. The predicting capability of the models was evaluated using six mixtures other than those used for model regression. The predicted OTWC results were a little lower than the measured OTWC. Two of the six mixtures under-predicted the OTWC more than 1.0% due to the use of binder from another source. This indicated that the current way to distinguish foamed asphalt based on the virgin binder PG was not enough. Another factor to indicate the overall foaming properties may be used for calibrating the models in the future.
- A mix design procedure was summarized based the three improvements. The effectiveness of this design method needs to be validated in the field.

7.2 Mineral Additives

This part of research investigated the effects of different mineral additives on the indirect tensile strengths of cold recycled foamed asphalt mixtures subjected to laboratory and field curing conditions. These additives included cement, hydrated lime, fly ash, and baghouse fines. A control mixture without additive was used for comparison. The findings and conclusions for this part of research are:

- After the laboratory curing condition (3 days at 40°C), cement was the best overall mineral additive considering improvements to both dry and wet ITS of the mixtures. Mixtures with cement and baghouse fines had higher dry ITS than mixtures with hydrated lime, fly ash, and no additive. Mixtures with cement and hydrated lime had higher wet ITS. Higher additive contents do not necessarily improve dry or wet ITS of mixtures. Only the ITS results of the mixture with cement increased as more cement was added.
- Curing moisture had a significant effect on a mixtures' dry ITS when cement or baghouse fines was used. Sufficient moisture helped increase hydration of cement but it significantly reduced the dry ITS of mixture with baghouse fines.
- Dry ITS of all mixtures with and without mineral additives increased with field curing time. After 30 days of field curing, changes in dry ITS were little for the mixtures with cement, baghouse fines, fly ash, and the control mixture. Mixtures with hydrated lime required more than 100 days to develop equivalent strengths as those with other additives.
- Top load and confinement had a significant effect on the mixture with cement. For mixtures with other additives, this effect on dry ITS was negligible. Comparing laboratory and field curing results, test results of mixtures cured for 3 days at 40°C provided consistent ranking as long-term curing (at least 100 days). Due to their higher wet-ITS values, cold recycled foamed asphalt mixtures using cement, hydrated lime, and fly ash are better for foamed asphalt mixtures than baghouse fines or no additive where minimum wet ITS criteria and/or TSR criteria are used.

7.3 Laboratory Testing Study

Five laboratory tests used to evaluate cold recycled mixes included the indirect tensile strength test, dynamic modulus test, Flow Number test, I-FIT, and the FWF method. This study obtained typical results used in HMA characterization along with the three proposed results to evaluate the cold recycled mixes. Based on the results presented above, the following findings and conclusions were made:

- The ITS results showed that the cold recycled foamed asphalt mixture with the finer RAP and higher PG binder with adequate foaming qualities (ER and HL criteria satisfied) had higher ITS results.
- RAP gradation had more influences on the dynamic modulus of cold recycled foamed asphalt mixtures than the effect of binder. However, the small-scale dynamic modulus test may not be ready to be implemented in the pavement M-E design to characterize the cold recycled foamed asphalt mixtures. Issues of specimen preparation and data quality indicators need to be solved.
- The steady-state strain development in the Flow Number test was defined. Two parameters were proposed for evaluating the resistance of cold recycled mixes to the permanent deformation. The steady-state strain accumulation rate (k_{st}) had higher variability than the accumulated strain at the beginning of the steady-state phase. This accumulated strain represented between 40 – 80% of the total accumulated strain. Both parameters showed that mixtures with fine RAP had better resistance to permanent deformation than the mixture with coarse RAP. The effect of binder PG on the resistance to permanent deformation was not evidently shown.
- The FI result can distinguish different RAP gradations of cold recycled mixtures and possibly provide an indication of their cracking resistance. This test result may also be suitable to characterize the mixtures with different binder properties if the variability is reduced.
- The FWF result was proposed based on the ITS test, which was inspired by the concept of FI result in I-FIT. The two results were found to rank the cold recycled mixes similarly. The variability of FWF result was a little lower.

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APPENDICES

Appendix A – Mix Design Study

RAP Material's OWC Determined by the Superpave Gyratory Compactor

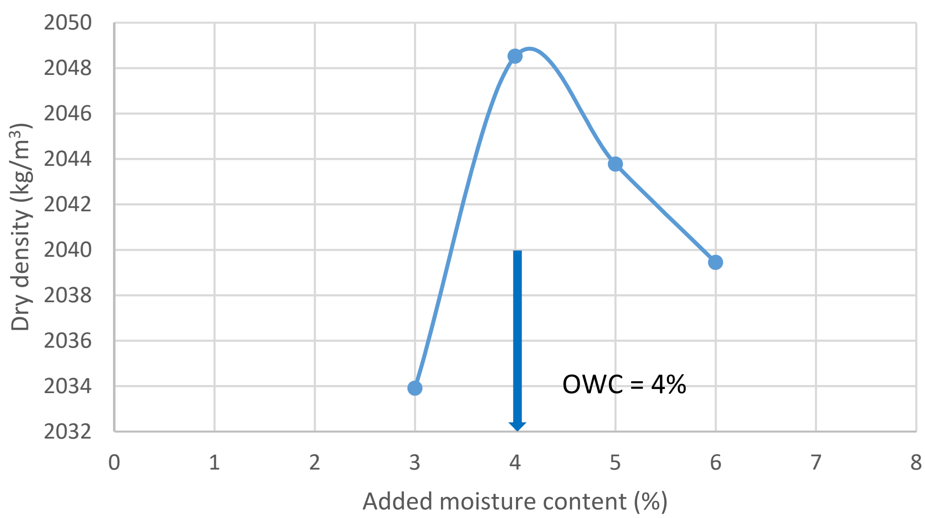

Figure A.1 Dry Density vs. Added Water Content by SGC for the DF(A) RAP

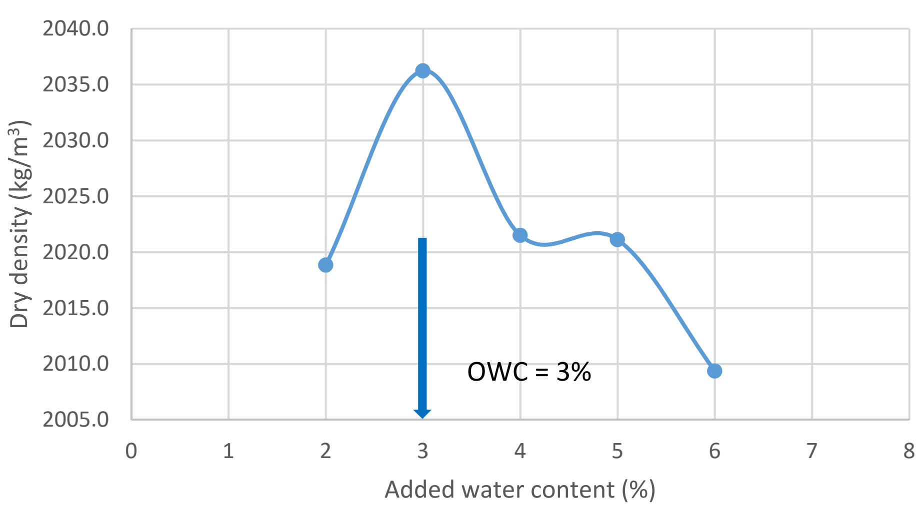

Figure A.2 Dry Density vs. Added Water Content by SGC for the DC(C) RAP*

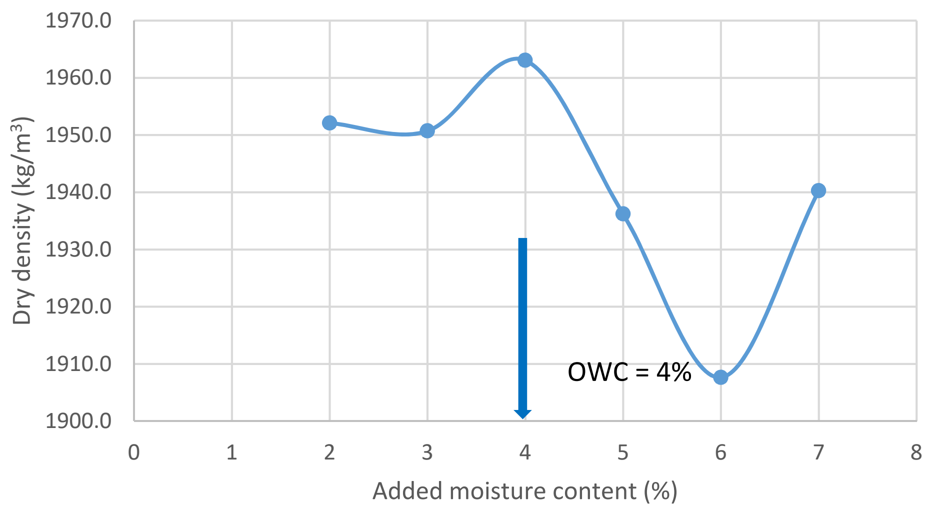

Figure A.3 Dry Density vs. Added Water Content by SGC for the VM1(D) RAP

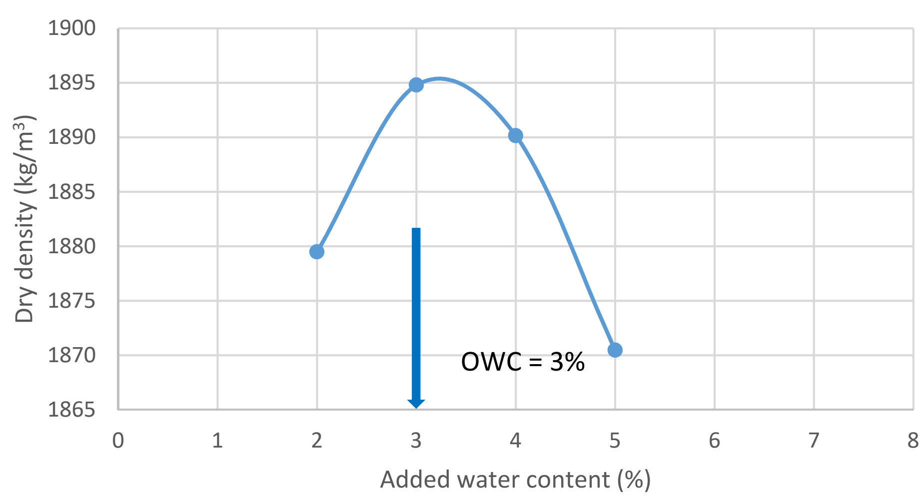

Figure A.4 Dry Density vs. Added Water Content by SGC for the VC1(E) RAP

RAP Material's OWC Determined by the Modified Proctor Test

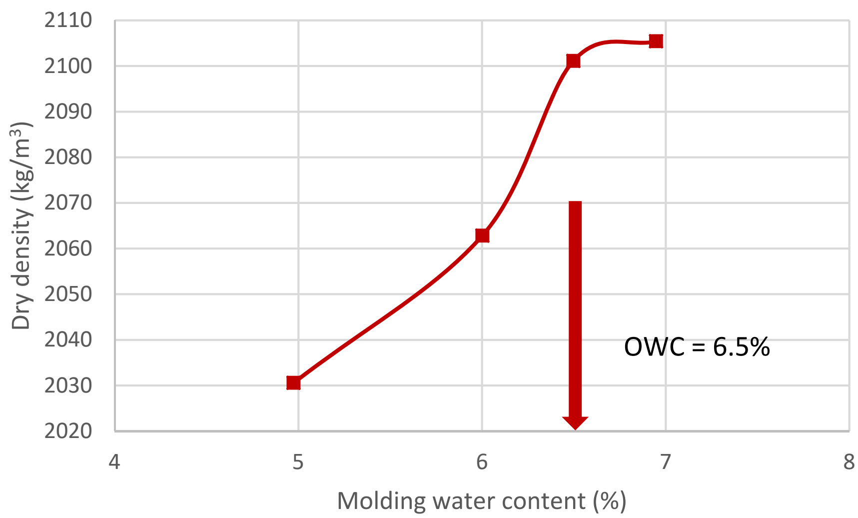

Figure A.5 Dry Density vs. Molding Water Content by the Modified Proctor Test for the DF(A) RAP

Note: For the DF(A) RAP, 7% added water achieved 6.5% molding water; 8% added water achieved 6.9% molding water. Adding 1% extra water neither increased much molding water content nor dry density but introduced excessive water. It will also bump up the total water content in the cold recycled mix which requires longer time to cure in the field before overlay or traffic. Therefore, the OWC of this DF(A) RAP by the modified Proctor test was determined as 6.5% instead of 6.9%.

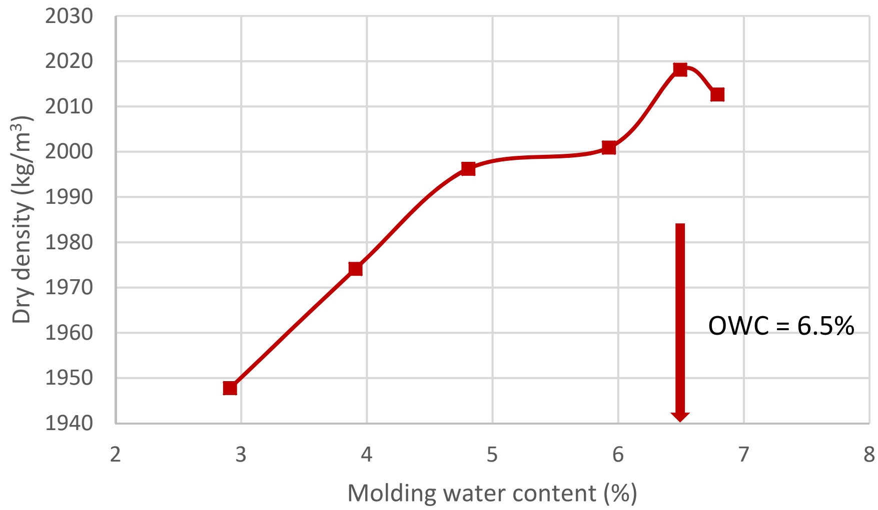

Figure A.6 Dry Density vs. Molding Water Content by the Modified Proctor Test for the VM1(D) RAP

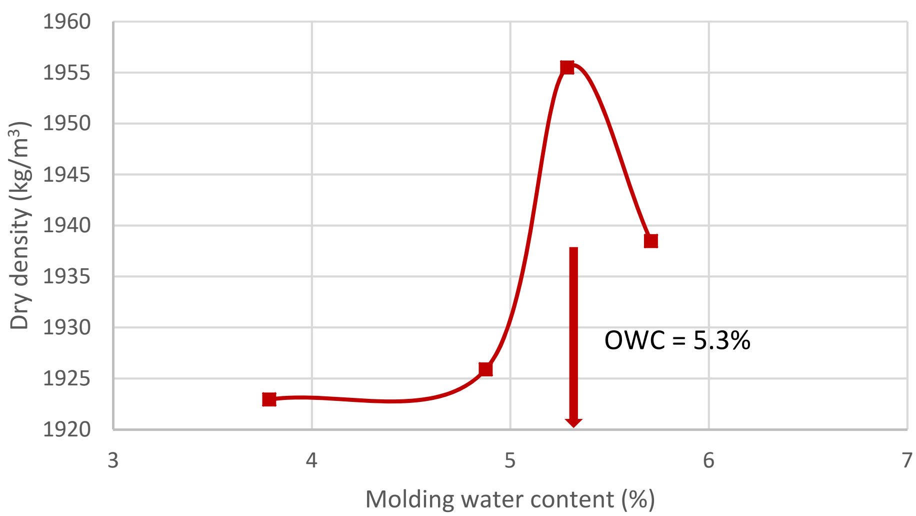

Figure A.7 Dry Density vs. Molding Water Content by the Modified Proctor Test for the VC1(E) RAP

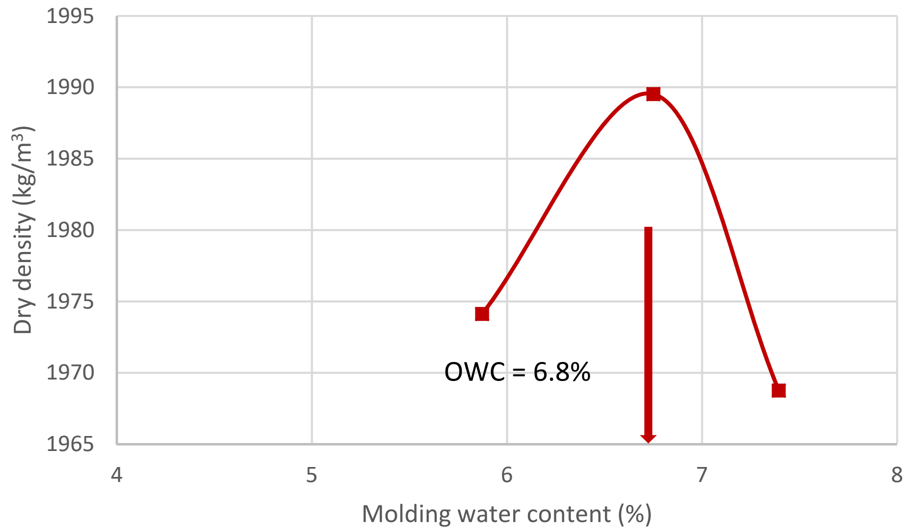

Figure A.8 Dry Density vs. Molding Water Content by the Modified Proctor Test for the VM2(G) RAP

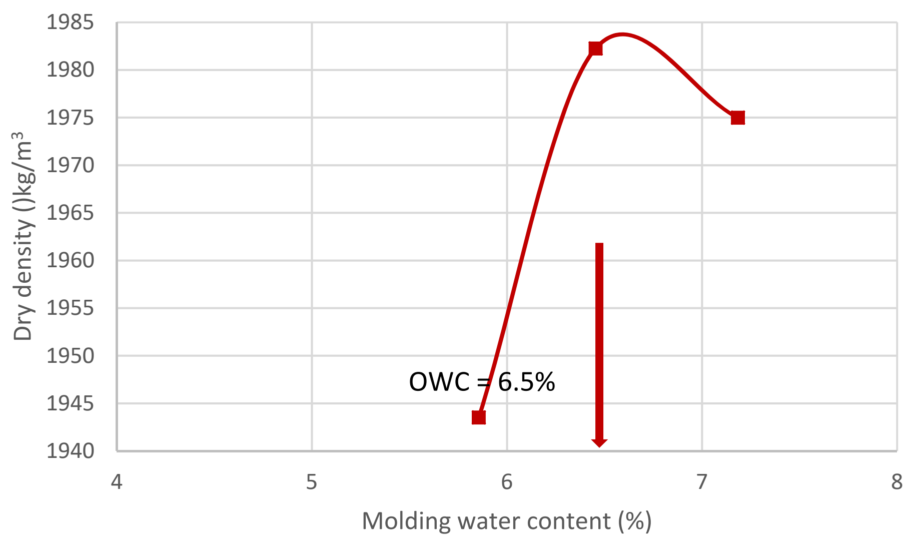

Figure A.9 Dry Density vs. Molding Water Content by the Modified Proctor Test for the VF(H) RAP

Note: Figure A.8 and A.9 showed the RAP materials used for US Hwy 280 cold recycling mix design. The RAP materials were crushed from the field cores and the amount was limited. Only two buckets of each RAP material were available for testing. Because these materials had similar gradations as previously tested RAP materials, only three water contents were tested to determine the OWC, saving RAP materials for producing foamed asphalt mixtures.

Foamed Asphalt Expansion Ratio and Half-Life Curves

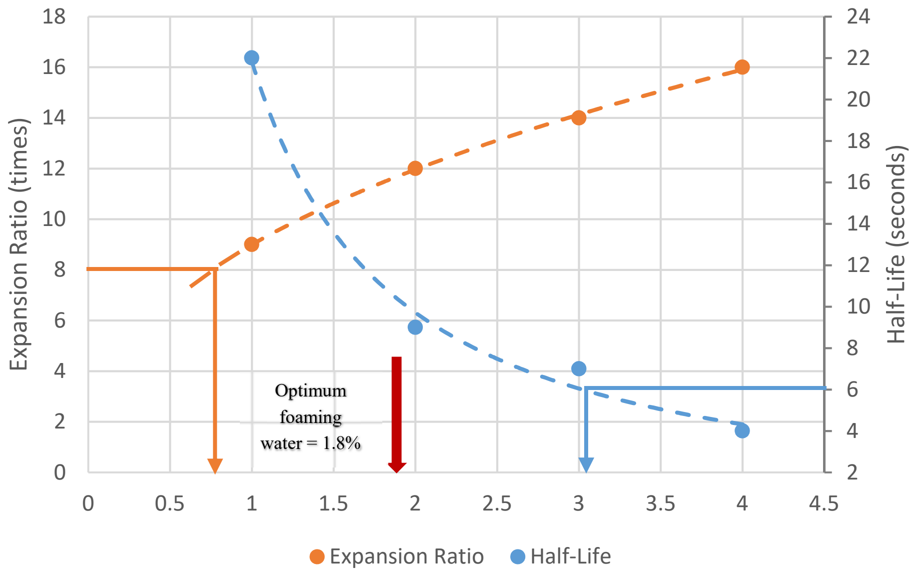

Figure A.10 Foaming Property Curves of Asphalt PG 67-22 (A)

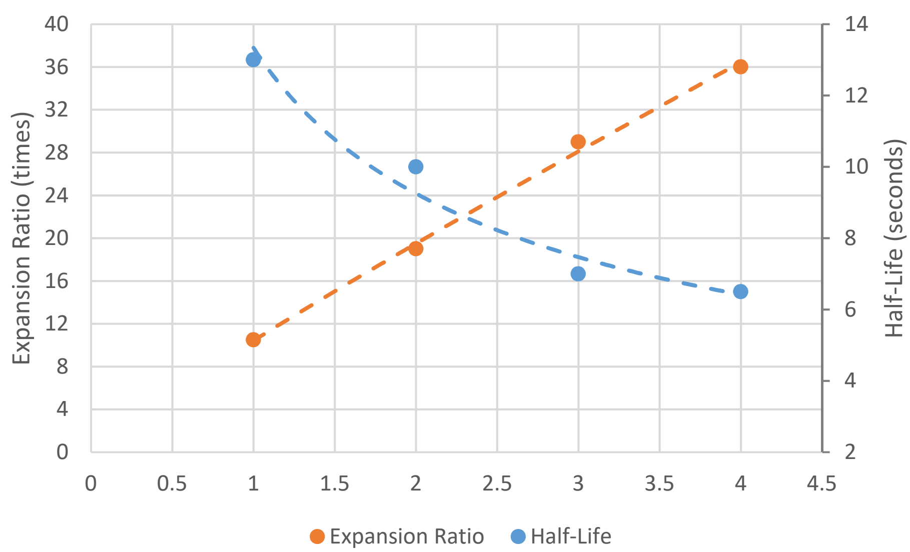

Figure A.11 Foaming Property Curves of Asphalt PG 58-34 (B)

Note: The minimum expansion ratio (8 times) corresponds to 0.7% water content and the minimum half-life (6 seconds) corresponds to 4.6% water content. This water content range (0.7 – 4.6%) was not plotted because the displayed range of the interpolated regression curve is limited in the figure. The initial optimum foaming water content at the center of this range was 2.7%. However, this was considered too high that would cause 25 times expansion ratio but only 8 seconds half-life. To make this foamed asphalt more stable or with longer half-life, the water content was reduced to 2.0% with sufficient expansion ratio (19 times) and extended half-life (10 seconds).

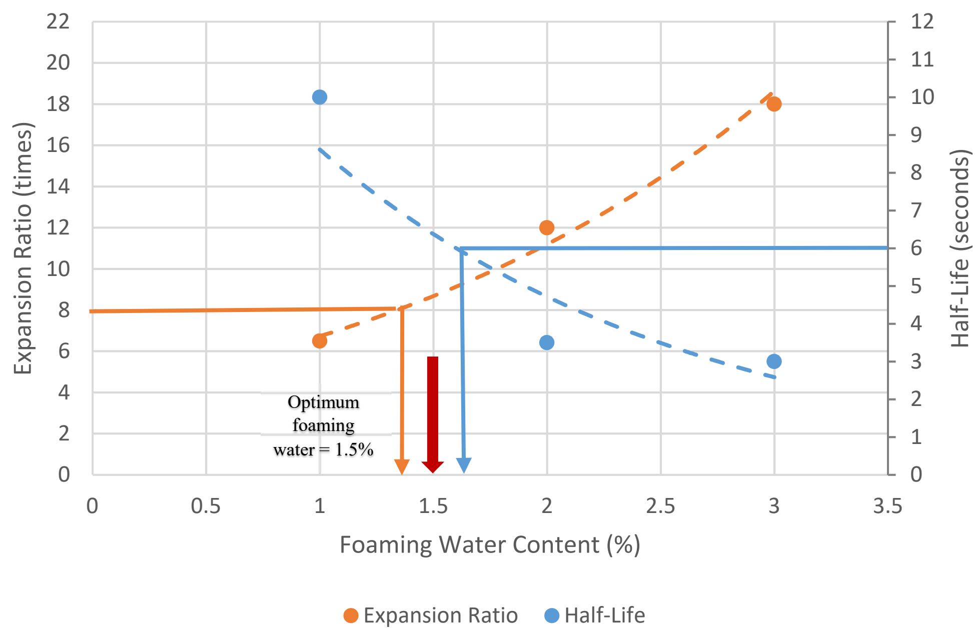

Figure A.12 Foaming Property Curves of Asphalt PG 67-22 (C)

Note: Minimum expansion ratio (8 times) and half-life (6 seconds) were used to select the initial optimum foaming water content as 1.5% by mass of asphalt. Then, the water content was adjusted to 1.3% to avoid the problem of marginal half-life.

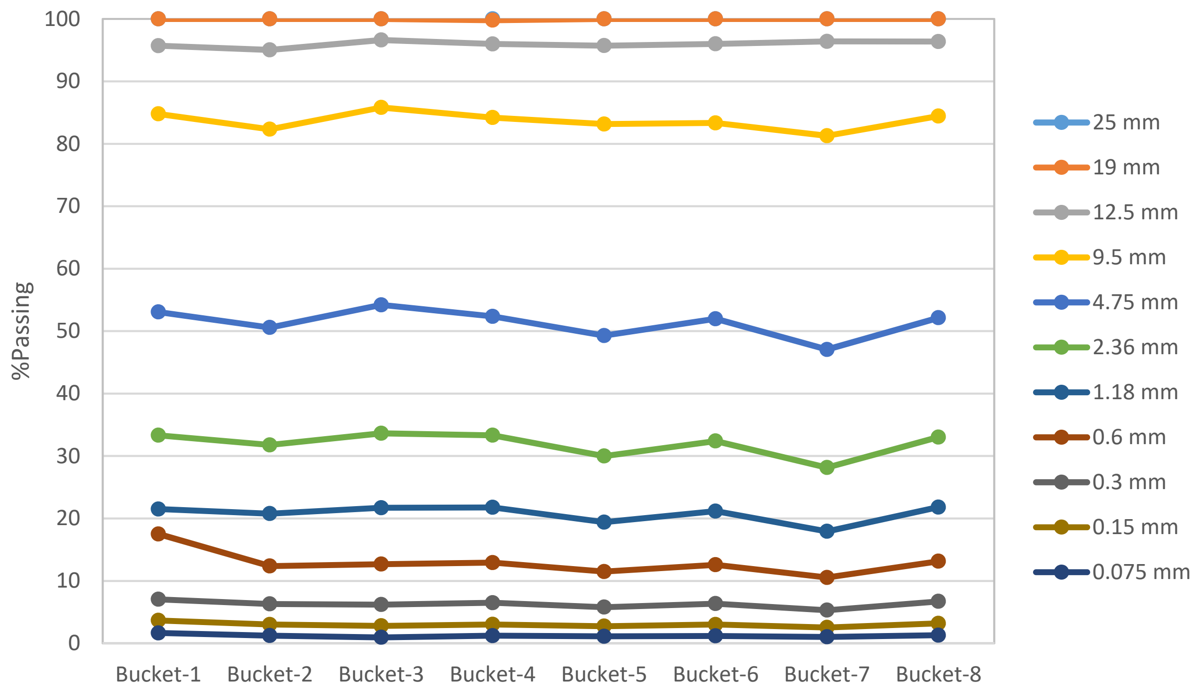

Figure A.13 Randomly Tested Gradation for Type A RAP Stored in Buckets

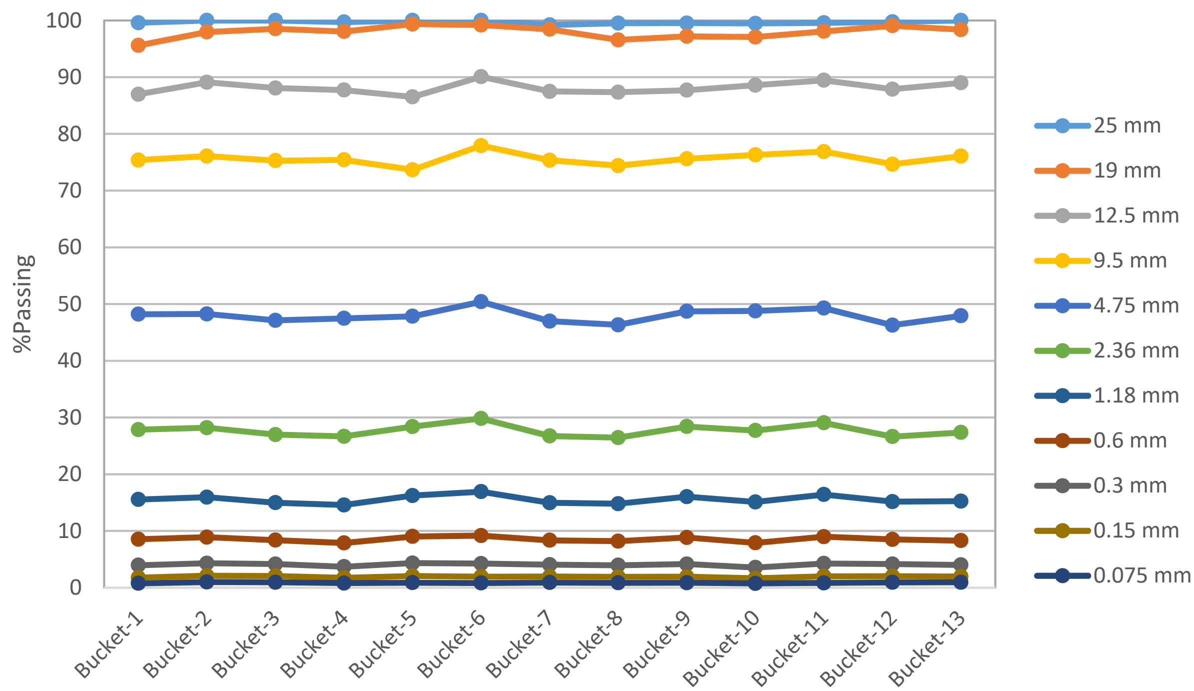

Figure A.14 Randomly Tested Gradation for Type B RAP Stored in Buckets

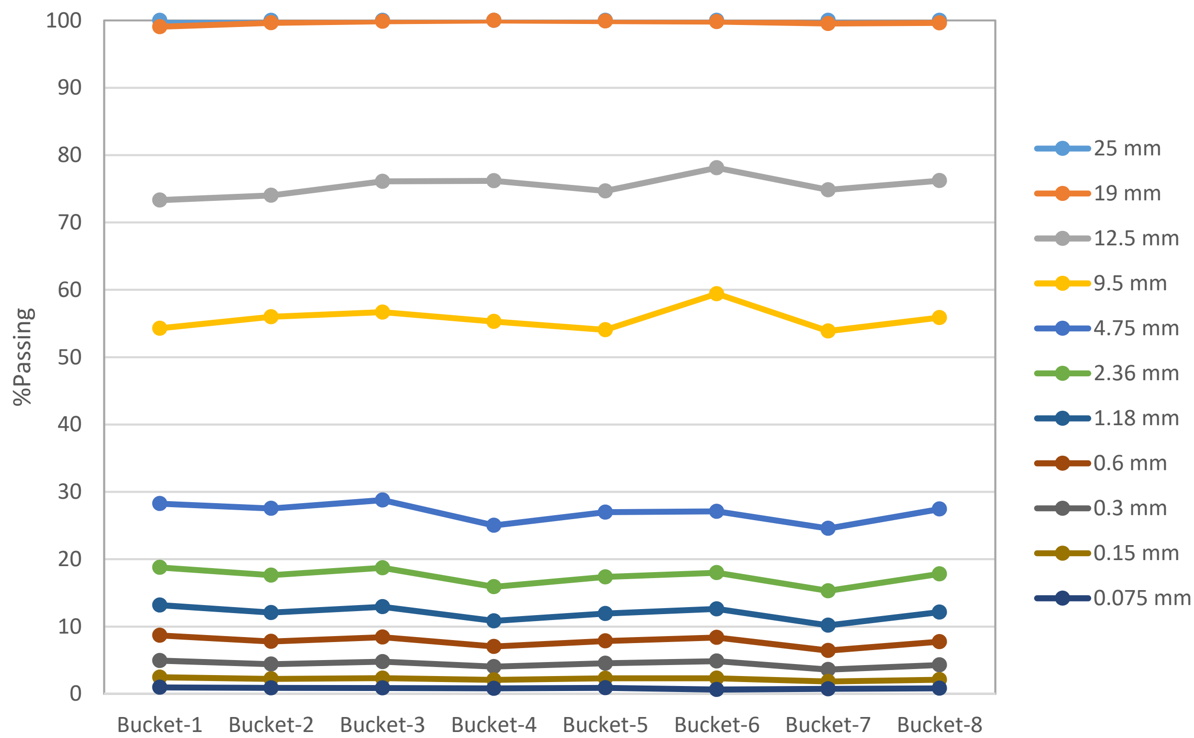

Figure A.15 Randomly Tested Gradation for Type C RAP Stored in Buckets

Appendix B - Additive Study

Table B.1 Summary of ITS Results for Specimens Cured in Dry and Wet Conditions

Additive in Mix	Additive Content	Dry ITS			Wet ITS			TSR (%)
		Avg. (kPa)	Std. Dev. (kPa)	COV (%)	Avg. (kPa)	Std. Dev. (kPa)	COV (%)	
Control	0	178.6	8.0	4.5	107.7	4.0	3.8	60.3
Cement	1.0	194.7	13.4	1.6	181.6	8.3	4.9	93.3
	1.5	210.3	3.5	1.6	181.6	9.0	5.0	86.4
	2.0	239.7	6.3	2.6	193.2	31.7	16.4	80.6
Hydrated lime	1.0	186.0	17.4	9.4	163.2	7.2	4.4	87.7
	1.5	183.0	16.0	8.7	159.7	17.1	10.7	87.3
	2.0	201.8	16.7	8.3	145.8	24.8	17.0	72.2
Baghouse Fines	1.0	171.0	23.9	14.0	118.3	7.4	6.3	69.2
	1.5	215.8	15.0	6.9	118.6	8.2	7.0	55.0
	2.0	218.2	7.4	3.4	110.1	10.7	9.7	50.5
	5.0	192.9	7.7	4.0	128.6	7.9	6.1	66.7
Fly Ash	1.0	168.0	22.2	13.2	133.4	12.2	9.2	79.4
	1.5	149.5	7.4	4.9	128.2	10.8	8.4	85.8
	2.0	172.3	4.7	2.7	125.4	3.6	2.9	72.8
	5.0	201.0	16.1	8.0	160.1	11.5	7.2	79.7

Table B.2 Numerical Ranking for Laboratory-Cured Mixtures Based on the Dry and Wet ITS

Ranking	Dry ITS			Wet ITS		
	Additive Type	Additive Content (%)	Average (kPa)	Additive Type	Additive Content (%)	Average (kPa)
1	CEM	2.0	239.7	CEM	2.0	193.2
2	BHF	2.0	218.2	CEM	1.5	181.6
3	BHF	1.5	215.8	CEM	1.0	169.1
4	CEM	1.5	210.3	HL	1.0	163.2
5	HL	2.0	201.8	FA	5.0	160.1
6	FA	5.0	201.0	HL	1.5	159.7
7	CEM	1.0	194.7	HL	2.0	145.8
8	BHF	5.0	192.9	FA	1.0	133.4
9	HL	1.0	186.0	BHF	5.0	128.6
10	HL	1.5	183.0	FA	1.5	128.2
11	CTRL	0	178.6	FA	2.0	125.4
12	FA	2.0	172.3	BHF	1.5	118.6
13	BHF	1.0	171.0	BHF	1.0	118.4
14	FA	1.0	168.0	BHF	2.0	110.1
15	FA	1.5	149.5	CTRL	0	107.7

Table B.3 Summary of ITS Results for Specimens Cured in Dry and Moisture Rooms

Additive in Mix	Additive Content	Dry Room Curing			Moisture Room Curing			ITS Ratio (%)
		Avg. (kPa)	Std. Dev. (kPa)	COV (%)	Avg. (kPa)	Std. Dev. (kPa)	COV (%)	
Control	0	188.3	11.2	5.9	174.0	11.6	6.7	92.4
Cement	1.0	198.8	23.9	12.0	227.9	10.6	4.6	114.6
	1.5	217.2	1.8	0.8	222.6	10.4	4.7	102.5
	2.0	228.6	10.6	4.6	250.3	5.5	2.2	109.5
Hydrated lime	1.0	179.3	17.1	9.5	187.1	9.3	5.0	104.4
	1.5	186.7	5.4	2.9	177.0	14.5	8.2	94.8
	2.0	199.4	12.5	6.3	190.7	19.7	10.3	95.6
Baghouse Fines	1.0	219.7	16.7	7.6	166.8	11.0	6.6	75.9
	1.5	234.4	22.8	9.7	186.9	11.2	6.0	79.7
	2.0	227.5	5.4	2.4	161.7	21.6	13.4	71.1
	5.0	194.5	5.2	2.7	185.9	11.2	6.0	95.6
Fly Ash	1.0	183.4	7.1	3.9	180.3	20.9	11.6	98.3
	1.5	163.0	3.7	2.3	149.3	14.1	9.4	91.6
	2.0	158.2	8.1	5.1	154.4	19.8	12.8	97.6
	5.0	166.1	16.9	10.2	193.8	12.7	6.5	116.7

Table B.4 Summary of Dry ITS Results for Five Groups of Mixtures after Field Curing

Additive Type (1% Content)	Field 3 Day			Field 14 Day			Field 30 Day			Field 100 Day		
	Avg.	Std Dev.	COV	Avg.	Std Dev.	COV	Avg.	Std Dev.	COV	Avg.	Std Dev.	COV
CEM	124.3	11.3	9.1	187.0	35.3	18.9	187.0	35.3	18.9	201.9	10.4	5.1
HL	121.2	11.1	9.2	163.1	16.3	10.0	158.0	25.2	16.0	213.7	23.3	10.9
BHF	134.0	17.9	13.3	178.5	13.3	7.5	200.0	15.1	7.5	196.6	19.3	9.8
FA	108.9	17.9	16.4	150.6	16.0	10.6	194.9	6.5	3.3	195.5	6.4	3.3
CTRL	156.8	24.6	15.7	205.9	23.5	11.4	206.8	36.9	17.9	191.5	4.6	2.4

Appendix C - Laboratory Testing Study

Table C.1 Density, ITS, and FWF Results from the ITS Test

FAC	Binder	RAP	Density (kg/m^3)	ITS (kPa)	FWF
2%	67-22	Fine	2,043.8	252.5	6.4
			2,046.4	251.2	4.3
			2,047.9	234.9	4.3
			2,037.6	230.1	4.1
2%	67-22	Medium	1,962.3	215.9	3.1
			1,968.5	212.2	3.7
			1,948.8	183.7	3.7
			1,938.6	202.2	6.9
2%	67-22	Coarse	2,015.9	203.5	8.0
			2,014.8	184.4	6.9
			1,999.4	161.0	6.9
			1,981.4	167.8	6.9
2%	58-34	Fine	2,035.7	231.1	5.2
			2,028.8	249.4	4.5
			2,020.5	239.8	3.9
			2,021.3	228.0	4.2
2%	58-34	Medium	1,933.8	180.2	6.6
			1,947.6	166.2	4.6
			1,939.3	149.9	3.6
			1,949.8	148.2	5.1
2%	58-34	Coarse	2,019.0	164.6	4.2
			2,003.6	150.4	6.0
			2,010.7	151.1	6.4
			2,009.4	141.0	4.3
3%	67-22	Fine	2,060.4	305.6	1.3
			2,050.8	270.3	2.1
			2,048.1	311.6	2.9
			2,036.1	298.2	6.0
3%	67-22	Medium	1,982.7	257.4	6.1
			1,957.0	247.7	4.9
			1,957.2	260.9	6.1
			1,949.1	241.5	5.0
3%	67-22	Coarse	2,041.4	235.1	10.0

			2,022.9	239.4	7.4
			2,030.6	241.0	7.7
			1,988.7	200.0	7.1
3%	58-34	Fine	2,063.6	257.7	6.4
			2,075.1	257.2	8.3
			2,062.3	259.7	8.2
			2,043.2	259.0	5.6
3%	58-34	Medium	1,963.7	243.3	4.4
			1,968.3	224.3	5.4
			1,969.1	260.5	3.9
			1,966.2	209.4	4.5
3%	58-34	Coarse	2,065.3	213.5	8.5
			2,068.4	238.2	10.4
			2,078.9	245.8	7.3
			2,043.1	211.3	6.0

Table C.2 Volumetric Properties of Dynamic Modulus and Flow Number test Specimens

Mixture	Binder	RAP Gradation	No. of Replicate	Bulk Density (kg/m^3)	VMA (%)	VFA (%)	V_a (%)
1	PG 67-22	Fine	1	2,129.9	24.9	66.4	8.4
			2	2,138.6	24.6	67.5	8.0
			3	2,137.6	24.6	67.4	8.0
			4	2,119.2	25.3	65.1	8.8
2*	PG 67-22	Medium	1	1,884.1	33.1	43.7	18.6
			2	1,886.9	33.0	43.9	18.5
			3	1,899.1	32.6	44.8	18.0
			4	1,877.8	33.3	43.3	18.9
3	PG 67-22	Coarse	1	2,140.7	27.8	53.6	12.9
			2	2,107.0	28.9	50.7	14.3
			3	2,106.1	29.0	50.6	14.3
			4	2,085.0	29.7	48.9	15.2
4	PG 58-34	Fine	1	2,142.0	24.7	67.2	8.1
			2	2,155.9	24.2	69.0	7.5
			3	2,140.6	24.8	67.0	8.2
			4	2,147.6	24.5	67.9	7.9
5	PG 58-34	Medium	1	2,101.7	24.8	65.1	8.7
			2	2,082.4	25.5	62.7	9.5
			3	2,085.2	25.4	63.1	9.4
			4	2,075.5	25.7	61.9	9.8
6	PG 58-34	Coarse	1	2,199.9	23.2	66.1	7.9
			2	2,177.6	24.0	63.3	8.8
			3	2,126.9	25.7	57.6	10.9
			4	2,131.0	25.6	58.0	10.7

Note: *Density was determined by the dry mass and calculated volume from dimensions. These properties were not used to determine the master curve. Instead, additional two specimens' bulk specific gravity (G_{mb}) were measured following AASHTO T331 and used along with the theoretical maximum specific gravity (G_{mm}) to determine the properties. The average density, VMA, VFA, and V_a were 31.8%, 46.4%, and 17.1%.

Table C.3 Dynamic Modulus Results (Mixes with PG 67-22 Binder)

Mix	Temp. (°C)	Frequency (Hz)	Dynamic Modulus		
			Avg. (MPa)	Std. Dev.(MPa)	COV (%)
67-22 Fine	4	0.1	2,007.1	455.0	22.7
	4	1	2,521.4	549.4	21.8
	4	10	3,186.4	662.8	20.8
	20	0.1	825.7	243.4	29.5
	20	1	1,160.3	344.5	29.7
	20	10	1,706.7	424.9	24.9
	40	0.01	192.1	35.0	18.2
	40	0.1	279.2	47.6	17.0
	40	1	439.6	68.7	15.6
	40	10	836.5	62.9	7.5
67-22 Medium*	4	0.1	1,025.2	39.7	3.9
	4	1	1,243.4	40.7	3.3
	4	10	1,588.7	42.1	2.6
	20	0.1	534.4	36.1	6.8
	20	1	717.6	31.0	4.3
	20	10	1,059.7	34.6	3.3
	40	0.01	197.0	51.0	25.9
	40	0.1	239.5	46.2	19.3
	40	1	338.3	49.4	14.6
	40	10	599.7	59.0	9.8
67-22 Coarse	4	0.1	1,836.4	297.5	16.2
	4	1	2,390.4	380.2	15.9
	4	10	3,132.8	498.5	15.9
	20	0.1	686.0	184.4	26.9
	20	1	1,030.8	231.6	22.5
	20	10	1,666.4	305.6	18.3
	40	0.01	118.5	13.2	11.1
	40	0.1	185.5	19.6	10.6
	40	1	324.0	31.9	9.9
	40	10	726.2	75.1	10.3

Note: *Specimens were compacted to the testing dimension and tested without cutting or coring procedure.

Table C.4 Dynamic Modulus Results (Mixes with PG 58-34 Binder)

Mix	Temp. (°C)	Frequency (Hz)	Dynamic Modulus		
			Avg. (MPa)	Std. Dev. (MPa)	COV (%)
58-34 Fine	4	0.1	2,033.4	54.5	2.7
	4	1	2,721.1	25.7	0.9
	4	10	3,555.1	62.0	1.7
	20	0.1	820.0	62.0	7.6
	20	1	1,225.4	46.3	3.8
	20	10	1,864.7	14.0	0.8
	40	0.01	144.1	14.3	9.9
	40	0.1	213.6	14.3	6.7
	40	1	355.3	15.3	4.3
	40	10	702.1	17.2	2.5
58-34 Medium	4	0.1	1,730.4	161.8	9.4
	4	1	2,263.4	247.8	10.9
	4	10	2,915.1	316.6	10.9
	20	0.1	686.5	56.4	8.2
	20	1	1,031.3	79.4	7.7
	20	10	1,614.1	105.3	6.5
	40	0.01	156.2	30.3	19.4
	40	0.1	217.8	22.6	10.4
	40	1	352.8	15.9	4.5
	40	10	774.1	69.1	8.9
58-34 Coarse	4	0.1	1,529.4	555.9	36.3
	4	1	2,104.4	769.8	36.6
	4	10	2,918.4	1,022.4	35.0
	20	0.1	686.9	153.9	22.4
	20	1	1,069.2	234.8	22.0
	20	10	1,708.1	371.2	21.7
	40	0.01	110.1	12.4	11.3
	40	0.1	162.1	22.5	13.9
	40	1	272.5	41.3	15.2
	40	10	614.0	78.9	12.9

Table C.5 Notch Depth from I-FIT

Mix	Binder PG	RAP Gradation	Notch Depth (mm)				
1	67-22	Fine	15.42	16.08	13.31	15.37	13.50
2		Medium	16.61	14.95	15.34	16.33	14.55
3		Coarse	15.77	14.57	15.70	14.00	15.09
4	58-34	Fine	14.30	15.85	15.70	15.37	14.08
5		Medium	16.00	15.71	14.43	14.37	16.13
6		Coarse	16.10	17.04	13.42	17.04	14.41

Table C.6 Ligament Length from I-FIT

Mix	Binder PG	RAP Gradation	Ligament Length (mm)				
1	67-22	Fine	43.02	42.16	44.69	42.35	43.86
2		Medium	39.92	42.07	41.60	40.61	42.29
3		Coarse	41.24	43.38	41.73	43.59	43.62
4	58-34	Fine	43.69	42.05	42.63	42.59	42.17
5		Medium	41.86	42.63	43.51	44.16	41.69
6		Coarse	41.67	40.04	43.80	39.84	37.61

Table C.7 Thickness from I-FIT

Mix	Binder PG	RAP Gradation	Thickness (mm)				
1	67-22	Fine	49.67	48.24	48.25	51.07	51.02
2		Medium	50.60	50.57	51.01	48.74	49.30
3		Coarse	49.62	49.98	51.68	51.54	49.48
4	58-34	Fine	51.58	51.43	51.53	51.83	52.25
5		Medium	51.75	51.26	51.65	52.39	51.77
6		Coarse	50.80	52.78	53.20	52.78	50.92

Table C.8 Bulk Density of I-FIT Specimens

Mix	Virgin Binder	RAP	Bulk Density (kg/m^3)					Average (kg/m^3)	Std. Dev. (kg/m^3)	COV (%)
1	PG 67-22	Fine	2,075	2,071	2,071	2,078	2,074	2,074	3	0.1
2		Medium	1,975	1,982	1,965	1,982	1,964	1,974	9	0.4
3		Coarse	2,084	2,077	2,074	2,081	2,085	2,080	5	0.2
4	PG 58-34	Fine	2,099	2,076	2,081	2,105	2,102	2,093	13	0.6
5		Medium	2,020	2,027	2,016	1,992	2,023	2,016	14	0.7
6		Coarse	2,150	2,165	2,107	2,129	2,123	2,135	23	1.1