University of Nevada, Reno

Evaluation of Warm Mix Asphalt Technologies and Recycled Asphalt Pavements in Truckee Meadows, Nevada

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ABSTRACT

This study evaluated the properties and laboratory-performance of Hot Mix Asphalt (HMA) and Warm Mix Asphalt (WMA) mixtures with different levels of Recycled Asphalt Pavements (RAP) content: none for control mixtures, around 15% by dry weight of aggregates, and more than 30% by dry weight of aggregates. The rheological properties were evaluated for virgin and recovered RAP asphalt binders. The target amount of RAP in the mixtures was determined by using Blending Charts and Mortar Experiments. The mixtures are design through the guidelines established in Marshall Mix Design Method considering additional modifications for RAP and WMA from Superpave Mix Design. The mixtures are evaluated for their resistance to moisture damage by means of measuring the Dynamic Modulus |E*| after three freeze/thaw cycles and the indirect tensile strength after one and three freeze/thaw cycles. The resistance of the mixtures to permanent deformation was also evaluated by using the Asphalt Mixture Performance Tester (AMPT) to measure the flow number (FN).

For this study, it was determined that the resistance to moisture damage decreases as the number of freeze/thaw cycles increases for most of the evaluated mixtures. Mixtures exhibited an increase in dynamic modulus as the RAP percentage increased. A decrease in the resistance to moisture damage was detected with the increase in RAP content for most of the mixtures. HMA mixtures exhibited a better performance in rutting than the WMA mixtures. An increase in rutting resistance was observed with the increase in RAP percentage for HMA mixtures whereas an inconsistent trend was observed for WMA mixtures. Further study is needed to validate the use of the high percentage of RAP in Washoe County.

DEDICATION

First of all, I dedicate this work to my beautiful wife Gianella and beloved son Tomás. You gave me the strength and motivation to move on when I needed the most. I also dedicate this work to my parents Félix and María, and my brothers Andrés and Nicolás. You were the ones who offered not only support and help, but you have always believed in the success of my various entrepreneurships.

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CHAPTER 1 - INTRODUCTION

1.1 Background

Environmental concerns have become of great interest recently across the various industries. The Pavement Industry has been active in pushing the use of green and environmentally-friendly technologies in road construction, while best optimizing the use of various resources.

The Asphalt Pavement Industry has incorporated two technologies in the mixtures process to improve the environmental aspect of hot-mix production: (1) recycling asphalt pavement (RAP) in order to reduce the use of natural aggregate sources and asphalt binders; and (2) warm-mix technology to reduce the temperatures at which asphalt mixtures are produced and placed on the roads. Both modifications to the traditional mixture productions are more or less recent, and research studies have continuously been aiming on improving mixture designs and performance.

Recycled Asphalt Pavements (RAP) became popular in the United States in the 1970s due to the high cost of crude oil during the Arab oil embargo. Since the cost of the raw materials comprises about 70% of the total cost to produce hot mix asphalt (HMA) [1], the use of RAP materials offered means to reduce the economic impact of the increase in asphalt binder cost. After a Demonstration Project for constructing pavement using RAP, the Federal Highway Administration (FHWA) published "Recycling Materials for Highways" in 1978 and Guidelines for Recycling Pavement Materials in 1980. In early 1990, FHWA and the US Environmental Protection Agency (EPA) estimated that more than 90 million tons of asphalt pavements were reclaimed every year, and over 80 percent of RAP was recycled.

In subsequent years, the use of RAP was usually limited to 15 percent, but the motivation of using greater percentages of RAP showed up after sharp increases in asphalt costs in 2006 and 2008. In addition, the emphasis on green technologies, increase in materials costs, and environmental restrictions led to an increase in the use of RAP as a priority for the asphalt pavement industry.

A Laboratory Evaluation on the Use of RAP in HMA mixtures was performed by Hajj et al. [2]. Three sources of RAP were analyzed and added to HMA mixtures at 15 and 30% with two types of asphalt binders typically used in Reno, Nevada. The mixtures were evaluated based on their resistance to moisture damage, rutting, fatigue and thermal cracking. The researchers found that up to15% RAP can be used with the polymer-modified asphalt binder evaluated without changing the virgin asphalt binder grade. It was also recommended that further research is needed in order to incorporate higher percentages of RAP. A subsequent study at UNR evaluated the addition of WMA technologies in mixtures with 15% RAP and the results were acceptable.

Loria et al [3] evaluated field-produced and laboratory-produced HMA mixtures with three levels of RAP content from filed sections in Manitoba, Canada. The researchers evaluated the applicability of the blending chart process to predict the PG grade of the blended asphalt binder and performed a comparison between the properties and the performance of the field- and laboratory- produced mixtures. They concluded that good correlations were observed between estimated critical temperatures from the blending charts and measured ones form recovered binders, the use of multiple Freeze/thaw cycles provided better characterization of the mixture resistance to moisture damage, higher or similar tensile strengths were observed for the laboratory-produced mixtures when compared to the field-produced mixtures, and field- and laboratory-produced mixtures ranked similarly in resistance to moisture damage, dynamic modulus and thermal cracking resistance at multiple Freeze/Thaw cycles.

War Mix Asphalt (WMA) Technologies were introduced in Europe in 1997 in response to the need of greenhouse gas reduction [4]. Europe subscribed to the protocol of Tokyo in 1997 for the agreement on climate change, and each country was encouraged to reduce greenhouse gas production. Consequently, reducing the fumes and emissions during asphalt production and construction was one of the targets In 2005, the National Asphalt Pavement Association (NAPA) and FHWA formed a WMA Technical Working Group which included, among others, experts from NAPA, State Departments of Transportation (DOTs), FHWA, National Center for Asphalt Technology (NCAT), and American Association of State Highway and Transportation Officials (AASHTO). The purpose of the group was to discuss issues and share knowledge for the advancement of Warm Mix Asphalt in the United States. In 2008, the NCHRP 9-47 Project was initiated to document the engineering properties, emissions, and field performance of WMA. Between 2008 and 2009, there was a significant increase in the number of WMA technologies marketed in the US, and by 2010, over half of the States had specifications permitting the use of WMA.

Prowell et al [5] evaluated three sections of WMA mixes using emulsion process in the NCAT Test Track. Laboratory rutting susceptibility tests conducted in the Asphalt Pavement Analyzer (APA) indicated similar performance for the WMA and HMA surface mixes, but laboratory tests indicated an increased potential for moisture damage with the WMA mixes. The WMA sections showed excellent filed performance in term of rutting.

Bower et al [6] evaluated the performance of HMA and WMA mixes obtained from various field sites in the state of Washington. The resistances of HMA and WMA samples to fatigue and thermal cracking, rutting and moisture sensitivity were conducted in the cores and extracted binders. The authors concluded that the overall short-term performance of WMA pavements is comparable to that of HMA pavements, except that WMA mixes seem to be more resistant to the early stages of reflective cracking than HMA mixes in the field.

Combining RAP and WMA in the same mixture should bring their environmental benefits into the same application. Regarding material characterization, mix design and mixture performance, NCHRP 9-43 Project developed a mix design method for WMA based on Superpave mix design methodology. Furthermore, the study included recommendations for a suite of performance tests to assess whether a WMA mix design would provide satisfactory field service and be applicable to any WMA technology used to lower the mixing and compaction temperatures [7].

Hill et al [8] performed a comprehensive study to characterize a set of WMA mixtures including different WMA additives and RAP contents. The researchers concluded that the introduction of RAP led to the increased resistance to permanent deformation and moisture damage, but RAP reduced thermal cracking resistance

This study evaluates the impact of a higher addition of RAP (more than 30% by asphalt binder replacement) from two different sources in Northern Nevada using three WMA technologies. Laboratory tests were conducted to verify the fulfillment of the Standard Specifications for Public Works Construction of the Regional Transportation Commission (RTC) of Washoe County [9].

1.2 Warm Mix Asphalt Technologies

Nowadays, environmental concerns, economic restraints and safety issues for workers have led to improvements and innovation in construction processes for several industries. The Pavement Industry has followed the new trend by introducing technologies in the fabrication and constructions of roads. Among them, Warm-Mix Asphalt (WMA) represents a group of technologies which allow a reduction in the temperatures at which asphalt mixtures are produced and placed [4]. Conventional hot mix asphalt is typically produced at temperatures ranging from 280 °F to 320 °F, while WMA is commonly produced between 212 °F to 280 °F.

WMA technologies can be classified as those that use water, those that use some form of organic additive or wax, or those that use chemical additives. This study takes into account these three groups by using three technologies: Advera, Evotherm 3G and SonneWarmix (Figure 1).

1.2.1 Water-bearing

The water-bearing technique relies on incorporating water in the hot asphalt binder, increasing the volume and turning it into steam. When the water is dispersed, it results in an expansion of the asphalt binder phase, improving the coating and compactability. Advera is a synthetic zeolite composed of aluminosilicates and alkalimetals, containing approximately 20% crystalized water. After Advera is added to the hot asphalt binder, the zeolite creates a controlled and prolonged foaming, increasing the asphalt binder volume and improving the workability of the mixture.

1.2.2 Chemical Modification

The chemical modification technique involves the use of additives or surfactants to help the asphalt binder to coat the aggregates at a lower temperature, and adding a lubricity effect to improve compactability. Evotherm 3G is a technology developed in the United States, and is a chemistry package designed to enhance coating, adhesion, and workability at reduced temperatures. Evotherm 3G corresponds to the third generation of the chemical additive.

1.2.3 Organic Additives

The Organic Additives show a decrease in the viscosity above the melting point of the wax. Thus, the additive should be selected with a melting point higher than that expected in in-service temperatures to reduce the risk of rutting. SonneWarmix is a paraffinic hydrocarbon blend (wax) with a melting point around 175 °F and it is a liquid between 195 °F and 200 °F.

These three technologies claim to reduce the mixing and compaction temperatures by 50 °F in respect to the regular HMA mixing and compacting temperatures.

1.3 Objectives

The main objective of this study is to conduct a comparison of several WMA technologies added to mixtures containing different percentages of RAP. In particular, the study aims to:

- Conduct a laboratory comparison of various asphalt mixtures including RAP and WMA technologies based on performance test properties meeting RTC specifications.

- Evaluate whether the use of WMA will allow the use of 30% RAP or more without affecting the good performance of the polymer-modified asphalt binder.

CHAPTER 2 - EXPERIMENTAL PLAN

The comparison of the various mixtures was applied to materials available in Truckee Meadows, Nevada. The experimental plan of this study comprises three stages: 1) Materials Characterization, 2) Mixture Designs, and 3) Performance Evaluation of the Mixtures. The experimental plan is schemed in Figure 2, and described as follows:

2.1 Materials Characterization

2.1.1 Aggregates Stockpiles

Several aggregate tests were performed in order to assess the fulfillment of the RTC specifications. Gradations of the various stockpiles were determined according to AASHTO T27 [10] in order to calculate the bin percentages in the final blend of the aggregates. The blend of aggregates should meet the gradation specifications of the Type 2 mixture [9]. Specific Gravities of the various stockpiles were measured according to AASHTO T85 [11] and T84 [12]. Aggregates were treated with hydrated lime at 1.5% (by dry aggregate weight) in order to reduce potential moisture damage in the mixtures following RTC specifications.

2.1.2 Asphalt Binder

The rheological properties of the selected asphalt binder were determined in order to grade the asphalt binder according to AASHTO R29 [13]. In addition, critical temperatures at three temperatures (high, intermediate and low) were determined for further calculations when mixed with the RAP asphalt binder.

2.2 Recycled Asphalt Pavement Material

The recycled Asphalt Pavement (RAP) material was characterized to determine the content and properties of RAP aggregates and the RAP asphalt binder.

Asphalt Binder Content of RAP materials was determined according to AASHTO T164 [14], and the RAP asphalt binder was extracted and recovered according to ASTM5404 [15]. The

Performance Grade of the recovered RAP asphalt binder was determined according to AASHTO R29 [13] and the critical temperatures (high, intermediate and low) were determined accordingly. Conversely, gradation of the recovered RAP aggregates was determined according to AASHTO T30 [16] and T27 [10], and Specific Gravities of the recovered RAP aggregates were determined according to AASHTO T85 [11] and T84 [12].

The amount of RAP material to include in the mixtures was determined by calculating the critical temperatures of the blending between the RAP asphalt binder and the virgin asphalt binder PG64-28NV using Blending Charts method [17] and Mortar Experiment [18]. This study considered the addition of two levels of RAP material: 15%, according to the maximum RAP material to include in mixtures according to RTC specifications, and a higher percentage in the neighborhood of the 30%.

2.2.1 Warm Mix Asphalt Technologies

Among several available Warm Mix Asphalt Technologies, three technologies were selected in the study to fabricate the WMA mixtures and are listed as follows:

- Evotherm 3G
- SonneWarmix
- Advera

The combination of the three levels of RAP material in the mixture (i.e., 0, 15, and 30%), three WMA technologies and the three HMA control mixtures configured twelve mixtures to be evaluated in this study. The nomenclature of the various mixtures is shown in Table 1.

2.3 Mix Design

The various Mix Designs were conducted based on Marshall Mix Design following RTC specifications [19]. Short-term aging at compaction temperature for two hours was applied to the mixtures in loose conditions prior to compaction of the test specimens. The volumetric and

Marshall properties of mixtures were measured to meet the specifications of RTC and comparisons among the various mixtures were performed.

2.4 Performance Tests

Performance Tests were conducted for the various mixtures after different moisture-induced damage cycles. In order to conduct a laboratory-based comparison of the mixtures, Moisture Sensitivity was evaluated at the optimum asphalt binder content on unconditioned samples and on samples conditioned after 1 and 3 Freeze/Thaw cycles according to AAHTO T283 [20]. Dynamic Modulus of the mixtures was measured at different loading frequencies and temperatures on unconditioned samples and on samples conditioned with 3 Freeze/Thaw cycles according to AASHTO TP79 [21] and PP61 [22]. The Resistance to Rutting of the various mixtures was evaluated through Flow Number test on unconditioned samples according to AASHTO TP79 [21]. Finally, the Resistance to Reflective Cracking was intended using the TTI Overlay Tester [23].

CHAPTER 3 - TEST METHODS

The overall objective of this study is to conduct a laboratory evaluation to assess whether the addition of WMA and high percentage of RAP influences the good observed performance for the polymer-modified mixture typically used in Washoe County. Virgin aggregates, virgin asphalt binder and RAP material were characterized, asphalt mix designs were conducted and the performance properties of the various mixtures were measured using materials available in the Truckee Meadows. The various test methods used in this study are explained in this chapter.

3.1 Aggregates Characterization

Aggregates amount is predominant in asphalt mixtures, where the fraction of aggregates is generally 90 to 95 percent by weight. Aggregate is primarily responsible for the load-supporting capacity of the pavement, hence asphalt mixture performance is highly influenced by aggregates structure [7].

In order to obtain a precise characterization of the aggregates, several tests were conducted and they are listed as follows.

3.1.1 Aggregates Gradation

The gradation of the aggregates is known as the distribution of the various particles sizes and can be obtained by combining two methods: washed sieve analysis according to AASHTO T11 [24] and dry sieve analysis according to AASHTO T27 [10]. The procedure involved washing the aggregates to determine the amount of particles smaller than 0.075 µm (sieve No. 200), and after drying, the sample was passed through a series of sieves with progressively smaller sizes. The mass retained in each sieve was measured and compared to the initial mass by computing the percentage of passing mass in each sieve.

3.1.2 Specific Gravity and Absorption of Aggregates

The specific gravity of aggregates is an essential property for a volumetric mix design. It is defined as the ratio between the weight of a unit volume of material and the weight of the same volume of water at approximately 23 ºC. Two methods are defined depending on the size of aggregates; for coarse aggregates AASHTO T85 [11] and for fine aggregates AASHTO T84 [12]. From Both methods the bulk specific gravity, the apparent specific gravity, the bulk specific gravity on the basis of saturated surface-dry aggregates (SSD), and the absorption are determined. They differ in the way to determine the SSD condition of the aggregates: while SSD condition is determined by visual inspection in AASHTO T85, SSD condition is determined by an indirect method in AASHTO T84.

Dukatz et al. [25] conducted a critical review of the various specific gravity measurement methods. Among the findings, inconsistency of the technique used to determine the SSD condition of the aggregates and required soak time prior to the test were detected. These highly influenced the specific gravity results. Since the Asphalt Mix Design methods are based on volumetric properties, specific gravity of the aggregates plays a significant role in the calculation of the asphalt mixture properties.

3.2 Asphalt Binder Characterization

Superpave asphalt binder performance grade (PG) system was developed to characterize asphalt binders for use in HMA pavements. Contrary to the previous asphalt grading systems, the Superpave asphalt binder specifications and tests are focused on pavement performance and assessing asphalt binder contributions to the following major types of distresses: rutting, fatigue cracking, and thermal cracking. According to AASHTO R-29 [13], several testing procedures were conducted and they are listed as follows.

3.2.1 Rutting Assessment

Rutting occurring in the HMA layer is defined as an accumulative permanent strain due to repeated traffic loading and applied shear stresses. The severity of permanent deformation is directly correlated to the in-service temperature of the asphalt pavement. Since rutting is developed during the early and mid-life of an asphalt pavement, original (non-aged) and short-term aged (RTFO) asphalt binders were tested on the Dynamic Shear Rheometer (DSR) at high temperatures. Recent studies have established the work dissipated per loading cycle is inversely proportional to $G^*/\sin(\delta)$ due to rutting mainly which occurs under a stress controlled condition. Hence, the probability of rutting is assessed by limiting the $G^*/\sin(\delta)$ value: the lower the value, the higher the probability of rutting. The PG system has established the minimum $G^*/\sin(\delta)$ of 1.00 kPa for original asphalt binders and 2.20 kPa for short-term aged (RTFO) asphalt binders [26]

3.2.2 Fatigue Cracking Assessment

Fatigue cracking is a type of distress occurring as a result of loading repetitions for the latter part of pavement service life. Since the asphalt binder ages during its service life, it becomes stiffer and tends to become more susceptible to fatigue cracking. Therefore, RTFO and Pressurized Aging Vessel (PAV) are used to age the asphalt binder when evaluating its capacity to avoid the occurrence of fatigue cracking during the later part of the pavement service life [27].

Similarly to rutting studies, other investigations have established the lower energy dissipated per cycle on a strain controlled condition, i.e., the smaller the probability of fatigue cracking. The energy dissipated in this condition is proportional to $G^*\sin(\delta)$ value. Therefore, the PG system has limited $G^*\sin(\delta)$ to 5,000 kPa at the intermediate temperature.

3.2.3 Thermal Cracking Assessment

Thermal cracking may occur due to thermally induced tensile stresses at low temperatures that exceed the tensile strength of the pavement during the in-service life or due to accumulation of permanent tensile strain product of multiple thermal stress cycles that consequently produce shrinkage at a very low rate. Thus, thermal cracking was assessed by determining the flexural creep stiffness of an asphalt binder using the Bending Beam Rheometer (BBR) at low temperature [28]. This method is used to determine how much a long-term aged asphalt binder deflected under a constant load of 100 g at a constant low temperature for 240 sec of loading time. The low temperature was selected according to the average annual lowest temperature of a specific site plus 10 °C due to the time-temperature superposition property of asphalt binders: as the temperature increases, it is possible to slow down the time of load application from 2 hours (time to develop thermal cracking at night) to 240 sec. In other terms, the stiffness at T temperature and during 2 hours load application is equivalent to stiffness at T+10 °C and during 240 seconds load application. The test provides the stiffness in function of time of load application, S(t), and the slope of this curve, m-value. S(t) can be interpreted as a measure of thermal stresses developed in asphalt binder due to creep loading, related to thermal contractions at time t. In the same way, m-value is the rate of how the stiffness changes with loading time: the ability of asphalt binder to relax these thermal stresses (stress relaxation). Therefore, it is necessary to establish a maximum stiffness for not exceeding the strength of the asphalt binder and also a minimum m-value to ensure a stress relaxation that provides a basis to avoid the occurrence of cracking.

For the purposes of this study, RTC specifications were followed for selecting the materials. Two types of PG asphalt binder are permitted for asphalt mixtures: PG64-22 and PG64-28NV. Since the performance of mixtures with polymer-modified asphalt binders is of interest, a PG64-28NV was considered and its requirements are indicated in Table 2.

3.3 Properties of Recycled Asphalt Pavement Materials

Recycled Asphalt Pavements (RAP) Materials are composed of aggregates and asphalt binder. The main difference between RAP materials and a fresh asphalt mixture is the aging of the asphalt binder: while the aging of asphalt binders is more or less known during production of regular asphalt mixtures, the actual rheological properties of the RAP asphalt binders are unknown. Nowadays, increasing the proportion of RAP in asphalt mixtures is a common trend among agencies and measuring the impact of RAP in the final mixture properties becomes a complex process [29].

Evaluating the properties of the RAP involved two processes: a) determining the proportion of asphalt binder to aggregates (asphalt binder content) and b) measuring the properties of both aggregates and recovered asphalt binder. For aggregates, the main properties are gradation and specific gravity. For recovered asphalt binder from RAP, asphalt binder properties were measured through the Performance Grade system. Recent research performed by Loria [30] compared various methods to recover RAP asphalt binder and to characterize RAP aggregates. Recovery methods may have some influence on the properties of the RAP aggregates and the Centrifuge method did not disturb the RAP asphalt binder and aggregates properties in artificially aged RAP material with respect to the control mixture.

3.3.1 Asphalt Binder content of Recycled Asphalt Pavement

Once RAP materials were obtained following AASHTO T2 [31], the stockpiles were homogenized and reduced to obtain a representative sample in order to characterize the RAP material. To assess the variability of the RAP stockpile due to the unknown source of the RAP, several samples were analyzed for asphalt binder content and gradation. Every sample was split following AASHTO T248 [32]: two quarters of the sample were selected to determine the moisture content and the other quarters were used to determine the asphalt binder content in accordance with AASHTO T164 [14].

Extractions were conducted using Toluene-Ethanol solvent in 85/15 proportion by volume, according to the following process: 1) RAP samples were totally immersed in solvent for 20 min for the first soaking; 2) Afterwards solvent was added to the RAP sample in amounts varying from 500 ml to 1,000 ml depending on the total mass of RAP for 20 minutes. After the centrifuge was performed, a second centrifuge was applied twice in order to retain the remaining fine particles greater or equal to 75 µm sieve (No. 200 sieve).

Separating asphalt binder and solvent was the last stage of the recovery process. The recovery of the asphalt binder was conducted following rotary evaporator procedure pointed out in ASTM D5404 [15]. The solution of solvent and asphalt binder was distilled by partially immersing the rotating distillation flask of the rotary evaporator in a heated oil bath, while the solution was subjected to a partial vacuum and a flow of nitrogen gas. After the solvent and asphalt binder were completely split, the recovered asphalt binder was collected for further analysis.

3.3.2 Aggregates properties of Recycled Asphalt Pavement Materials

Once the RAP asphalt binder and RAP aggregates were separated, RAP aggregates were dried up to a constant mass and reduced into two fractions in order to perform two sets of tests: a) One set was subjected to the guidelines stated in AASHTO T30 [16],i.e., extracted aggregates were washed to determine the gradation according to AASHTO T27 [10] and coarse specific gravity according to AASHTO T85 [11], and b) Second set of aggregates (unwashed fraction) were selected to determine fine specific gravities according to AASHTO T84 [12].

3.3.3 Properties of Recycled Asphalt Pavement Asphalt Binder

Determining the properties of the RAP asphalt binder is critical for the mix design process. AASHTO M323 [33] includes recommendations to develop blending charts and determine the physical properties and critical temperatures of the recovered RAP asphalt binder. The procedure to determine the properties of the RAP asphalt binder was similar to the process performed to a virgin asphalt binder (stated in 3.2) with the following modifications: a) rotational viscosity, flash point, and mass loss tests were not required, b) Low critical temperatures were performed using BBR on the RTFO-aged recovered RAP asphalt binder as if it was PAV-aged. Therefore, the high temperature grade was determined on recovered and RTFO-aged RAP asphalt binder, the intermediate temperature and low temperature grades were determined on RTFO-aged RAP asphalt binder. Table 3 summarizes the differences between the two methodologies.

3.4 Blending Charts

When virgin asphalt binder and RAP asphalt binder are blended, determining the properties of the blend is a challenge that several researchers have tried to solve. A full blending of virgin and RAP asphalt binder is assumed to develop methods to estimate the properties of the blended asphalt binder. NCHRP Project 9-12 studied whether the blend asphalt binder acts like a blend between black rock and virgin asphalt binder, the RAP asphalt binder partially blends with the virgin binder or a full blending is produced. Researchers found there is no statistical evidence that RAP material acts like a black rock, but partial blending apparently occurs to a significant extent [34].

An approach to estimate the properties of the asphalt blending is outlined in NCHRP-452 [17]. Under the recommended guidelines of Superpave mixtures including RAP materials, there are three levels of RAP usage: 1) Maximum amount of RAP that can be used without changing the virgin asphalt binder grade, 2) Percentages of RAP that can be used when the virgin grade is decreased by one grade, and 3) For higher RAP contents, it is necessary to extract, recover, and test the RAP asphalt binder and to construct a blending chart. These recommendations are summarized in Table 4.

A blending chart can be constructed by having three of four pieces of information:

a) Desired final asphalt binder grade in the blend.
b) Physical properties and Critical temperatures of the recovered RAP asphalt binder.
c) Physical properties and Critical temperatures of the virgin asphalt binder.
d) Percentage of RAP in the mixture.

Equation 1 was used to determine the critical temperatures (high, Intermediate and low) of the asphalt binder blend:

$$T_B = \%RAP \cdot (T_R - T_V) + T_V \qquad \text{(Eq. 1)}$$

where
T_B: critical temperature of the blended asphalt binder,
%RAP: percentage of RAP expressed as a decimal (i.e., 0.30 for 30 percent),
T_R: critical temperature of the recovered RAP asphalt binder, and
$T_{V:}$ critical temperature of the virgin asphalt binder.

By inspecting Equation 1, each critical temperature can be obtained using a weighted linear combination of the virgin and RAP asphalt binder critical temperatures by percentage of RAP. For example, if %RAP is zero, Equation 1 is equal to the critical temperature of the virgin asphalt binder, while a %RAP equal to 1 (100%) turns Equation 1 equal to critical temperature of the RAP asphalt binder. A %RAP in between delivers a linear combination between the critical temperatures of the virgin asphalt binder and the difference between the critical temperatures of the recovered RAP asphalt binder and the virgin asphalt binder.

Equation 1 can be managed to determine other cases:

a) To determine the virgin asphalt binder critical temperatures blending a known RAP percentage and a known RAP asphalt binder properties, Equation 2 is used:

$$T_V = \frac{T_B - \%RAP \cdot T_R}{1 - \%RAP} \qquad \text{(Eq. 2)}$$

b) To determine the RAP percentage in a blended asphalt binder by combining a virgin asphalt binder and RAP asphalt binder knowing their critical temperatures, Equation 3 is used:

$$\%RAP = \frac{T_B - T_V}{T_R - T_V} \qquad \text{(Eq. 3)}$$

3.5 Mortar Experiment

As mentioned before, Swiertz et al. [18] developed a method to estimate the influence of the RAP binder on the continuous grade profile of virgin asphalt binder. The methodology considers evaluating the Superpave PG properties of mortars to estimate the performance properties of the expected blend of virgin and RAP asphalt binders. An AASHTO draft standard came out from this research and is included in the Appendix of this study [35].

A brief description of the mortar experiment is described below.

3.5.1 Samples preparation

The procedure considered drying and sieving the RAP material in order to collect the fraction between 300 µm (No. 50) and 150 µm (No. 100), denominated R_{100} material. The procedure required at least 500 g of R_{100} material. A minimum of 250 of R_{100} material was used to determine the binder content through the ignition oven, and the burned portion was saved to prepare the mortar samples.

In order to assess the influence of RAP binder in the mortars, two set of mortar types were prepared at different aging levels of the virgin binder (original, RTFO-aged, RTFO+PAV-aged):

a) SRAP Mortar (RAP+binder) consisted of R_{100} material mixed with virgin binder at user-selected level of aging.

b) RRAP Mortar (Burned RAP + binder) consisted of R_{100} burned aggregates mixed with virgin binder at user-selected level of aging and containing total binder content the same as SRAP mortar.

Table 5 summarizes the combination of binder and mortar types at different age-stages needed to perform the Mortar Experiment.

3.5.2 Test Procedure

Virgin asphalt binder was graded according to AASHTO M320 [36]. Mortar tests were conducted as follows:

a) High Temperature Performance: RRAP and SRAP mortar samples were prepared mixing aggregates from RAP and selected RAP, respectively, with original virgin asphalt binder and RTFO-aged virgin asphalt binder. Tests were conducted in the Dynamic Shear Rheometer at a gap of 2 mm at the tests temperatures, and other conditions specified in AASHTO M320 remain unchanged. Tests were conducted at two temperatures: 1) starting at PG High temperature of virgin binder, and 2) PG High + 6 °C.

b) Intermediate Temperature Performance: RRAP and SRAP mortar samples were prepared mixing aggregates from RAP and selected RAP, respectively, with RTFO-aged virgin asphalt binder. Mortar samples were long term aged using the PAV for 24 hours under the conditions specified in AASHTO R-28 [27]. The required amount of mortar was adjusted in order to obtain PAV-age 50 g of binder. After PAV-aged, the mortars were tested in the DSR at the same intermediate temperatures as the virgin asphalt binder.

c) Low Temperature Performance: Mortar samples were prepared the same as Intermediate Temperature Performance samples and poured into BBR molds. Tests were performed at the same temperatures at which the virgin asphalt binder was tested for low temperature performance following the guidelines of AASHTO T313 [28]. The applied loads during the test were adjusted according to the test temperature as shown in Table 6.

3.5.3 Data analysis of Mortar Experiment

The data obtained from the various tests at different temperatures allow for the establishment of a relationship between the asphalt binder replacement percentage and the change in binder continuous grade. The binder replacement percentage is defined as the percentage by weight of recycled asphalt binder to the total weight of asphalt binder used to prepare the mortar samples.

Change in asphalt binder continuous grade is defined as the change in blended asphalt binder continuous grade due to increase in the asphalt binder replacement [°C / %RAP binder replacement].

The sequence of data analysis is described below.

a) The continuous grade of the virgin asphalt binder was determined according to ASTM D7643 [37].

b) The test results for the mortars (RRAP and SRAP) were compared at each testing temperature. Since the difference between RRAP and SRAP is only due to the presence of RAP asphalt binder, the variation in test results can be associated with RAP asphalt binder properties and influence in performance. The difference in performance at a given temperature (δ_{Tx}) is expressed in Equation 4. Values were calculated in logarithmic scale except for m-value.

$$\delta_{\text{Tx}} = \frac{\text{RAP Mortar Property}}{\text{Aggregate Mortar Property}} \qquad \text{(Eq. 4)}$$

c) Since at least two temperatures were tested for every property at a given temperature, the average of the change in performance was calculated and defined as δ_{RAP}.

d) The effect of blending RAP and virgin asphalt binders for every property was computed by multiplying the property of the virgin binder and δ_{RAP}.

e) The influence of blending RAP and virgin asphalt binders can be extrapolated to any binder replacement at a certain temperature (low, intermediate, high). The slope of the extrapolation was based on two points: the first one was the property of the virgin binder, i.e., 0% RAP, and the second one was the estimated property of the blend at the binder replacement percentage in the mortar experiment. Thus, the slope was computed as shown in Equation 5. The extrapolation was applied to high, intermediate and low temperature performance properties.

$$Rate\ of\ Change\ in\ CG = \frac{Est.Blended\ Binder\ CG - Fresh\ Binder\ CG}{Recyvled\ PBR} \qquad \text{(Eq. 5)}$$

where:
Rate of Change in CG: Rate of virgin binder grade change per percent binder replaced, [°C/% replacement]
Est. Blended Binder CG: Estimated blended binder continuous grade, [°C]
Virgin Binder CG: Virgin binder continuous grade, [°C]
Recycled PBR: Percent binder replacement. [%]

3.6 Marshall Mix Design

3.6.1 Historical background

The bases of Marshall Mix Design were introduced by Bruce Marshall in the 1930's and then was improved and standardized by US Army Corps of Engineers. The scope of the original method is limited to hot-mix asphalt containing aggregates with a 25 mm maximum size. In 1990, a new version of the method was proposed in order to address mix designs of mixtures containing aggregates with a maximum size up to 38 mm. The Marshall Mix Design method was intended for laboratory design as well as a field control of asphalt mixtures. Marshall Stability and Flow tests are empirical measures of the mixture properties obtained through this method.

3.6.2 Outline of the method

Several tests were performed in order to assess the fulfillment of the specifications. The outline of the method is described as follows.

a) Verification that all materials met the properties in the specification of the project or agency.

b) Aggregates blend meets the gradation requirement of the project or agency.

Bin percentages were defined for each stockpile in order for the blend gradation to meet the aggregate specifications.

c) Bulk specific gravities of all aggregates used in the blend and the specific gravity of the binder were determined.

As mentioned in section 3.1.2, coarse and fine specific gravities were measured for every aggregate source in the final blend.

d) Maximum Theoretical Specific Gravity (Gmm)

After the final blend of aggregates was determined according to bin percentages, volumetric properties were calculated for the test specimens. Air voids of the test specimens are required, therefore the Maximum Theoretical Specific Gravity of the mixture was calculated according to AASHTO T209 [38].

The Gmm was computed at the expected optimum binder content by determining the ratio of the density of the loose mixture to the density of the water at 25 °C. The volume of the loose sample was determined by applying vacuum to the sample immersed in water, and weighting the water displaced by the sample at 25 °C (Mass determination in water method).

e) Preparation of Test Specimens

A series of test specimens were prepared with different binder content in order to determine thickness and air void of the samples, and measure the Marshall Stability and Flow. A minimum of three replicates per set were prepared for every binder content, and an increment of 0.5% binder content between sets was targeted. Obtaining two sets above and below the optimum binder content was aimed for obtaining well defined curves of volumetric properties versus binder content.

The basis of the procedure is compacting samples using a normalized hammer dropped from a normalized height and by applying a certain numbers of blows per face.

A brief summary of the preparation of Test Specimen is listed as follows:

- Aggregates were dried to constant weight at 105 °C – 110 °C, (221 °F – 230 °F) and aggregates were separated into desired fractions.

- Mixing and Compaction range of temperatures were considered according to the asphalt binder supplier (Paramount Petroleum Inc.) recommendations. Asphalt Binder was heated at the mixing temperature for 2 hours.

- Bowls were cleaned and heated in the oven at the mixing temperatures. Each specimen was prepared considering an amount of aggregates that resulted in a compacted specimen of 2.50 ± 0.05 inches in height. In general, this amount was about 1,180 g but it varied according to the asphalt binder content of the test specimen. After the aggregates were poured into the bowl, a crater was formed in the center of the aggregates. Hot asphalt binder was poured into the crater, and aggregates and asphalt binder were thoroughly mixed within the limits established for the Mixing Temperature range of the asphalt binder. After the mixing was completed, the sample was poured from the bowls into pans.

- Test specimens were subjected to short-term aging. The short term aging consisted of aging the specimens in the pan at the expected compacting temperature for 2 hours.

- Mold set (collar, mold, and plate) and hammer were carefully cleaned and heated between 95 °C and 150 °C (200 °F and 300 °F).

- Mold sets were assembled and a paper filter was placed over the plate (bottom of the mold). The sample was poured into the mold set and the mixture was spaded 10 times in the center and 15 times in the edge with a hot spatula. The surface of the samples was smoothed and a paper filter was placed on top of the sample. The temperature of the sample should be within the limits of the Compacting Temperature range.

- Sample in the mold was placed and held in the pedestal of compaction hammer. 75 blows per face were applied to the sample from a free fall of 18 inches. After the blows were applied, the collar was removed, mold was reversed and the same amount of blows was applied on the other face of the sample. The compaction lasted 90 sec at maximum per face. After the compaction was finished, paper filters were removed and the sample was extruded from the mold. The specimen was cooled in air.

f) Test Procedure

Thickness and bulk specific gravity in the test specimens were determined according to ASTM D3549 [39] and AASHTO T166 [40], respectively. They were measured right after the test specimens were cooled down.

Marshall Stability and Flow were determined for each test specimen of each set. The procedure is briefly described as follows:

- Testing heads are cleaned and guide rods are lubricated to allow a fluid displacement of the upper head without bending. Testing heads and guide rods were maintained between 21.1 °C and 37.8 °C (70 °F to 100 °F).

- The sample specimen was immersed in a hot bath at 60 °C for 30 to 40 min, removed from the water bath and its surface was dried.

- The Sample was placed in the lower head testing, centered and the upper testing head was placed on top of the sample, fitting the upper testing head to the guide rods.

- Ram of load cell was adjusted to apply a low pressure contact to the upper head, and load readings and displacements were set to zero.

- Load was applied at a constant rate of 51 mm (2 inches) per minute. The Marshall Stability is defined as the maximum load obtained during the test and Flow is defined as the displacement measured since the beginning of the test up to when Marshall Stability was reached (as seen in Figure 4).

- After the Marshall Stability and Flow are recorded, the sample was removed from the testing heads.

- The test lasted less than 30 sec between from when the sample was removed from the hot water bath, and Marshall Stability was reached.

3.6.3 RAP in Mix Design

Traditional Mix Designs are based on the blend of asphalt binder and virgin aggregates. Adding RAP materials in the Mix Designs involves blending RAP aggregates and RAP asphalt binder, and virgin materials (asphalt binder and aggregates) as shown in the phase diagram in Figure 3 . The mode to quantify RAP contribution varies depending on the considered reference: total weight of mixture, total weight of aggregates, etc. Hence, RAP contribution can be expressed by RAP aggregate percentage, percentage of RAP material in total mixture or by RAP asphalt binder replacement at the optimum binder content. In the first approach, the RAP contribution is computed by considering the bin percentage of the RAP aggregate in the total mix the same as the virgin aggregates, regardless of the optimum binder content or the final mix design. The second approach considers that once the RAP aggregate bin percentage was fixed and the optimum binder content was computed, the percentage of RAP material in the mixture is determined related to the total mixture. A third approach relies on the amount of virgin asphalt binder that is replaced by the asphalt binder of the RAP material. If the percentage of RAP aggregates in blend aggregates is fixed, then the contribution of RAP binder in the final mixture is known. As mentioned before, Marshall Mix Design was performed by varying the asphalt binder content for each specimen set. If the binder content increases, the ratio of RAP binder to the total binder content decreases. This ratio is defined as Binder Replacement and it can be calculated for each mixture at the optimum binder content.

3.6.4 Analysis of Test Data

Selecting the adequate asphalt binder content was guided by applying limit criteria in mixture test data.

In general, the following relationships are analyzed and considered for defining the final mixture:

- Density vs. Binder Content
- Air Voids vs. Binder Content
- Stability vs. Binder Content
- Flow vs. Binder Content
- Voids Filled with Asphalt (VFA) vs. Binder Content
- Voids in the Mineral Aggregates (VMA) vs. Binder Content

As first approach, the optimum binder content was selected for a target air void of 4.0%, which is the middle range of air voids established in MS-2 [19]. However, the tentative binder content shall meet the remaining specifications established by either the agency or MS-2. The additional limits were minimum Marshall Stability, range of Marshall Flow, minimum VMA, and a range for VFA. Hence, the optimum binder content of the various mixtures was selected considering these requirements and engineering judgment.

3.7 Performance Properties of Asphalt Mixtures

3.7.1 Moisture Sensitivity

Stripping, or loss of bond, is mainly caused by the presence of moisture between the asphalt and the aggregate. Many factors may contribute to stripping, such as wrong selection of materials, inadequate mix design, deficient construction practices and pavement design. The moisture sensibility test used to evaluate HMA for stripping was conducted according to AASHTO T283 [20].

Samples were mixed at optimum binder content and aging was applied to the specimens in loose condition at 60 °C for 16 hours. Specimens were compacted at 7.0±0.5% of air voids using the Marshall Compactor in order to maintain consistency with the Marshall Mix Design Method. After specimens were prepared and air voids were determined, subsets of three specimens each were selected according to their similar air void criteria. Two subsets were conditioned to one or three freeze/thaw cycles. A freeze/thaw cycle consisted of: 1) Samples were saturated to a level between 70 and 80%, 2) Samples were frozen at -18 °C for 16 hours minimum, 3) Samples were immersed in a water bath at 60 °C for 24 hours, and 4) Samples were conditioned at 25 °C in a water bath prior to testing or being subjected to a new conditioning cycle. The other subset (dry subset) was maintained in the laboratory at ambient temperature.

Test specimens were evaluated by determining the indirect tensile strength of the samples. Dry subset was evaluated by applying a constant load of 2 inches per minute and recording the maximum load at a laboratory temperature. Similarly, the conditioned subsets were removed from the 25 °C and immediately tested to obtain the maximum load at laboratory temperature. The tensile strength for each specimen was determined using Equation 6:

$$TS = \frac{2 \cdot P}{\pi \cdot t \cdot d} \qquad \text{(Eq. 6)}$$

where,
TS: tensile strength of specimen, psi
P: maximum load recorded during the test, lb
t: thickness of the specimen, inches
d: diameter of the specimen, inches

The average of the tensile strength of the both unconditioned and conditioned subsets was calculated. The ratios of the conditioned subsets to the unconditioned subsets were calculated and defined as Tensile Strength Ratio (TSR). Superpave requires a minimum TSR of 0.80. Lower values may indicate mixtures exhibiting stripping problems after construction [7].

3.7.2 Dynamic Modulus |E*|

Several factors influence the pavement response along the lifespan of an asphalt pavement, such as temperature variation, aging of the asphalt binder, traffic, etc. The AASHTO Mechanistic-Empirical Pavement Design Guide (MEPDG) includes the various influences in the pavement response models by considering the effect of temperature and vehicle speed variations on the stiffness of asphalt layer. The characterization of the stiffness of the asphalt layer can be assessed by determining the Dynamic Modulus |E*| of the asphalt mixtures. Dynamic Modulus |E*| is defined as the absolute value of the complex modulus calculated by dividing the peak-to-peak stress by the peak-to-peak strain for a material subjected to a sinusoidal loading.

The test was conducted according to AASHTO TP79 [21] and consisted of applying a controlled sinusoidal compressive stress of various frequencies at a specific temperature. The applied stresses and resulting axial strains were measured as a function of time and used to calculate the Dynamic Modulus and phase angle.

Test specimens were fabricated in the Superpave Gyratory Compactor according to AASHTO PP60 [41]. The loose mixtures were subjected to short term aging of 2 h at the compaction temperature and specimens were compacted in order to obtain cylindrical samples of 150 mm diameter by 170 mm tall (minimum). The height of the compacted cylinders is selected to 175 mm, since the same procedure was considered to fabricate the Flow Number specimens (defined later in 3.7.3). E* samples were obtained by coring and sawing the compacted samples after the compacted samples were cooled down. E* samples consisted of cylindrical specimens 100 mm diameter by 150 mm tall. Bulk specific gravity (Gmb), diameter and thickness of the E* samples were determined. Compacted cylinders were fabricated by adjusting the mass of mixture to compact in order to obtain E* samples at 7.0±0.5 % air voids after sawing the cylinders.

This research also considered determining the E* of samples subjected to three Freeze/Thaw cycles. The conditioning cycles of E* samples were performed the same as the samples used to determine the Moisture Sensitivity of the mixtures (summarized in 3.7.1).

The Dynamic Modulus tests were performed at three temperatures (4 °C, 20 °C and 40 °C) applying sinusoidal stresses at three frequencies at low and intermediate temperatures (10 Hz, 1 Hz, and 0.1 Hz), and four frequencies at high temperature (10 Hz, 1 Hz, 0.1 Hz and 0.01 Hz). The tests were performed for low to high temperatures, applying high frequency to low frequency load for each temperature test. Typically, the Dynamic Modulus of a mixture was calculated considering at least two specimens.

The time-dependency behavior of asphalt mixtures was taken into account in the development of the Dynamic Master Curve, constructed at a reference temperature (70 °F) by shifting Dynamic Modulus data from various temperatures. In other words, collected Dynamic Modulus data was manipulated to obtain a continuous function describing the Dynamic Modulus as a function of frequency and temperature.

The general form of the Dynamic Modulus master curve is a modified version of the Dynamic Modulus master curve equation included in the MEPDG and it is shown in Equation 7:

$$log|E^*| = \delta + \frac{(Max-\delta)}{1+e^{\beta+\gamma\cdot log(fr)}} \quad \text{(Eq. 7)}$$

where
|E*|: dynamic modulus, psi,
δ, β and γ: fitting parameters,
Max: the limiting maximum modulus, psi, and
f_r: the reduced frequency, Hz.

The reduced frequency was computed using the Arrhenius equation and it is shown in Equation 8:

$$log(fr) = log(f) + \frac{\Delta Ea}{19.14714} \cdot \left(\frac{1}{T} - \frac{1}{Tr}\right) \quad \text{(Eq. 8)}$$

where
fr: the reduced frequency at the reference temperature, Hz,
f: the loading frequency at the test temperature, Hz,
ΔEa: the activation energy (treated as a fitting parameter),
T: the test temperature, °K, and
Tr: the test temperature, °K.

The final form of the dynamic master curve was obtained by substituting Equation 8 into Equation 7, as shown in Equation 9:

$$log|E^*| = \delta + \frac{(Max - \delta)}{1 + e^{\beta + \gamma \cdot \left(log(f) + \frac{\Delta Ea}{19.14714} \cdot \left(\frac{1}{T} - \frac{1}{Tr}\right)\right)}} \quad \text{(Eq. 9)}$$

The shift factors at each temperature are given in Equation 10:

$$log\big(a(T)\big) = \frac{\Delta Ea}{19.14714} \cdot \left(\frac{1}{T} - \frac{1}{Tr}\right) \quad \text{(Eq. 10)}$$

where a(T) is the shift factor at temperature T.

On the other hand, the maximum limiting modulus was estimated from HMA volumetric properties using the Hirsch model and limiting binder modulus of 1 GPa as seen in Equations 11 and 12:

$$|E^*|max = Pc \cdot \left(4{,}200{,}000 \left(1 - \frac{VMA}{100}\right) + 435{,}000 \left(\frac{VFA \cdot VMA}{10{,}000}\right) + \frac{1 - Pc}{\frac{\left(1 - \frac{VMA}{100}\right)}{4{,}200{,}000} + \frac{VMA}{435{,}000 \cdot VFA}} \right) \quad \text{(Eq. 11)}$$

where

$$Pc = \frac{\left(20 + \frac{435{,}000 \cdot VFA}{VMA}\right)^{0.58}}{650 + \left(\frac{435{,}000 \cdot VFA}{VMA}\right)^{0.58}} \quad \text{(Eq. 12)}$$

|E*|max: limiting maximum HMA dynamic modulus, psi,
VMA: voids in the mineral aggregate, %,

VFA: voids filled with asphalt, %, and
Max parameter is calculated by applying log to |E*|max.

The fitting parameters of the Equation 9 were calculated by numerical optimization using Solver tool in Microsoft Excel. The goodness of fit s met requirements established in AASHTO PP61 [22].

3.7.3 Flow Number

Determining or estimating the potential of permanent deformation in asphalt mixtures is a challenging task for researchers and engineers nowadays. The MEPDG takes into account distresses predictions for rutting, fatigue cracking, thermal cracking and International Roughness Index (IRI). Rutting describes any distortion in the asphalt layer as a result of accumulated traffic loading. A rut is a surface depression in the wheel paths and pavement uplift might occur along the sides of the rut. Rutting can be caused by plastic movement of the asphalt mix either in hot weather or from inadequate compaction during construction [42], [43].

The Flow Number (FN) test was intended to predict the resistance to permanent deformation of asphalt mixtures. Mainly, an asphalt specimen was subjected to a repeated haversine axial compressive pulse of 0.1 s every 1.0 s without confining pressure. The resulting permanent axial strains were measured as a function of the load cycles and numerically differentiated to calculate the flow number. For the purposes of this study, the test was conducted according to AASHTO TP79 [21].

FN samples were fabricated according to AASHTO PP60 [41]. Loose mixtures were short-term aged for 2 hours at compaction temperature, and then compacted in the Gyratory Superpave Compactor at a target air void of 7.0±0.5%. The compacted specimen consisted of a cylinder of 150 mm diameter by 175 mm tall. After the sample was cooled down, it was cored and sawed in order to obtain the test sample of 100 mm by 150 mm tall. Three samples were fabricated to obtain the FN of a mixture.

The FN Test Temperature was chosen according to the Appendix X2 of AASHTO R35 [44]. The FN Test was conducted at the design temperature at 50% reliability as determined using LTPP Bind Version 3.1 [45] and the temperature was computed at 20 mm. Prior to the FN number test was conducted, samples were conditioned for 3 h in an oven at the Flow Number Temperature Test.

As the test was performed, three stages can be identified when permanent strain vs. load repetition is plotted [46]:

a) Primary Stage: High initial level of rutting, with a decreasing rate of plastic deformations, predominantly associated with volumetric change,

b) Secondary Stage: Small rate of rutting exhibiting a constant rate of rutting change that is also associated with volumetric changes. However, shear deformations increase at an increasing rate, and

c) Tertiary Stage: High level of rutting predominantly associated with plastic (shear) deformation under no volume change condition.

The point at which the tertiary flow starts is called the Flow Number as seen in Figure 5.

Various methods have been developed for mathematically determining the FN. Some of them are the Three-Stage Method, the Stepwise Increase Approach and the Francken Method. This study considered the Francken Method to determine the FN of the various mixtures.

The Francken method was developed based on triaxial repeated load tests under various temperatures and stress levels and it is a combination of a power law function and an exponential function [47]. The model was obtained through a complex regression mathematical model as shown in Equation 13.

$$\varepsilon p(N) = A \cdot N^B + C \cdot (e^{D \cdot N} - 1) \qquad \text{(Eq. 13)}$$

where:

εp: plastic/permanent axial strain,
N: the number of loading cycles, and
A, B, C, D: the regression constants.

Strain rate can be obtained by computing the derivate of Equation 13 with respect to N as shown in Equation 14:

$$\frac{\partial\ \varepsilon p(N)}{\partial N} = A \cdot B \cdot N^{B-1} + C \cdot D \cdot e^{D \cdot N} \quad \text{(Eq. 14)}$$

The cycle number associated with the Flow Number was computed at the point where the rate of change of slope changes sign (goes from negative to positive. This point indicated the inflection point in the permanent strain versus number of cycle curve as seen in Figure 5. The second derivative of the Francken model was evaluated at each cycle to obtain the rate of change using Equation 15.

$$\frac{\delta^2 \varepsilon p(N)}{\delta N^2} = A \cdot B \cdot (B-1) N^{(B-2)} + C \cdot D^2 \cdot e^{DN} \quad \text{(Eq. 15)}$$

The number of cycle at which Equation 15 changes of sing is defined as Flow Number. The Flow Number of each specimen was computed using Solver in Microsoft Excel.

3.7.4 Resistance to Reflective Cracking

Reflective Cracking consists of the occurrence of cracks in the new asphalt overlays showing the same pattern of the joints and cracks that already existed in the underlying pavement. This type of distress includes cracks that propagate through the asphalt layer of an overlaid Portland Cement Concrete pavement at its joints or cracks that propagate through an overlay over a deteriorated asphalt surface [7]. Typically, the distress is initiated at the bottom of the new overlay and propagated through the thickness of this overlay.

The Texas Transportation Institute (TTI) Overlay Tester (OT) was developed to characterize the mixtures resistance to reflective cracking by subjecting a sample to repeated opening and closing movements. The test was specifically designed to simulate the horizontal opening and closing of joints and cracks that exist underneath a new HMA overlay and initiate the cracks [23]

The OT is a fixed displacement test and the percentage drop in applied load was used to define specimen failure. Based on extensive OT testing, the researchers recommend that failure is defined as the cycle number where the load to open the sample is less or equal to 7 percent of the load measured in the first cycle (93 percent load reduction).

To fabricate the test samples, the loose mixture was subjected to short-term aging at a compaction temperature for 2 hours. The mixture was compacted using the Superpave Gyratory Compactor in order to obtain a cylindrical sample of 150 mm diameter by 170 mm tall. The OT test specimens were obtained by sawing the middle section of the cylindrical sample. After cutting, the dimensions of the test specimens are 150 mm in length (6 inches), 75 mm wide (3 inches) and 38 mm thick (1.5 in) with a target air voids of 7.0±1.0%. Figure 6 shows the process of obtaining the TTI Overlay Tester sample. Since the samples received water during the sawing process, they were dried using fans for two days.

The test specimen was glued on two metallic plates. The glue was spread all over the plates and a 0.25 in width tape is placed over the gap between the plates. A 4.5 kg (10 lb) weight was placed over the sample to ensure full contact between the test specimen and the metallic plates. This study considered a minimum setting time of 12 hours to allow the glue to obtain its maximum strength. The rest specimen was set inside the OT chamber for a minimum of 2 hours to thermal condition. The recommended temperature by TTI for this test is 77°F (25°C), applying cycles of loading rate of 10 seconds (5 s loading and 5 s unloading) with an opening of 0.025 inches until completing 1,200 cycles if the test sample does not reach failure criteria. However, a research performed at UNR determined that a test temperature of 50°F (10°C) with an opening of 0.018 inches is more appropriate for the type of mixtures and asphalt binder grades used in northern

Nevada. The test is also run until 4,000 cycles if failure does not occur [48] . After the test was finished, the number of cycles until failure occurs was recorded.

3.8 Warm Mix Asphalt Modifications to Mix Design and Performance Tests

After NCHRP 09-43 [49], AASHTO R35 [44] established considerations and practices for Warm Mix Asphalt (WMA) mix designs. Since MS2 does not include references to prepare and include WMA technologies into the Marshall Mix Design, the Superpave approach was used in this study to assess these new technologies.

Regarding Technology-Specific Fabrication Procedures, the recommendations are focused on aggregates, addition of WMA technologies to aggregates, binders and mixtures. The recommendations applied in this study are listed as follows:

3.8.1 Batching aggregates and RAP fractions

Aggregates were heated in an oven at approximately 15 °C higher than the planned production temperature at minimum 2 hours prior to mixing. The RAP material was heated in separated pans but the heating time of RAP was limited to 2 hours.

3.8.2 Preparation of WMA specimens with WMA Additive added to Binder

Evotherm 3G and SonneWarmix are two different types of warm-mix additives added to the asphalt binder [4]. *Evotherm 3G:* Asphalt binder was heated at 135 °C until the binder was sufficiently fluid to pour. Evotherm 3G was added to the asphalt binder prior mixing at a rate of 0.6% by total weight of asphalt binder in the mixture, including RAP asphalt binder. Asphalt binder and Evotherm 3G was stirred until the additive was totally dispersed in the binder. Finally, the WMA modified binder was heated until mixing temperature and mixing proceeded as a regular HMA mixture.

SonneWarmix: The asphalt binder was heated at 135 °C until the binder was sufficiently fluid to pour. SonneWarmix was added to the asphalt binder prior to mixing at a rate of 1.0% by total weight of binder in the mixture, including RAP asphalt binder. Asphalt binder and SonneWarmix were stirred until the additive was totally dispersed in the binder. Finally, WMA modified binder was heated until mixing temperature and mixing proceeded as a regular HMA mixture.

3.8.3 Preparation of WMA specimens with WMA Additive added to the Mixture

Advera qualifies as a WMA additive added to the mixture [4]. Asphalt binder, the RAP material and aggregates were heated as mentioned before. Aggregates and RAP fractions were poured into the bowl; a crater was created in the middle of the aggregates. Advera was added to the pool of binder after poured on the crater formed in the aggregates at a rate of 0.25% by weight of total mixture. Finally, mixing proceeded as a regular HMA mixture.

3.8.4 Mixture Properties

AASHTO R35 Appendix establishes recommendations for determining the mixture properties of WMA mixtures the same as HMA mixtures using the Superpave Mix Design Method. However, properties of WMA mixtures of this study were determined according to MS-2 Marshall Mix Design Method [19]. The procedure was described above (3.6).

3.8.5 WMA Mixtures Evaluation

After the optimum binder content was determined, WMA mixtures were evaluated for coating, moisture sensitivity and rutting resistance. Compactability testing could not be performed due to issues associated with determining the Ndesign parameter for the mixtures. Compactability test is consistent with Superpave Mix Design Method, but Marshall Mix Design Method was conducted for the various mixtures. An appropriate Ndesign parameter was intended to be determined for HMA mixtures but the results were not satisfactory when related to expected traffic level.

The procedures of WMA Mixture Evaluations are described as follows:

a) Coating

The coating level of aggregates by the asphalt binder was measured according to AASHTO T195 [50]. Samples were prepared at optimum binder content and not short-term aged. The recommended coating criterion is minimum 95% of the coarse aggregate particles being fully coated.

b) Compactability

Even though the Compactability of mixtures was not performed, the procedure is described as follows:

Compactability evaluation is used to determine the appropriate compaction temperature of the WMA technology after research performed in NCHRP 9-43 [49]. The temperature sensitivity of the WMA mixtures is back calculated by comparing ratio of two numbers of gyrations to obtain 92% of Gmm obtained after compacting at two different temperatures. The procedure is described as follows:

- WMA samples are prepared as mentioned before and short-term aged by 2 hours at the planned compaction temperature.
- Duplicate specimens are compacted using the Superpave Gyratory Compactor at the planned compaction temperature to N_{design} gyrations according to AASHTO T312 [51]

and the specimen height of each gyration is recorded. The bulk specific gravities are determined for each specimen according to AASHTO T166 [40].

- The corrected specimen relative densities for each gyration of the specimens are determined according to Equation 16:

$$\%Gmm_N = 100 \cdot \left(\frac{Gmb \cdot h_d}{Gmm \cdot h_N}\right) \quad \text{(Eq. 16)}$$

where:
$\%Gmm_N$: relative density at N gyrations,
Gmb: bulk specific gravity of the specimen compacted to N_{design} gyrations,
h_d: height of the specimen after N_{design} gyrations, mm
h_N: height of the specimen after N gyrations, mm.

- The average number of gyrations needed to reach 92% of relative density is determined form the compaction of both specimens.

- The process is repeated, but compacting at 30 °C below the planned field compaction temperature.

- The gyration ratio is determined using Equation 17:

$$Ratio = \frac{(N_{92})_{T-30}}{(N_{92})_T} \quad \text{(Eq. 17)}$$

where
Ratio: gyration ratio
$(N_{92})_{T-30}$: gyration needed to reach 92% relative density at 30 °C below the planned field compaction temperature,
$(N_{92})_T$: gyration needed to reach 92% relative density at the planned field compaction temperature.

- The recommended compactability criterion is a gyration ratio less or equal to 1.25. Thus, the compaction temperature is adjusted as low as possible to meet the criterion.

c) Moisture Sensitivity

The Moisture Sensitive was evaluated according to AASHTO T283 [20] and was described in 3.7.1. The recommended moisture sensitivity criteria were a tensile strength ratio greater than 0.80 and no visual evidence of stripping.

d) Rutting Resistance

Rutting resistance was evaluated according to AASHTO TP79 [22]. Some modifications to the AASHTO TP79 were introduced to characterize the Rutting resistance of WMA mixtures and are listed as follows:

- Three specimens were mixed as described before. Specimens were short-term aged at the compaction temperature for 2 hours.
- Specimens are compacted to a target of 7.0 ± 1.0 % air voids using the Superpave Gyratory Compactor.
- Test was conducted on 100 mm diameter by 150 mm high test specimens that were sawed and cored from larger Gyratory specimen 150 mm diameter by at least 175 mm high.
- Test was conducted at the design temperature at 50% reliability as determined using LTPP Bind Version 3.1 [45]. Test temperature was computed at 20 mm (surface courses).
- Test was performed unconfined using a repeated deviatoric stress of 600 kPa (87.02 psi) with a contact deviatoric stress of 30 kPa (4.35 psi).

The flow number for each specimen and the average were computed. The results were compared with the criteria shown Table 7.

CHAPTER 4 - TEST RESULTS AND ANALYSIS

In order to fulfill the objectives of this study, this chapter describes the results of the characterization of virgin aggregates, virgin asphalt binder and RAP material, the various asphalt mix designs and the performance properties of the various mixtures.

4.1 Aggregates Characterization

The aggregates considered for this study were supplied by Western Nevada Materials, Reno. The pit is located in Spanish Springs, Sparks, and five stockpiles were obtained in July 2012. Gradation and Specific Gravity tests were performed for every stockpile and the results are shown in Table 8.

The various stockpiles were combined in order to get a blend aggregate that meets the RTC Type 2 [9] shown in Table 9. As a starting point, a blend of 20% of ¾ inches coarse aggregates, 25% of ½ inch coarse aggregates, 5% of Crushed Sands, 30% of Impact Sand and 20% of Wade Sand was selected for the control mixture and it is equivalent to an actual mixture used in some contracts in Washoe County. Figure 7 shows the blend of the various stockpiles meeting the RTC Type 2 specifications. As shown in Table 8 , the difference in bulk specific gravity among various stockpiles was less than 0.2; hence a correction of the blend gradation by volume was not needed.

4.2 Asphalt Binder Performance Grade

The performance grade of the asphalt binder was determined according to AASHTO M320 [36] and the results are shown in Table 10. The asphalt binder was graded as PG 64-28. The continuous grade of the virgin asphalt binder was PG 68.6-32.5 which will be taken into consideration in subsequent analysis. The PG plus compliance of the asphalt binder was not performed in this study.

4.3 Properties of Recycled Asphalt Pavement Material

The RAP material which was stored in sealed barrels had 1.0 % moisture content. The presence of moisture was taken into account when calculating the asphalt binder content and alerted to dry the samples at 60 °C for 24 hours prior samples preparation.

The average RAP binder content was 4.24 % (by TWM) and the grade of the RAP binder was determined to be PG 82-16. The continuous grade for the RAP binder was PG 87.9 – 20.0. The RAP asphalt binder PG grade is summarized in Table 11.

Recovered RAP aggregates were evaluated and the results are shown in Table 12. The nominal maximum size of the aggregates was 9.5 mm (3/8" sieve), with a large amount of fine particles. A high absorption for the RAP aggregates was calculated.

4.4 Blending Charts

Since PG grade of both virgin and RAP asphalt binders were determined, a blending chart for high, intermediate and low critical temperatures was constructed for several binder replacements using Equation 1. The blending Chart is shown in Figure 8.

Two binder replacements were remarked in Figure 8: 15% and 35%. Both percentages indicated a change of one degree in the high PG grade of the blended binder but allowed keeping the same low PG grade as the virgin asphalt binder. Hence, no changes in the low PG grade of the blended binder were observed, replacing up to 35% of virgin asphalt binder with RAP asphalt binder.

4.5 Mortar Experiment

Mortar experiments were conducted to estimate the maximum amount of RAP in the final mixture without changing the virgin binder PG Grade. For the purpose of this study, two mortar experiments were conducted at different binder replacement percentages.

First of all, the binder content of SRAP was determined using both the ignition oven and centrifuge method. Determination of binder content using ignition oven may cause biased results because some aggregates may lose mass on ignition. Correction factors were not taken into account in the calculation to address this matter since the RAP source was undetermined. Thus, both ignition oven and centrifuge methods were conducted to determine the binder content of SRAP samples. Both results were practically equivalent, therefore a RAP asphalt binder content of SRAP of 7.05% (by TWM) was considered for the mortar experiment. The results of both tests are summarized in Table 13.

4.5.1 Mortar Experiment No. 1

The first mortar experiment was conducted at a binder content of 50%. A minimum of 30% by weight is recommended to ensure workability of mortars and achieve DSR and BBR samples free of voids. Targeting 50% of total asphalt binder content in the mortar samples implied a binder replacement amount of 7.58%.

A spreadsheet was developed to process the data collected from the mortar experiment. Results are shown in Table 14 (Low Temperature Testing Results), Table 15 (Intermediate Temperature Testing Results), and Table 16 (High Temperature Testing Results). The major outcome of this experiment was the Rate of Change in Continuous Rate for every critical temperature: 0.294 °C/%PBR for Low Temperature, 0.151 °C/%PBR for Intermediate Temperature and 0.221 °C/%PBR for High Temperature.

Figure 9 summarizes the Mortar Experiment 1. The interpolation for blending binders between 0 and 7.58% binder replacement were plotted in continuous lines for high, intermediate and low temperatures according to Equations 4 and 5, defined in 3.5.3. The extrapolation for blending binders between 7.58% and more binder replacement were also plotted for the various critical temperatures in dash lines. Two binder replacements were plotted: 15% and 35% binder replacement. According to mortar experiment 1, there was a change in the high PG temperature in one grade but the low PG temperature remained the same as the virgin asphalt binder (PG64-28). Therefore, a greater binder replacement implied a change of grade in both high and low temperature, as shown for example in a 35% binder replacement. If the binder replacement was 100%, the expected blend of RAP binder and virgin asphalt binder would match with the PG grade of the RAP binder. However, the PG blend extrapolation was similar to the RAP binder PG grade at high temperature, but much harder at low temperature.

4.5.2 Mortar Experiment No. 2

In order to check the results of the first mortar experiment, a second mortar experiment was conducted at a different binder replacement. Mortar Experiment 2 was performed at a lower total asphalt binder content to assess the properties of the mortar at a higher binder replacement.

Results are shown in Table 17 (Low Temperature Testing Results), Table 18 (Intermediate Temperature Testing Results), and Table 19 (High Temperature Testing Results. The Rate of Change in Continuous Grade for every critical temperature was 0.228 °C/%PBR for Low Temperature, 0.302 °C/%PBR for Intermediate Temperature and 0.165 °C/%PBR for High Temperature.

Figure 10 summarizes the Mortar Experiment 2. The interpolation for blending binders between 0 and 15% binder replacement were plotted in continuous lines for high, intermediate and low temperatures according to Equations 4 and 5 defined in 3.5.3. The extrapolation for blending binders between 15% and more binder replacement were also plotted for the various critical temperatures in dashed lines. Two binder replacements were plotted: 15% and 35% binder replacement. According to the results of the second mortar experiment, there was also a change in the high PG temperature in one grade but the low PG temperature remains the same as the virgin asphalt binder (PG64-28) at 15% binder replacement. Therefore, a greater binder replacement implied a change in grade in both high and low temperature, as shown for example at 35% binder replacement. If the binder replacement was 100%, the expected blend of RAP binder and virgin asphalt binder would match with the PG grade of the RAP binder. However, the PG blend extrapolation was similar to the RAP binder PG grade at high temperature but much harder at low temperature.

4.5.3 Comparison between Mortar Experiments No. 1 and No. 2

A comparison between the Rate of Change in Continuous Grade for every critical temperature was performed. Assuming that a 100% binder replacement should end up with the critical temperatures of the RAP binder PG grade (87.9, 27.9, and 20.0 °C), the following conclusions can be made:

- Both experiments indicated a binder replacement up to 15% with no change in the low PG grade temperature, while a change in the high PG grade temperature was observed in both experiments.

- The high PG grade of the RAP binder was well predicted in both experiments, with a deviation of +/- 2.8 °C.

- Intermediate critical temperature for RAP binder was well predicted in Mortar Experiment 1 (lower binder replacement experiment), but poorly predicted in Mortar Experiment 2 (higher binder replacement experiment).

- The low PG grade of the RAP binder was not well predicted in both experiments. The outcomes were higher than the measured RAP binder low PG grade. The offset was more than 10°C in both temperatures; hence the prediction of the low PG grade for the blend at any binder replacement would not be accurate. Data obtained in this study did not permit establishing the reason of these offset; further research would be needed to validate the linearity of the relationship of extrapolating critical temperatures of the asphalt blend

Figure 11 shows the Mortar Experiment 1 and 2 combined data. Low, Intermediate and High Temperature Extrapolation Curves are plotted based on three sets of data: virgin asphalt binder (PG64-28), critical temperatures of the asphalt blend obtained in Mortar Experiment 1 and same data obtained in Mortar Experiment 2. The coefficient or determination of the curves is above 0.9 which indicates a good fitting of the data point in the linear regression. However, if a binder replacement of 100% (just RAP) was applied to the linear regression equations, the expected Low and Intermediate Critical Temperatures of the RAP asphalt binder would be different than the critical temperatures estimated by actual grading of the recovered RAP asphalt binder.

The results of Blending Chart Methods and the two Mortar Experiments are summarized in Table 20. The different Rate of Change in Temperature obtained after the three tests leads to estimate different critical temperatures when virgin asphalt binder and RAP asphalt binder are blended. In fact:

- Since both mortar experiments and blending charts agreed that a binder replacement of 15% did not change the low PG grade of the blending and the prediction of the high and intermediate critical temperatures were similar, a 15% of binder replacement was targeted as a first addition of RAP into mixtures.

- On the other hand, and in the search of a higher binder replacement, the blending chart indicated that a binder replacement of 35% was still feasible without changing the low PG grade of the blend, while both mortar experiments indicated a change in the low PG grade. However, the prediction of low PG grade was not accurate throughout the mortar experiment. Hence, a 35% binder replacement was targeted as a second target for addition of RAP into the mixtures.

4.5.4 Additional tests for verifying NV compliance of blending

Two blends of virgin asphalt binder and recovery RAP binder were prepared in order to verify the NV compliance of binder according to RTC specifications show in Table 2. The additional tests performed in the blends are listed below:

- Ductility at 4°C, Toughness and Tenacity at 25°C on original asphalt binder
- Ductility at 4°C on the RTFO-aged asphalt binder.

The blends were prepared according to the target binder replacements defined in 4.5.3, i.e., 15% and 35% binder replacements. The samples were sent to Paramount Petroleum Inc. to perform the tests and the results are shown in Table 21. Both two blends failed on Ductility test on the original and RTFO-aged conditions. Hence, both two blends did not fulfill the specifications established for the virgin binder PG64-28NV.

4.6 Marshall Mix Designs

The Marshall Mix Design of the Control Mixture was performed following the guidelines established in MS-2 [19] and meeting the specifications of RTC of Washoe County shown in Table 22. The Control Mixture was labeled as HMA0, where “HMA” stands for Mixture with no Warm Mix Technology and “0” stands for no RAP in the mixture. Several Warm Mix Technologies (Advera, Evotherm 3G and SonneWarmix) were used in this study and along with three levels of RAP in the mixture; labels were defined consequently for the twelve mixtures in the study. For example, ADV15 stands for a mixture containing Advera as a Warm Mix Technology and 15% of RAP as a target for binder replacement in the mixture at the optimum asphalt binder content. Regardless of the nomenclature, some mixtures may have a binder replacement in the neighborhood of the target binder replacement.

Five stockpiles were considered from Western Nevada Materials, Martin Marietta – Spanish Spring pit. Table 8 summarizes Gradation and Specific Gravity of the five stockpiles. The range between the various bulk specific gravity of the stockpiles was less than 0.2; therefore a correction by volume of the gradation of the final blend was not required. Aggregates were preheated in the oven at mixing temperature overnight in order to prepare the sample specimens.

Since the asphalt binder was graded as PG64-28 (and labeled as polymer-modified asphalt binder), mixing and compacting temperatures were based on recommendations given by the supplier. The range of mixing and compaction temperatures were 160 °C to 166 °C (321 °F to 330 °F) and 149 °C to 154 °C (301 °F to 310 °F), respectively. Regarding the use of Warm-Mix Technologies, the mixing temperature was chosen at 135 °C (275 °F) based on the recommendations of the suppliers [4]. The compaction temperature was selected at 25 °F below the mixing temperature; therefore, the compaction temperature was chosen at 121 °C (250 °F). Table 23 summarizes the twelve Marshall Mix Designs conducted on aggregates from Spanish Springs, three levels of RAP and three WMA technologies.

Figure 12 shows the Air Voids percentages and Optimum Binder Content (by TWM) for the various mixtures. The requirements established in the RTC specifications [9] were met at Optimum Binder Contents (OBC) for the various mixtures. The OBCs were selected in the middle range for the control mixtures (HMA-YY series) while OBCs in WMA mixtures were selected in the lower end of the specifications in order to obtain a similar OBC for the various mixtures. Despite this criterion, the range of OBC was 0.7% among the mixtures leading to Air Voids percentages in the neighborhood of the upper end of the specification for some mixtures. On the other hand, the range of Air Voids was 1.0%, varying from the target Air Voids established in the Orange Book [9] to 5.0% of Air Voids. In general, the OBC for the WMA technologies with RAP were greater or equal to the Control Mixtures with the exception of the mixtures with no RAP. Air Voids of the WMA mixtures at OBC were greater than the Control Mixture. Air Voids at OBC tend to decrease if the percentage of RAP increases, with the exception of the control mixture.

Since the same compaction temperature was selected for the WMA mixtures (275 °F), OBC of the WMA mixtures may change if the compaction temperatures of the WMA technologies change. The adjustment of the compaction temperature can be achieved through the Compactability Test.

Marshall Stabilities of the various mixtures are shown in Figure 13. All mixtures presented a Marshall Stability above the minimum requirement. A trend of increasing Marshall Stability while RAP increases, is detected for Control and WMA mixtures with the exception of Advera mixtures. It is also noted a clear trend that WMA Stabilities are lower than the HMA-YY Stabilities.

Marshall Flows of the various mixtures are pointed out in Figure 14. All mixture's Flow met the specification established in the Orange Book. Flows in WMA mixtures were slightly greater than the Control Mixture, and there is no clear trend of Marshall Flow as RAP increases.

For WMA-mixtures, all nine mixtures showed a good coating at the OBC. The minimum criterion of 95% of coating the coarse particles is achieved. Some values around 95% are detected when 4.0% samples were prepared during Mix Design, indicating that higher asphalt binder content was needed. This phenomenon was more often when binder replacement increased.

Compactability testing was not performed. Superpave Mix Design establishes Ndesign values according to expected traffic level and they could not be directly associated with the Marshall Mix Design. Hence, the adjustment for compaction temperature was not conducted in order to verify the OBC of the WMA mixtures.

4.7 Performance Properties of Asphalt Mixtures

4.7.1 Moisture Sensitivity

Moisture Sensitivity samples were prepared targeting 7.0 ± 0.5 % air voids for the various Marshall Mix designs. Since the Marshall Compaction process is less controlled than the Gyratory Superpave Compactor, determining the mass and the number of blows per face was challenging. These parameters to compact the specimens were conducted by trial and error iterations. After the iterations, specifications of TS for the various mixtures specimens were met when the mass of mixture was selected around 1,150 g and number of blows per face varied from 25 to 41.

The results of the Moisture Sensitivity tests are shown in Figure 15 and Figure 16. Tensile Strength (TS) values of the various mixtures and after different Freeze/Thaw cycles were calculated using Equation 6 and shown in Figure 15. All the TS values were way above the minimum value specified in RTC Specifications [9]. In fact, the lower TS value was 105 psi, about 62% greater than the minimum TS required (65 psi). On the other hand, two trends can be identified: first, WMA mixtures presented lower TS when compared to HMA mixture. The second trend would indicate an increase of at least maintaining the TS when RAP content increased. Overall, a significant decrease in TS was noted after 3 Freeze/Thaw cycles rather than 1 Freeze/Thaw cycle at 95% of confidence (as shown in Table 24). Some disturbances were induced in these trends; an explanation could be the different asphalt binder contents increases or decreases when adding RAP material, or a binder content close to the lower end in the range based on Marshall Mix Designs.

Figure 16 points out the Tensile Strength Ratio (TSR) of the various mixtures and after different conditioning. First of all, all TSR were greater than the minimum (75%) specified in the RTC Orange Book. On the other hand, AASHTO R35 establishes a minimum of 80% for WMA mixtures, and all the mixtures met the minimum of 80% TSR. In general, TSR after 1 FT cycles had a tendency to be greater than the TSR after 3 FT cycles with some exceptions (2 out 12 mixtures). Anther observed trend was a decrease in the TSR ratio as RAP increases, with some exceptions (5 out 12 mixtures). Test samples did not show evidence of stripping after Freeze/Thaw cycles. Some disturbances could be associated with different asphalt binder contents, VFA, and the variability itself of the test. In addition, the preparation of the samples using Marshall Compactor seems to be less accurate than using the Superpave Gyratory Compactor.

4.7.2 Dynamic Modulus |E*|

The dynamic Modulus was measured for the various mixtures without moisture damage conditioning and after three Freeze/Thaw cycles. Figure 17 shows the Dynamic Modulus of the various mixtures at different Freeze/Thaw cycles at 10 Hz, representing highway traffic [52], and at a temperature of 68 °F. Data would point out two trends: a) as RAP content increased, Dynamic Modulus increased, and b) more or less, the Dynamic Modulus for the various WMA-mixtures was lower than the Dynamic Modulus of the HMA-mixtures at the same RAP content. On the other hand, the decrease in Dynamic Modulus in the twelve mixtures was statistically not significant after three Freeze/Thaw cycles at a 95% confidence level (as shown in Table 25). Dynamic Master Curves for the various mixtures and after different conditioning are shown in Figure 22 to Figure 41.

The ratio of the moisture-conditioned to the unconditioned Dynamic Modulus after three Freeze/Thaw cycles is shown in Figure 18. Overall, the reduction of Dynamic Modulus had a tendency to increase as the RAP content increases, with the exception of the Sonneborn mixtures. Some mixtures show an increase in the Dynamic Modulus after three Freeze/Thaw cycles. The greatest reduction of Dynamic Modulus was detected in the Evotherm-WMA mixture with the 35%RAP.

Shift Factors used in the construction of the Dynamic Master Curves are shown in Figure 19 to Figure 21. Since |E*| Curves were calculated by testing samples at three temperatures (4° C, 20 °C and 40 °C), three plots of Shift Factors are shown with respect to the reference temperature 21.1 °C (70 °F) and plotted in log-scale on the y-axis. For most mixtures, Shift Factors decreased after induced moisture damage. HMA mixtures showed a tendency to decrease the Shift Factor as RAP percentage increase, whereas the opposite trend was observed for WMA mixtures.

4.7.3 Flow Number

The Flow Number was determined for the various mixtures and the results are shown in Figure 42. Test Temperature was selected at 58 °C for Reno according to the LTPP Bind 3.1 software [45]. Details of the LTTP Bind output are shown in Table 26.

According to the data shown in Figure 42, some tendencies were detected: a) Flow Numbers for WMA-mixtures were significantly lower than the HMA-mixture, b) Flow Number for HMA-mixtures increased as RAP content increased. The same behavior was observed for WMA-Advera mixtures, while WMA-Evotherm and WMA-SonneWarmix did not present a clear trend In the case of WMA-SonneWarmix, increasing the RAP content did not statistically affect the Flow Number at a 95% confidence level (as shown in Table 27). Minimum Flow Number Requirements for WMA mixtures were included in AASHTO R35 [44] (as shown in Table 7) and included in Figure 42. All WMA mixtures met the requirements for expected design traffic between 3 and 10 million of ESALs with the exceptions of WMA-Evotherm and Advera with 35%RAP which qualify for a higher traffic level. On the other hand, HMA-mixtures met the specification for expected design traffic between 10 and 30 million of ESALs.

4.7.4 Resistance to Reflective Cracking

The reflective cracking tests could not be conducted due to a failure in the TTI Overlay Tester equipment.

CHAPTER 5 - CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

The virgin asphalt binder used in this study is graded as PG64-28 and meets the RTC specifications. As expected, the continuous grade of the asphalt binder is PG68.8-32.5. The Compliance of the binder to the Nevada PG plus specification was not determined in this study but it was assumed to meet the RTC specifications. On the other hand, asphalt binder recovered from RAP was graded as PG82-16 and the continuous grade was PG87.9-20 which is expected for local RAP material.

The Mortar Experiment was conducted at two different binder replacements and their results varied among them with respect to the blending chart. However, both mortar experiments exhibited comparable critical temperatures of the blended asphalt binder when binder replacement was low, but extrapolating the binder replacement to 30 percent or more did not offer a good repeatability. Moreover, the expected critical temperatures differed with respect to the blending chart. In addition, the asphalt blends at 15 percent and 35 percent binder replacement did not meet the RTC specifications for PG plus specification.

The Marshall Mix Design was selected to perform the mix designs for HMA and WMA mixtures. Overall, the results in terms of Marshall Stability were consistent with the expected effect of adding RAP and WMA technologies. The more RAP material is applied, the higher the Marshall Stability of the mixture. Adding WMA technologies implied lower Marshall Stabilities, but still higher than the specification limit. When it comes to Marshall Flows, there was not a clear trend based on varying the WMA technology or binder replacement.

For Moisture Sensitivity, the TSR in general decreased with the increase in number of applied Freeze/Thaw cycles. The TS of WMA-mixtures was found to be lower than the TS of HMA-mixtures, and there was not a clear trend for TS or TSR when binder replacement increased. The lower TS values were observed for the SonneWarmix mixtures, while the lower TSR values were observed for the Advera mixture at the three levels of binder replacement. Both TS and TSR values for all mixtures were greater than the specification limits.

The Dynamic Modulus increased as the binder replacement increased for all evaluated mixtures. When WMA technology was included, the Dynamic Modulus decreased when mixtures with the same binder replacement were compared. Some mixtures showed some recovery in the Dynamic Modulus after applying the Freeze/Thaw cycles and it may be attributed to the combined action of the hydrated lime and the WMA technology.

The Flow Number data showed a significant decrease when WMA technology was applied in the HMA mixtures. In the case of the HMA mixtures, a steady increase of the resistance to flow was observed when binder replacement was increased. However, WMA mixtures did not exhibit a clear change in the flow number when binder replacement was increased. Since AASTHO R35 establishes Flow Number Requirement according to expected traffic level, the majority of the WMA mixtures would be suitable for traffic level between 3 and 10 million of ESALs, while HMA mixtures would be suitable for greater traffic levels.

The use of WMA mixtures with high percentages of RAP would be allowed in specific projects (for example, low expected traffic levels and light load levels). However, further research regarding performance properties at intermediate and low temperature is needed to establish a judgment about the use of mixtures with high percentages of RAP for RTC projects.

5.2 Recommendations

Further research should be performed on Mortar Experiment. Some difficulties were observed during samples preparation regarding segregation of the mortars and handling the samples after RTFO-aging. In particular, preparation of the mortar beams to estimate the low temperature properties was subjected to high temperatures in the oven for two or more hours in order to pour the mortar into the beam molds. This overheating of the samples may lead to some bias in the stiffness of the beams at low temperature when evaluated in the BBR. In addition, further research is needed to determine the validity of the assumptions of linearity for Rate of Change of CG.

Performing the Mix Design using Superpave methodology may lead to more consistent results and performance assessments. In particular, conducting the compaction of the test specimens for mix design using the Superpave Gyratory Compactor would be closer to the actual compaction process in the field. Moreover, the compaction variable would be avoided when comparing consistency of results of Resistance to Moisture Damage to results of Resistance to Rutting and Dynamic Modulus.

If a Superpave Mix Design is not conducted, comprehensive research should be conducted to obtain an appropriate Ndesign number for the mixtures. Hence, a consistency between mix design and Compactability test would be addressed.

Asphalt binder should be recovered and graded for the various mixtures to determine the performance grade of the binder at high RAP content and whether the WMA technologies play a role in the performance grade of the asphalt binder with different RAP contents

2D imaging should be conducted to assess the aggregate coating and the number of aggregate contacts. Fatigue Beam Tests, Resistance to Reflective Cracking and Resistance to Thermal Cracking should be assessed in order to fully characterize the performance of various mixtures.

As aggregates were conditioned with lime to diminish the potential of moisture damage, antistripping properties of the WMA technologies were not identified. A further research would be needed in order to identify whether the WMA technologies display potential resistance to moisture damage.

Further study should be performed to assess the Cost/Benefits of including WMA technologies and high percentages of RAP in mixtures. Including WMA technologies may reduce some cost related to less fuel consumption but plant production rates should be estimated and measured. In addition, Costs/Benefits including WMA technologies should differentiate the impact in cost for Constructor as well as for Agency.

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TABLES

Table 1: Asphalt Mixtures Nomenclature

Label	Warm Mix Technology	RAP Content, %	Remarks
HMA0	None	0	Control HMA Mix
HMA15		15	-
HMA35		35	-
ADV0	Advera	0	-
ADV15		15	-
ADV35		35	-
EVO0	Evotherm 3G	0	-
EVO15		15	-
EVO35		35	-
SON0	SonneWarmix	0	-
SON15		15	-
SON35		35	-

Table 2: Specifications for Polymerized Performance Graded Asphalt Cement [9]

Test	Test Method	Requirements	Limit with Tolerance	Rejection Limit
		PG 64-28NV		
Tests on Original Asphalt Cement				
Rotational Viscosity @135° C (Pa.s)	AASHTO T316	3.00 Maximum	3.21 Maximum	3.50 Maximum
Flash Point using Cleveland Open Cup (°C)	NEV T716	230 Minimum	222 Minimum	163 Minimum
Ductility @4° C, 5 cm/min (cm)	NEV T746	50 Minimum	50 Minimum	29 Minimum
Toughness @25° C (inch-lbs)	NEV T745	110 Minimum	110 Minimum	57 Minimum
Tenacity @25° C (inch-lbs)	NEV T745	75 Minimum	75 Minimum	22 Minimum
Sieve Test (%)	NEV T730	Pass	Pass	Fail
Dynamic Shear, $G^*/\sin\delta$ @64° C, 10 rads/sec (kPa)	AASHTO T315	1.00 Minimum	0.90 Minimum	0.75 Minimum
Tests on Residue from Rolling Thin Film Oven (AASHTO T 240)				
Ductility @4° C, 5 cm/min (cm)	NEV T746	25 Minimum	25 Minimum	4 Minimum
Dynamic Shear, $G^*/\sin\delta$ @64° C, 10 rads/sec (kPa)	AASHTO T315	2.20 Minimum	1.98 Minimum	1.65 Minimum
Average Mass Change (percent)	AASHTO T240	0.50 Maximum	0.50 Maximum	0.71 Maximum
Tests on Residue from Pressure Aging Vessel @100° C (AASHTO R28)				
Dynamic Shear, $G^*\sin\delta$ @22° C, 10 rads/sec (kPa)	AASHTO T315	5000 Maximum	5500 Maximum	6250 Maximum
Flexural Creep Stiffness	AASHTO T313(1)			
Stiffness Modulus, S @ -18° C, 60 sec (MPa)	AASHTO T313(1)	300 Maximum	330 Maximum	375 Maximum
m-value @ -18° C, 60 sec	AASHTO T313(1)	0.300 Minimum	0.290 Minimum	0.245 Minimum
Direct Tension, Failure Strain @-18° C, 1.0 mm/min (%)	AASHTO T314(1)	1.00 Minimum	0.90 Minimum	0.75 Minimum

1. If the creep stiffness is below 300MPa, the direct tension test is not required. If the creep stiffness is between 300 and 600MPa, the direct tension failure strain requirement may be used in lieu of the creep stiffness requirement. The m-value requirement must be satisfied in both cases.

Table 3: Comparison between Performance Grade grading in regular and recovered from RAP asphalt binders

Property	PG Grading	RAP binder grading
Flash Point	Performed on original asphalt binder	N/R
Rotational Viscosity	Performed on original asphalt binder	N/R
Dynamic Shear Rheometer, G*/sinδ	Performed on original asphalt binder	Performed on recovered asphalt binder
RTFO aging	Performed on original asphalt binder	Performed on recovered asphalt binder
Mass Loss	Performed on original asphalt binder	N/R
Dynamic Shear Rheometer, G*/sinδ	Performed on RTFO-aged asphalt binder	Performed on RFTO-aged recovered asphalt binder
PAV aging	Performed on RTFO-aged asphalt binder	N/R
Dynamic Shear Rheometer, G*×sinδ	Performed on RTFO+PAV-aged asphalt binder	Performed on RTFO-aged recovered asphalt binder
Bending Beam Rheometer, S(60s) and m(60s)	Performed on RTFO+PAV-aged asphalt binder	Performed on RTFO-aged recovered asphalt binder

N/R: Not required

Table 4: Binder Selection Guidelines for RAP Mixtures [17]

	RAP Percentage		
	Recovered RAP Grade		
Recommended Virgin Asphalt Binder Grade	**PG XX-22 or lower**	**PG XX-16**	**PG XX-10 or higher**
No change in binder selection	<20%	<15%	<10%
Select virgin binder one grade softer than normal (e.g., select a PG 58-28 if a PG 64-22 would normally be used)	20-30%	15-25%	10-15%
Follow recommendations from blending charts	>30%	>25%	>15%

Table 5: Required Test Specimens for Mortar Experiment Analysis

Material	**Test Temperature / Device**			
	Low / BBR	**Intermediate / DSR**	**High / DSR**	
Virgin asphalt binder	Same as PG System			
SRAP	PAV-Aged: RTFO binder + RAP		RTFO Binder + RAP	Original Binder + RAP
RRAP	PAV-Aged: RTFO binder + RAP aggregates		RTFO Binder + RAP aggregates	Original Binder + RAP aggregates

Table 6: Bending Beam Rheometer Test Load in mN

Test Temperature, °C	PAV asphalt binder	PAV Mortar
0	980	980
-6		1,980
-12		2,980
-18		3,980
-24		4,980

Table 7: Minimum Flow Number Requirements

Traffic Level, Millions ESALs	Minimum Flow Number
< 3	NA
3 to < 10	30
10 to <30	105
≥ 30	415

Table 8: Aggregates Characterization

	Percentage passing, %				
Sieve	**WS**	**IS**	**CS**	**1/2"**	**3/4"**
1"	100	100	100	100	100
3/4"	100	100	100	100	100
1/2"	100	100	100	100	47
3/8"	100	100	100	88	7
#4	98	94	100	22	1
#10	63	62	85	4	1
#16	44	48	65	3	1
#40	20	33	28	3	1
#100	5	23	6	2	1
#200	3	17	3	2	1
Coarse Specific Gravity					
Gsb	-	-	-	2.664	2.677
Gsb SSD	-	-	-	2.689	2.692
Gsa	-	-	-	2.731	2.720
Absorption, %	-	-	-	0.9	0.6
Fine Specific Gravity					
Gsb	2.603	2.639	2.548	-	-
Gsb SSD	2.646	2.671	2.596	-	-
Gsa	2.721	2.727	2.678	-	-
Absorption, %	1.7	1.2	1.9	-	-

WS: Wade Sand
IS: Impact Sand
CS: Concrete Sand
1/2”: Coarse Aggregate, Nominal Maximum Size 1/2 inches
3/4”: Coarse Aggregate, Nominal Maximum Size 3/4 inches

Table 9: Gradation Limits for RTC asphalt wearing courses [9]

Sieve Size	**Percentage by Weight Passing Sieve**
	Type 2
1 inch	100
¾ inch	90 – 100
3/8 inch	63 – 85
No. 4	45 – 65
No. 10	30 – 44
No. 40	12 – 22
N0 200	3 – 8

Table 10: Performance Grade Analysis of Virgin Binder

Test	**Temperature °C**	**Average Value**	**PG Criteria**	**Met specification?**
Flash point, °C	-	308	≥ 230	Yes
Rotational Viscosity, Pa s	135	0.76	≤ 3.00	Yes
DSR, Original binder. G* / sin(δ), kPa	58 64 70	2.92 1.62 0.91	≥ 1.00	Yes Yes No
Mass Loss, %	-	0.4	≤ 1.0	Yes
DSR, RTFO Aged Binder. G* / sin(δ), kPa	58 64 70	6.25 3.50 2.01	≥ 2.20	Yes Yes No
DSR, RTFO + PAV Aged Binder. G* x sin(δ), kPa	16 19 22	3,350 2,303 1,559	≤ 5,000	Yes Yes Yes
BBR. RTFO + PAV Aged Binder. Stiffness at t = 60 sec, MPa	-12 -18 -24	59 137 311	≤ 300	Yes Yes No
BBR. RTFO + PAV Aged Binder. m-value at t = 60 sec	-12 -18 -24	0.39 0.33 0.29	≥ 0.30	Yes Yes No

Table 11: Performance Grade of RAP Binder

Test	Temperature (°C)	Average Value	PG Criteria	Met specification?
DSR, Original binder. G* / sin(δ), kPa	82 88 94	2.75 1.34 0.65	≥ 1.00	Yes Yes No
DSR, RTFO Aged Binder. G* / sin(δ), kPa	76 82 88	6.87 4.42 2.13	≥ 2.20	Yes Yes No
DSR, RTFO Aged Binder. G* x sin(δ), kPa	34 31 28	2,334 3,450 4,917	≤ 5,000	Yes Yes Yes
BBR. RTFO Aged Binder. Stiffness at t = 60 sec, MPa	-6 -12	123 231	≤ 300	Yes Yes
BBR. RTFO Aged Binder. m-value at t = 60 sec	-6 -12	0.33 0.28	≥ 0.30	Yes No

Table 12: Recovered RAP Aggregates Properties

Gradation	
Sieve	Percentage passing, %
3/4"	100
1/2"	100
3/8"	99
#4	79
#10	58
#16	48
#40	32
#100	20
#200	14
Coarse Fraction	
Gsb	2.636
Gsb SSD	2.668
Gsa	2.724
Absorption, %	1.2
Fine Fraction	
Gsb	2.553
Gsb SSD	2.613
Gsa	2.717
Absorption, %	2.4

Table 13: SRAP binder content using ignition oven and centrifuge methods

	Centrifuge	Ignition Oven	Standard Deviation between results
Binder Content, % (by TWM)	7.04	7.05	0.01

Table 14: Low Temperature Testing Results, 50% binder content

Calculation of δ_{RAP}				
Test Temperature, °C	**-12**		**-18**	
Test Specimen	**S(60) MPa**	**m-value**	**S(60) MPa**	**m-value**
Virgin binder	59	0.385	137	0.332
SRAP	196	0.368	369	0.308
RRAP	158	0.393	329	0.328
δ_{TX}	1.04	0.94	1.02	0.94
δ_{RAP}	1.03	0.94		
Calculation of Estimated Blended Binder Properties				
Test Temperature, °C	**-12**		**-18**	
Test Specimen	**Log S(60) MPa**	**m-value**	**Log S(60) MPa**	**m-value**
PAV Aged Virgin Binder	1.77	0.385	2.14	0.332
Estimated Blended Binder Properties	1.83	0.361	2.20	0.312
Stiffness Continuous Grading Temperature, °C				-32.3
m-value Continuous Grading Temperature, °C				-29.4
Rate of change in continuous grade				
Stiffness				0.163
m-value				0.294

Table 15: Intermediate Temperature Testing Results, 50% binder content

Calculation of δ_{RAP}		
Test Temperature, °C	**19**	**22**
Test Specimen	**G*×sinδ PAV kPa**	**G*×sinδ PAV kPa**
Virgin binder	2,303	1,559
SRAP	7,390	5,515
RRAP	6,535	4,600
δ_{TX}	1.014	1.022
δ_{RAP}	1.018	
Calculation of Estimated Blended Binder Properties		
Test Temperature, °C	**19**	**22**
Test Specimen	**G*×sinδ PAV kPa**	**G*×sinδ PAV kPa**
PAV-Aged Binder	3.362	3.193
Estimated Blended Binder Properties	3.422	3.250
Intermediate Continuous Grading Temperature, °C		14.2
Rate of change in continuous grade		
Intermediate Temperature		0.151

Table 16: High Temperature Testing Results, 50% binder content

Calculation of δ_{RAP}				
Test Temperature, °C	**64**		**70**	
Test Specimen	**G*/sinδ**			
	Original	**RTFO**	**Original**	**RTFO**
	Pa			
Virgin binder	1,620	3,505	910	2,006
SRAP	7,390	13,105	4,170	7,245
RRAP	5,940	10,850	3,470	6,185
δ_{TX}	1.025	1.020	1.023	1.018
δ_{RAP}	1.024	1.019		
Calculation of Estimated Blended Binder Properties				
Test Temperature, °C	**64**		**70**	
Test Specimen	**G*/sinδ**			
	Original	**RTFO**	**Original**	**RTFO**
	Pa			
Binder	3.210	3.545	2.959	3.302
Estimated Blended Binder Properties	3.286	3.613	3.030	3.366
Original High Temperature Continuous Grading Temperature, °C				70.7
RTFO-aged High Temperature Continuous Grading Temperature, °C				70.6
Rate of change in continuous grade				
Original High Temperature				0.221
RTFO-aged High Temperature				0.206

Table 17: Low Temperature Testing Results, 34% binder content

Calculation of δ_{RAP}				
Test Temperature, °C	**-12**		**-18**	
Test Specimen	**S(60) MPa**	**m-value**	**S(60) MPa**	**m-value**
Virgin Binder	59	0.385	137	0.332
SRAP	481	0.354	996	0.278
RRAP	353	0.394	860	0.303
δ_{TX}	1.05	0.90	1.02	0.92
δ_{RAP}	1.04	0.91		
Calculation of Estimated Blended Binder Properties				
Test Temperature, °C	**-12**		**-18**	
Test Specimen	**Log S(60) MPa**	**m-value**	**Log S(60) MPa**	**m-value**
PAV Aged Virgin Binder	1.77	0.385	2.14	0.332
Estimated Blended Binder Properties	1.84	0.350	2.22	0.302
Stiffness Continuous Grading Temperature, °C				-32.1
m-value Continuous Grading Temperature, °C				-28.2
Rate of change in continuous grade				
Stiffness				0.097
m-value				0.228

Table 18: Intermediate Temperature Testing Results, 34% binder content

Calculation of δ_{RAP}		
Test Temperature, °C	**19**	**22**
Test Specimen	**G*×sinδ PAV kPa**	**G*×sinδ PAV kPa**
Virgin binder	2,303	1,559
SRAP	13,450	9,675
RRAP	6,535	5,470
δ_{TX}	1.082	1.066
δ_{RAP}	1.074	
Calculation of Estimated Blended Binder Properties		
Test Temperature, °C	**19**	**22**
Test Specimen	**G*×sinδ PAV kPa**	**G*×sinδ PAV kPa**
PAV-Aged Binder	3.362	3.193
Estimated Blended Binder Properties	3.612	3.430
Intermediate Continuous Grading Temperature, °C		17.6
Rate of change in continuous grade		
Intermediate Temperature		0.302

Table 19: High Temperature Testing Results, 34% binder content

Calculation of δ_{RAP}				
Test Temperature, °C	**64**		**70**	
Test Specimen	**G*/sinδ**			
	Original	**RTFO**	**Original**	**RTFO**
	Pa			
Virgin binder	1,620	3,505	910	2,006
SRAP	19,050	35,400	9,920	18,650
RRAP	13,250	26,300	7,400	14,400
δ_{TX}	1.038	1.029	1.033	1.027
δ_{RAP}	1.036	1.028		
Calculation of Estimated Blended Binder Properties				
Test Temperature, °C	**64**		**70**	
Test Specimen	**G*/sinδ**			
	Original	**RTFO**	**Original**	**RTFO**
	Pa			
Binder, log	3.210	3.545	2.959	3.302
Estimated Blended Binder Properties	3.324	3.644	3.064	3.395
Original High Temperature Continuous Grading Temperature, °C				71.5
RTFO-aged High Temperature Continuous Grading Temperature, °C				71.3
Rate of change in continuous grade				
Original High Temperature				0.165
RTFO-aged High Temperature				0.151

Table 20: Comparison of Rate of Change in Temperature for the various experiments

Experiment	**Change in Continuous Grade, °C/%PBR**		
	High	**Intermediate**	**Low**
Blending Chart	0.193	0.149	0.125
Mortar Experiment 1	0.221	0.151	0.294
Mortar Experiment 2	0.165	0.302	0.228

Table 21: NV Compliance of Asphalt Binder Blends at 15% and 35% Binder Replacement

Aging Condition	**Binder Replacement, %**	**Test**	**Result**	**Meet Specification?**
Original	15	Ductility at 4 °C, cm	47	No
		Toughness at 25 °C, in-lb	160	Yes
		Tenacity at 25 °C, in-lb	113	Yes
	35	Ductility at 4 °C, cm	14	No
		Toughness at 25 °C, in-lb	170	Yes
		Tenacity at 25 °C, in-lb	143	Yes
RTFO-Aged	15	Ductility at 4 °C, cm	15	No
	35	Ductility at 4 °C, cm	0.5	No

Table 22: Mixture Requirements [9]

Test	**Test Method**	**Requirements**	
		Design ESALs > 10^4, 75 blows per side	
Air Voids, % (target)	ASTM D 3203	3	4
Voids in Mineral Aggregates, %	MS-2	-	65 – 75
Voids Filled with Asphalt, %	MS-2	Per Table 5.2 of MS-2	
Marshall Stability, lb	ASTM D 6927	1,800 minimum	
Marshall Flow, 0.01 inch	ASTM D 6927	8 – 20	
Tensile Strength, psi	AASHTO T283	65 minimum	
Tensile Strength Ratio, %	AASHTO T283	70 minimum	
Hydrated Lime, % (target)	-	1.5	
RAP in total mix, % (by dry weight of aggregates)	-	15 maximum	

Table 23: Mixture Mix Designs

Mixture ID	**HMA0**	**HMA15**	**HMA35**	**EVO0**	**EVO15**	**EVO35**	**ADV0**	**ADV15**	**ADV35**	**SON0**	**SON15**	**SON35**
Hydrated Lime, %	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
WMA Technology		-			Evotherm 3G			Advera			SonneWarmix	
Mixing/Compacting Temperature, F		330/310			275/250			275/250			275/250	
Optimum Asphalt Content, % (DWA)	5.0	5.2	5.2	5.0	5.7	5.3	4.9	5.2	5.3	4.9	5.2	5.2
Optimum Asphalt Content, % (TWM)	4.8	4.9	4.9	4.8	5.4	5.0	4.7	4.9	5.0	4.7	4.9	4.9
Air Voids, %	4.0	4.0	4.2	5.0	5.0	4.5	4.9	4.8	4.3	4.9	4.8	4.6
Stability, lb	4,480	4,682	5,166	3,882	2,909	3,698	3,283	3,521	3,934	3,379	3,491	3,575
Flow, 0.01"	11	10	13	13	16	13	13	12	14	10	12	13
Maximum Specific Gravity at OBC, Gmm	2.493	2.491	2.497	2.498	2.477	2.481	2.491	2.480	2.492	2.494	2.486	2.492
Voids Filled with Asphalt (VFA), %	70.7	70.0	67.0	65.9	66.4	66.7	66.0	66.4	66.9	65.6	65.6	65.3
Voids in the Mineral Aggregate (VMA), %	14	13	13	15	15	14	15	14	13	14	14	13
Binder Replacement, %	0.0	13.7	31.8	0.0	12.4	31.1	0.0	13.7	31.1	0.0	13.7	31.8
Pba, %	0.7	1.0	1.3	0.7	1.1	1.1	0.6	0.8	1.3	0.6	0.9	1.2
Aggregates Stockpiles						**Bin Proportions, %**						
3/4”: Coarse Aggregate, NMS 3/4 inches	20	20	20	20	20	20	20	20	20	20	20	20
1/2”: Coarse Aggregate, NMS 1/2 inches	25	22	21	25	22	21	25	22	21	25	22	21
Concrete Sand	5	11	5	5	11	5	5	11	5	5	11	5
Impact Sand	30	17	5	30	17	5	30	17	5	30	17	5
Wade Sand	20	14	12	20	14	12	20	14	12	20	14	12
RAP	0	16	37	0	16	37	0	16	37	0	16	37

Table 24: Statistical Evaluation of Tensile Strength Results at 77 °F after 0, 1 and 3 Freeze/Thaw cycles

Mixture	1 FT cycles		3 FT cycles	
	p-value	Significance	p-value	Significance
HMA0	0.177	NS	0.228	NS
HMA15	0.086	NS	0.580	NS
HMA35	0.003	S	0.013	S
EVO0	0.250	NS	0.493	NS
EVO15	0.005	S	0.018	S
EVO35	0.025	S	0.065	NS
ADV0	0.064	NS	0.020	S
ADV15	0.119	NS	0.064	NS
ADV35	0.028	S	0.022	S
SON0	0.031	S	0.021	S
SON15	0.648	NS	0.003	S
SON35	0.392	NS	0.240	NS

S: Significant
NS: Not Significant

Table 25: Statistical Evaluation of Dynamic Modulus Results at 68 °F, 10 Hz after 0 and 3 Freeze/Thaw cycles

Mixture	p-Value	Significance
HMA0	0.407	NS
HMA15	0.333	NS
HMA35	0.034	NS
EVO0	0.878	NS
EVO15	0.307	NS
EVO35	0.145	NS
ADV0	0.456	NS
ADV15	0.391	NS
ADV35	0.550	NS
SON0	0.615	NS
SON15	0.257	NS
SON35	0.235	NS

S: Significant
NS: Not Significant

Table 26: LTPPBind Detailed Report for Reno Cannon Intl AP - LTTBind 3.1

LTPPBind V3.0 Detailed Report: (Date 10/14/2013)					
Reno Cannon Intl AP - LTTBind 3.1					
State/Province	NV				
Station Name	RENO CANNON INTL AP				
Station ID	NV6779				
County / District	WASHOE				
Last Year Data Avail.	1997				
Latitude	39.5				
Longitude	119.78				
Elevation, m	1246				
Air Temperature	Mean	Std Dev	Min	Max	Years
High Air Temperature, Deg. C	36.4	1.2	33	39.3	35
Low Air Temperature, Deg. C	-17.4	4.4	-26.5	-10	35
Low Air Temp. Drop, Deg. C	27.6	2.7	23	33.5	35
Degree Days over 10 Deg. C	3151	190	2830	3488	35
Pavement Temperature and PG	HIGH	LOW	High Rel	Low Rel	
Pavement Temperature, C	57.8	-11.6	50	50	
50% Reliability PG	58	-16	59	88	
>50% Reliability PG	64	-16	98	88	
=	64	-22	98	98	

Table 27: Statistical Evaluation of Flow Number Results

Comparison Type	**p-value**	**Significance**
HMA15 vs. HMA0	0.003	S
HMA35 vs. HMA15	0.004	S
EVO15 vs. EVO0	0.045	S
EVO35 vs. EVO15	<0.001	S
EVO35 vs. EVO0	0.001	S
ADV15 vs. ADV0	0.005	S
ADV35 vs. ADV15	0.096	NS
SON15 vs. SON0	0.311	NS
SON35 vs. SON15	0.081	NS
SON35 vs. SON0	0.465	NS

S: Significant

NS: Not Significant

FIGURES

a)

b)

c)

Figure 1: WMA Technologies used in the study: a) Advera, b) Evotherm3G and c) SonneWarmix

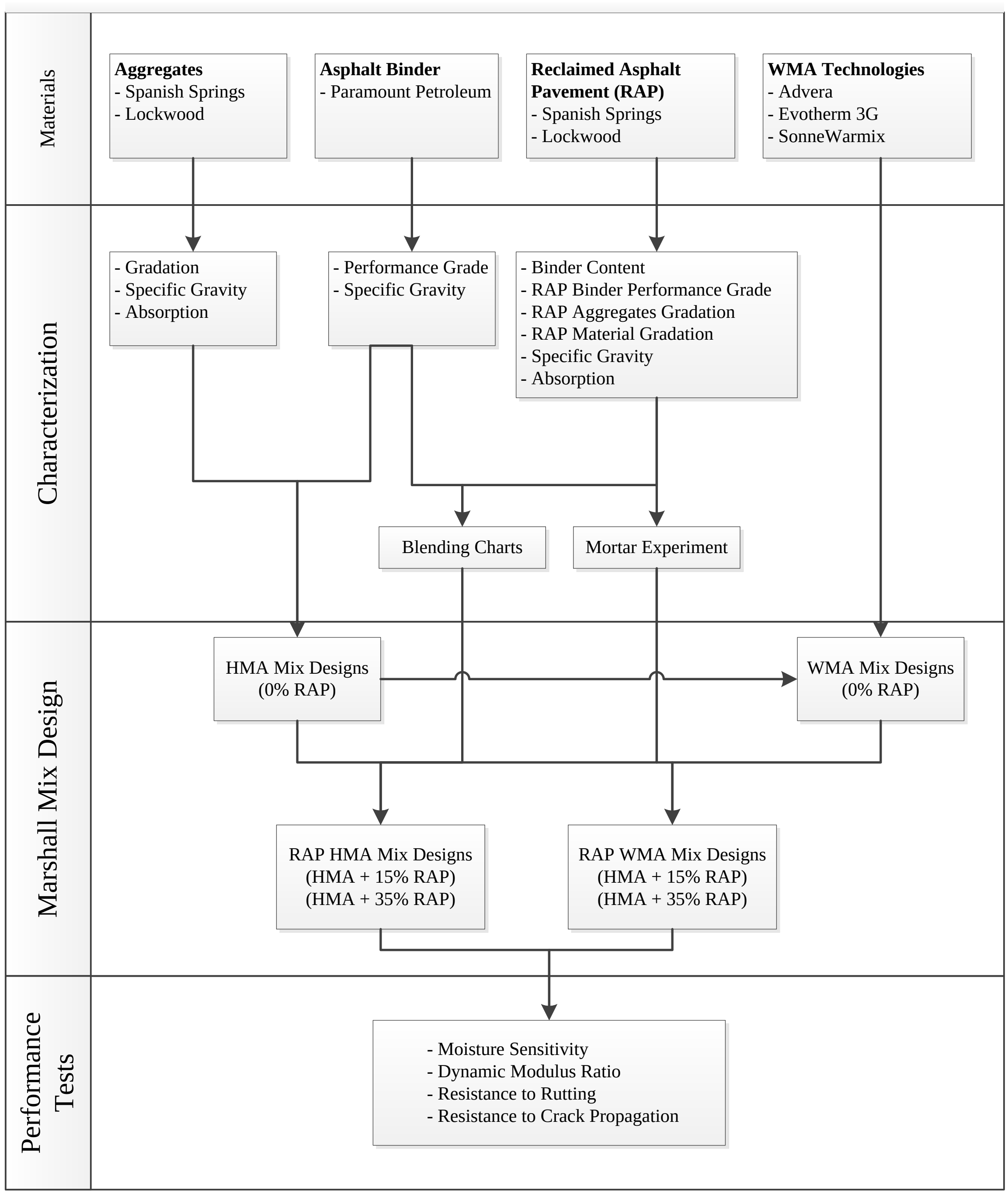

Figure 2: Overall Experimental Program

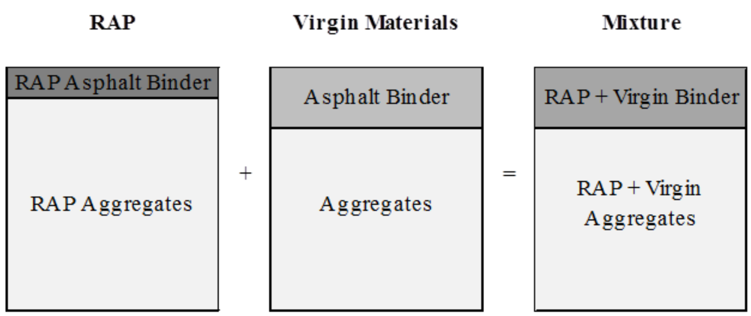

Figure 3: Phase diagram of mixing RAP and virgin materials in Hot Mix Asphalt.

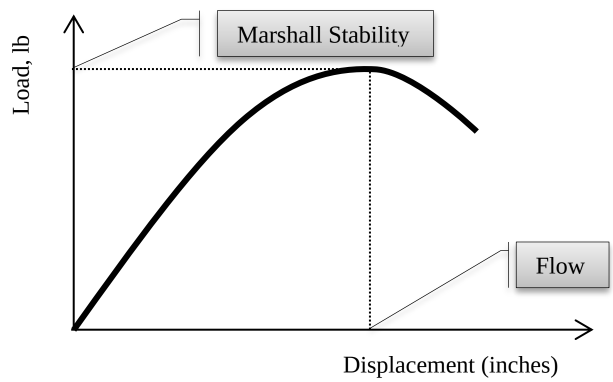

Figure 4: Load-displacement curve in Marshall Stability test

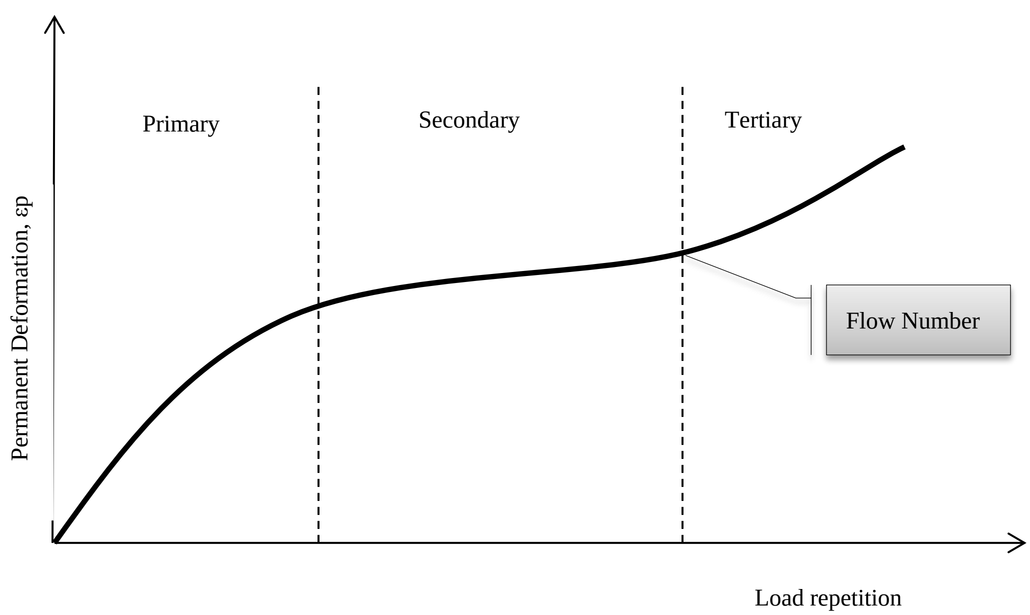

Figure 5: Repeated Load Permanent deformation behavior of asphalt mixtures

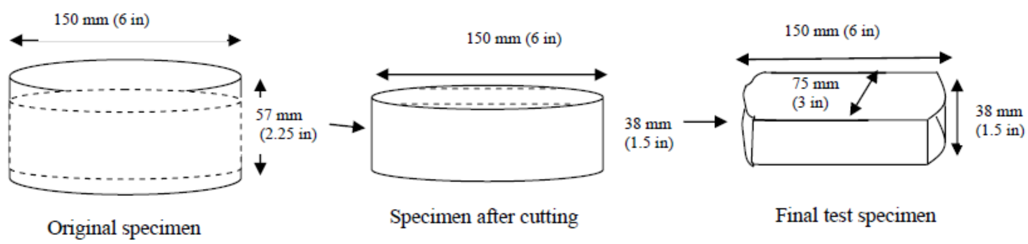

Figure 6: Process of Obtaining a TTI Overlay Tester sample

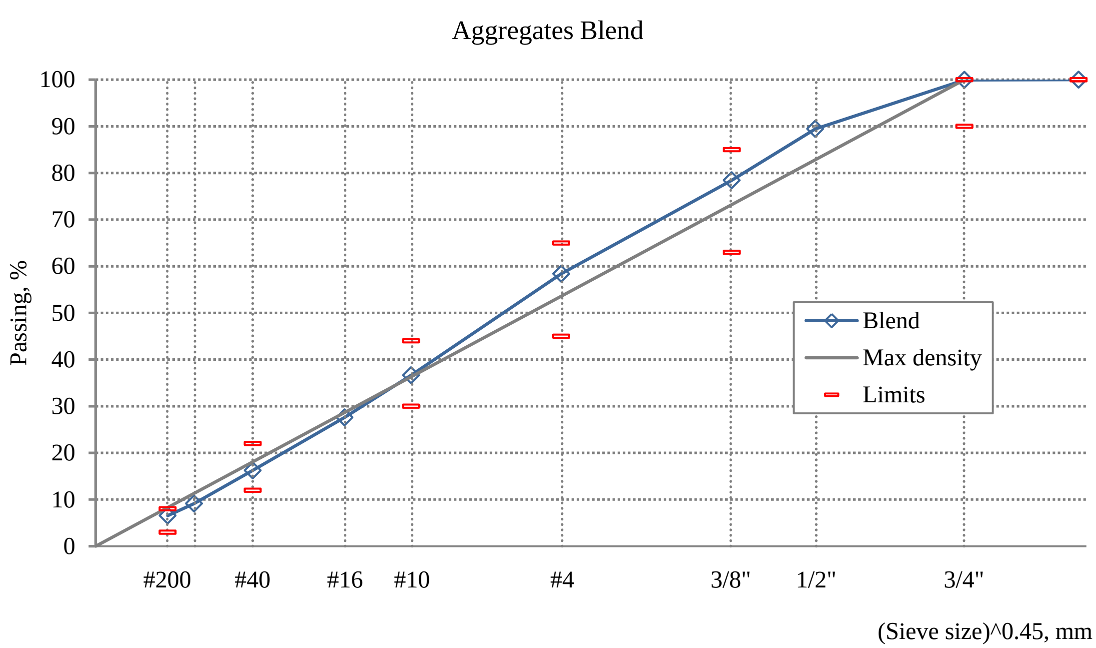

Figure 7: Gradation of Aggregates Blend to meet RTC Type 2 Aggregates Limits

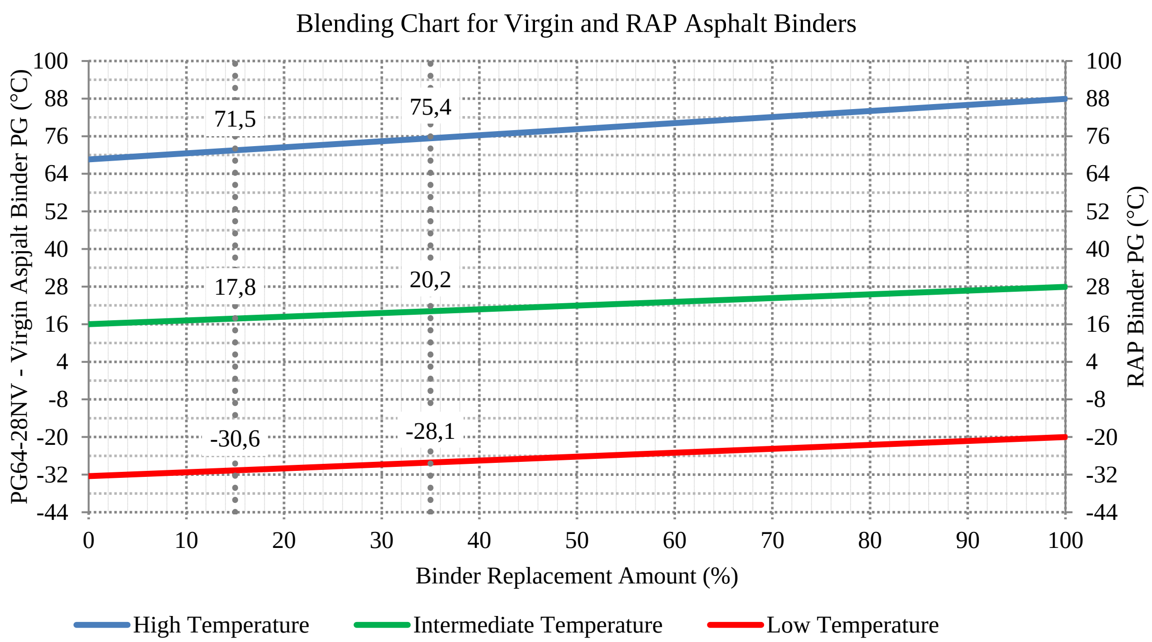

Figure 8: Blending Chart for Virgin and RAP asphalt binders at two Binder Replacement: a) 15%, b) 35%

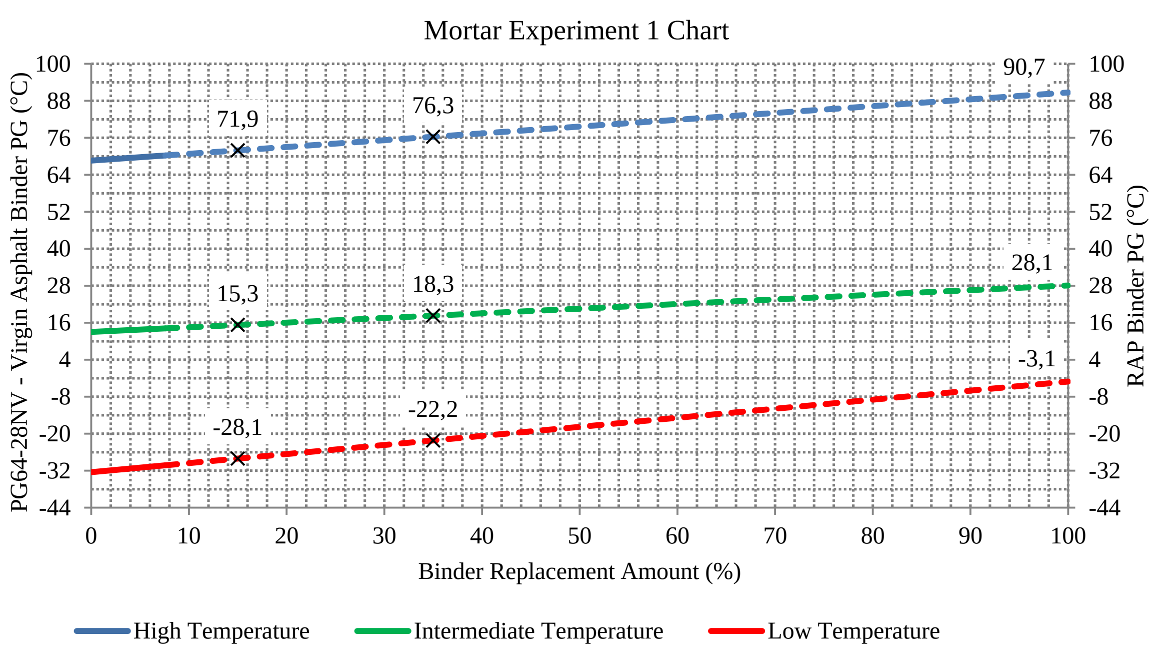

Figure 9: Mortar Experiment 1 performed at 50% total binder content (7.58% binder replacement).

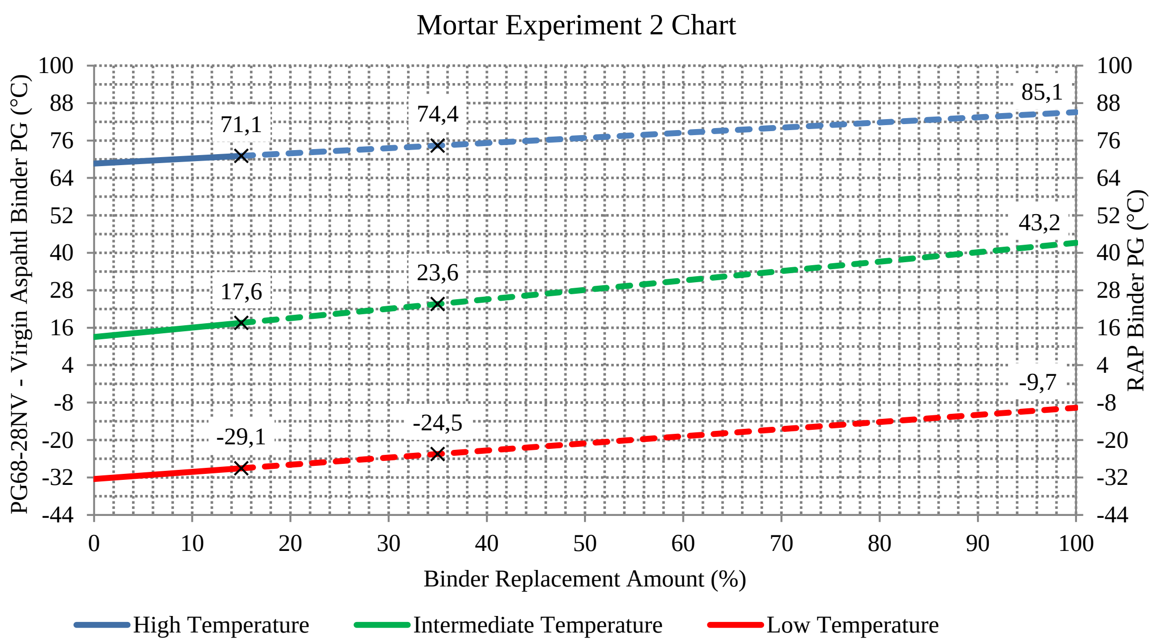

Figure 10: Mortar Experiment 2 performed at 34% total binder content (15% binder replacement).

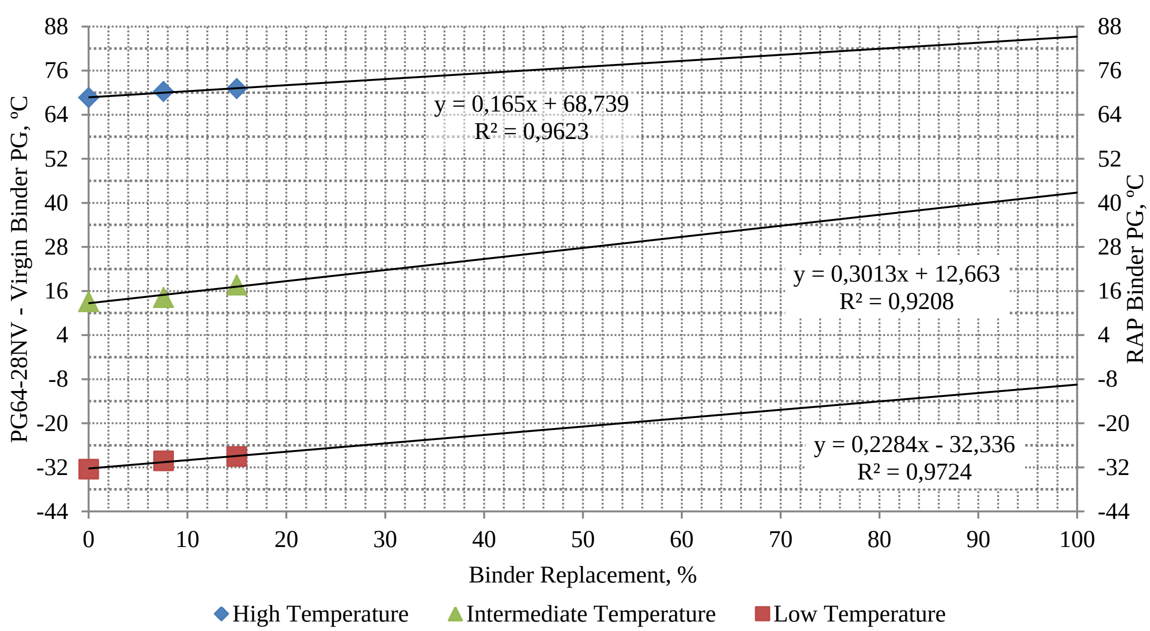

Figure 11: Extrapolation of Blend Critical Temperatures based on Virgin Asphalt Binder, Mortar Experiment 1 and 2 Critical Temperatures.

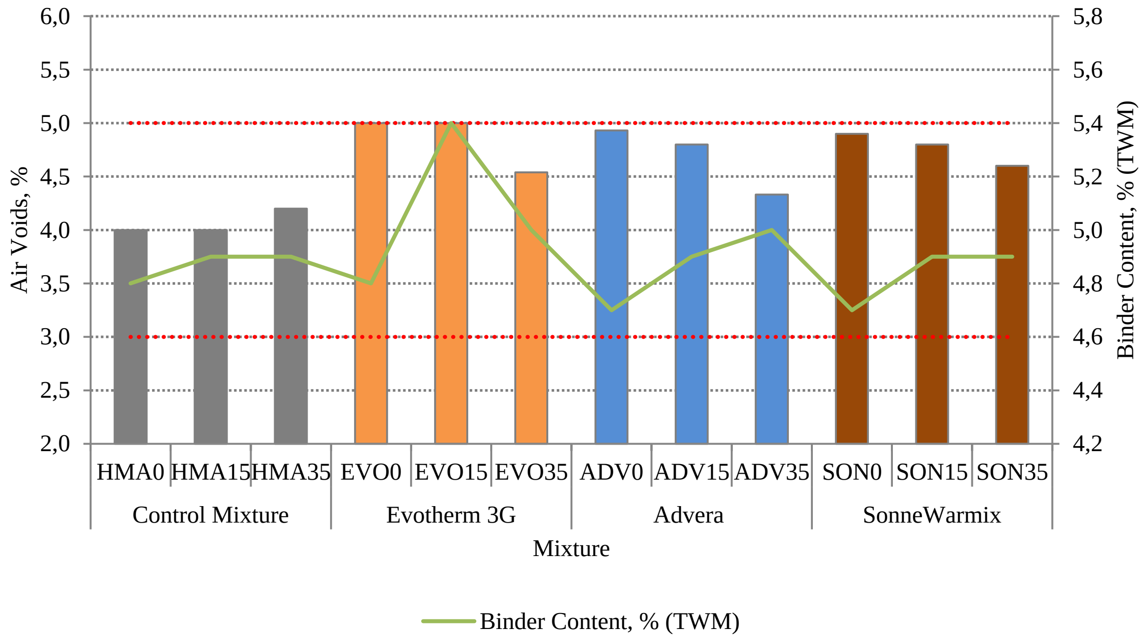

Figure 12: Air Voids and Optimum Binder Content of Marshall Mix Designs.

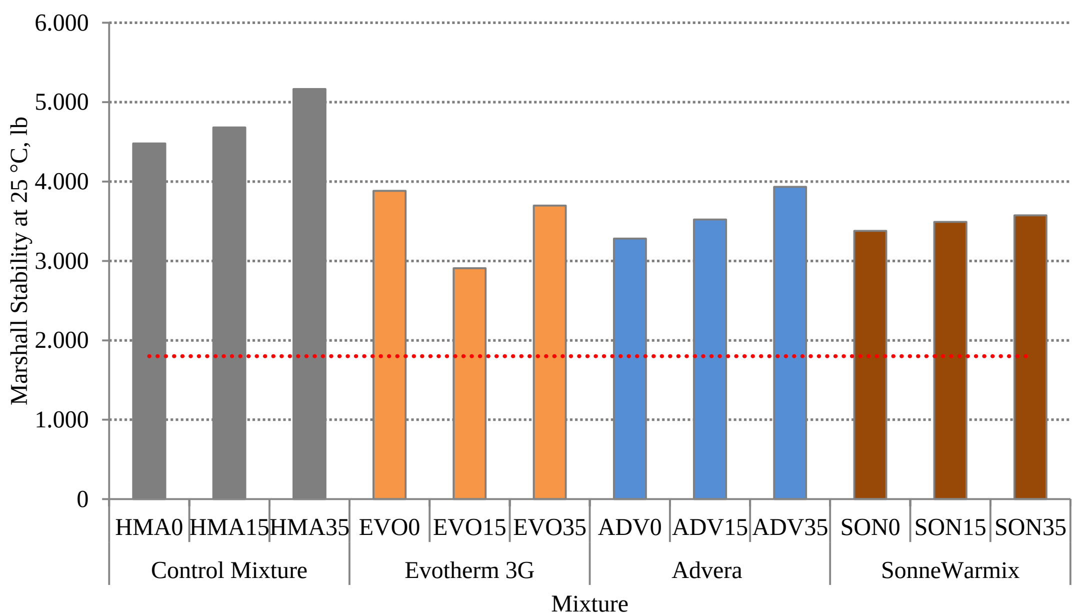

Figure 13: Marshall Stabilityat 25 °C for the various mixtures at Optimum Binder Content

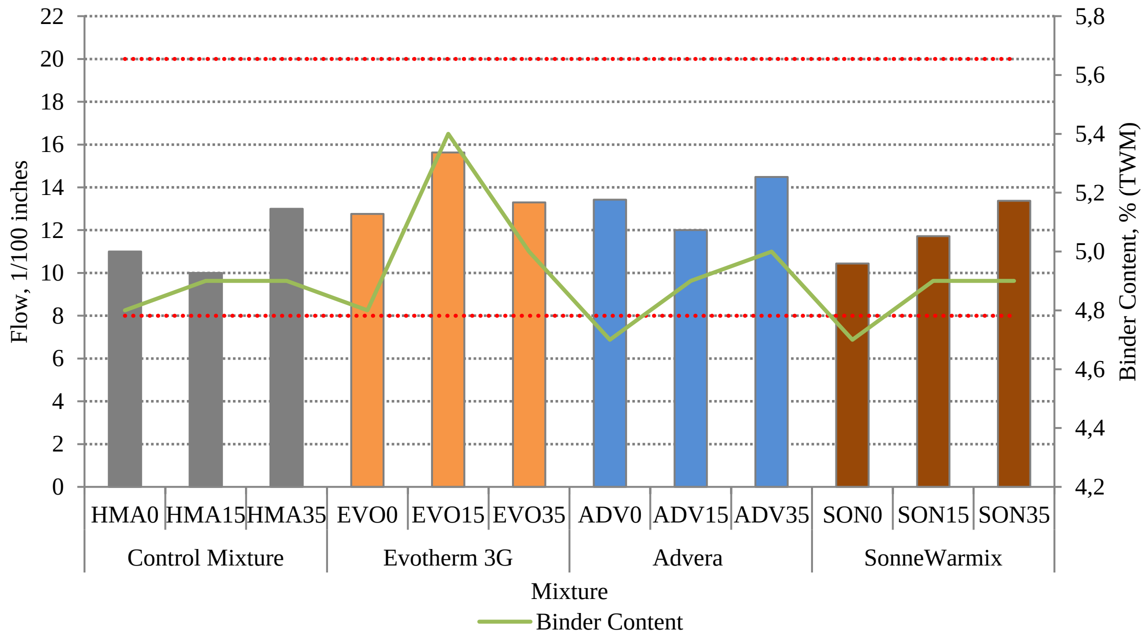

Figure 14: Marshall Flow for the various mixtures at Optimum Binder Content

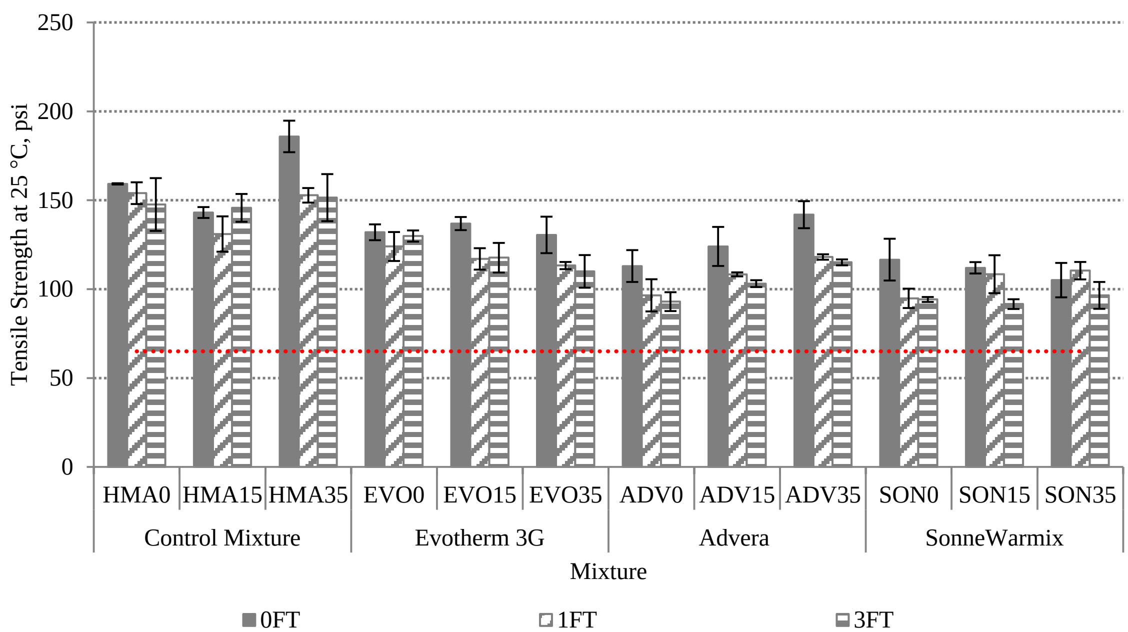

Figure 15: Tensile Strength of the various mixtures after 0, 1, and 3 Freeze/Thaw cycles (Bars represent 95% Confidence Interval)

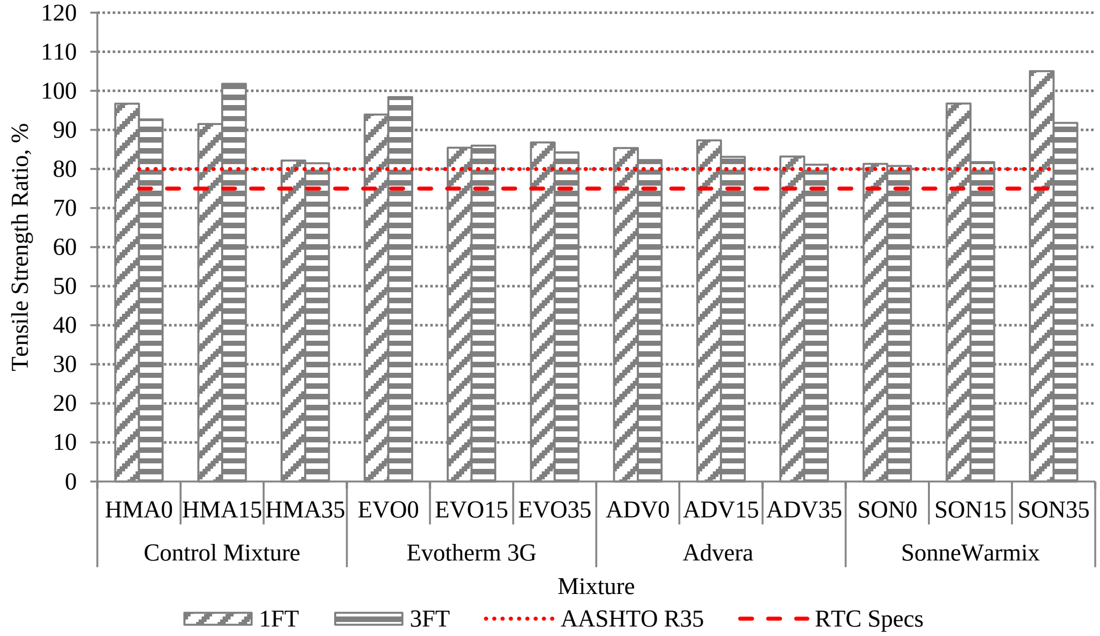

Figure 16: Tensile Strength Ratio of the various mixtures after 1 and 3 Freeze/Thaw cycles

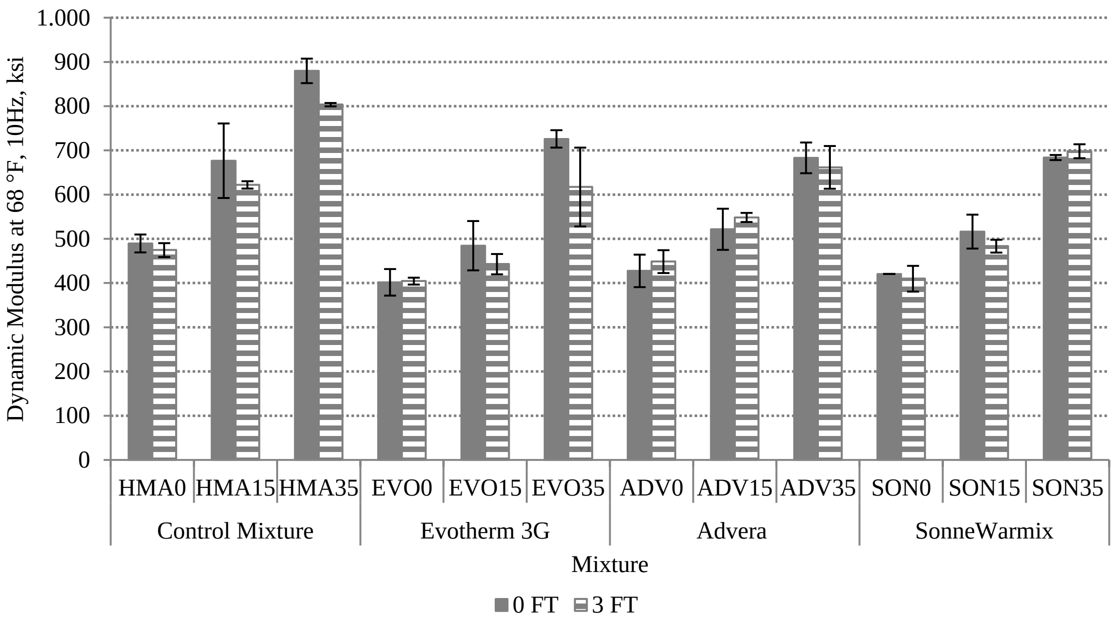

Figure 17: Dynamic Modulus |E*| at 68°F, 10 Hz, after 0 and 3 Freeze/Thaw Cycles (Bars represent 95% Confidence Interval)

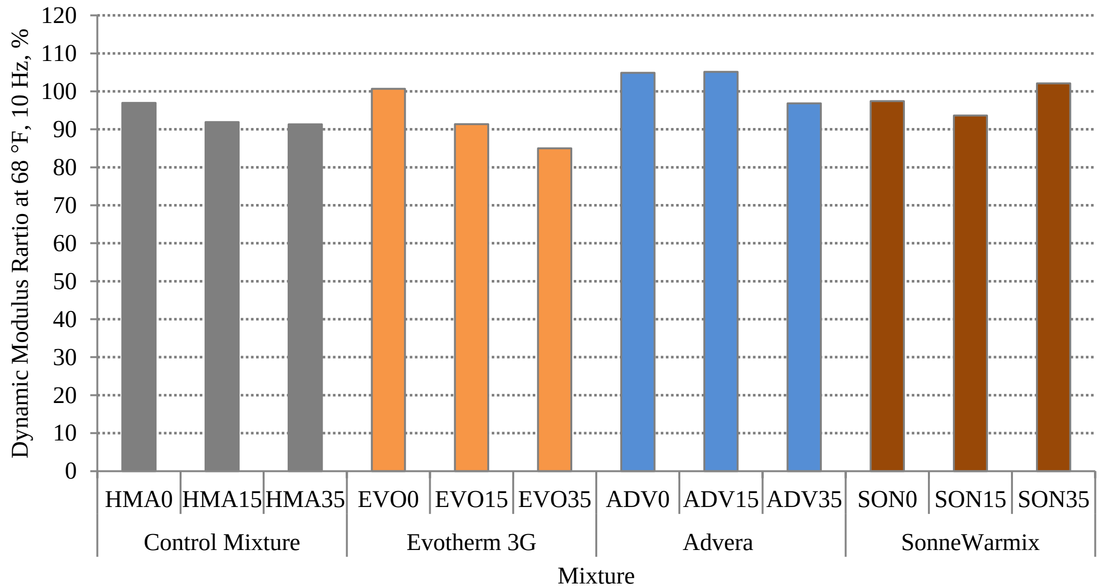

Figure 18: Dynamic Modulus |E*| Ratio at 68°F, 10 Hz 3 Freeze/Thaw Cycles

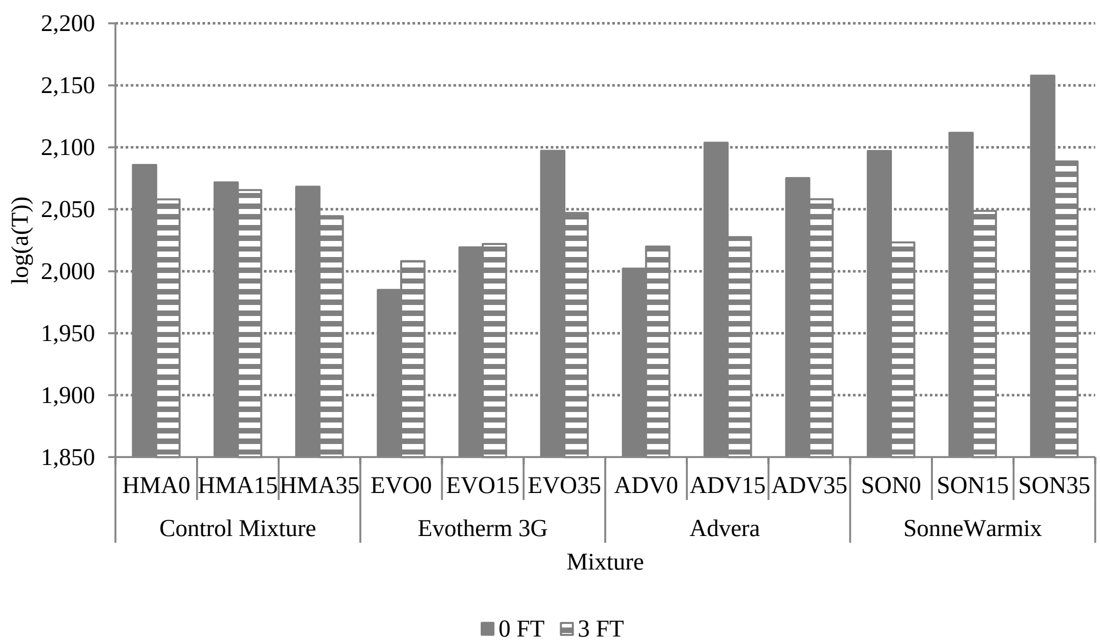

Figure 19: Log of Shift Factors 4C/21.1C used in Dynamic Modulus Master Curves

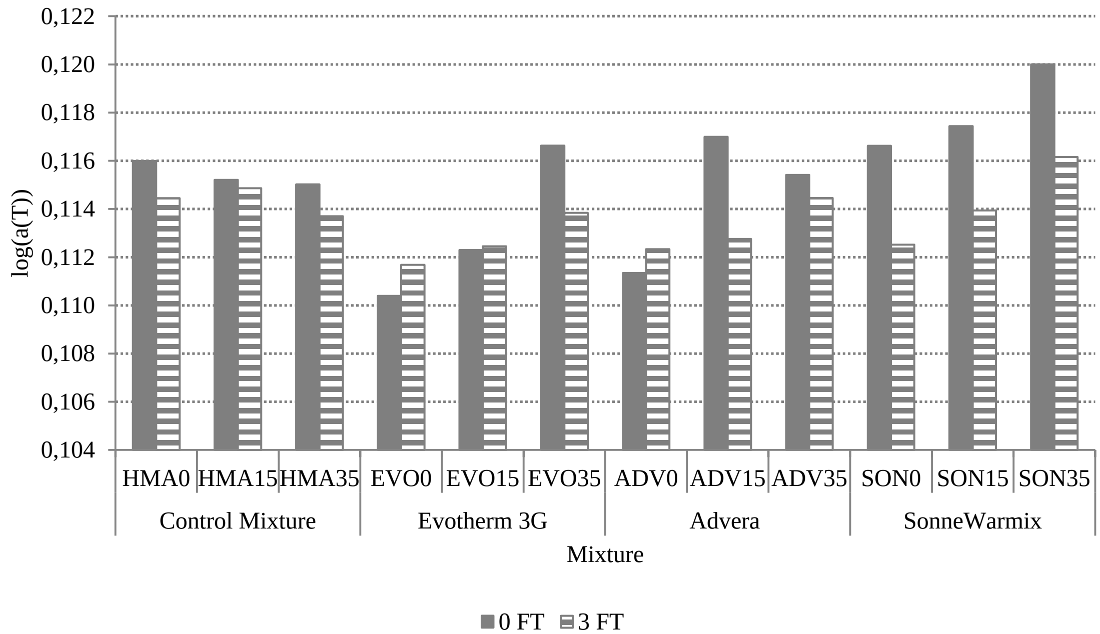

Figure 20: Log of Shift Factors 20C/21.1C used in Dynamic Modulus Master Curves

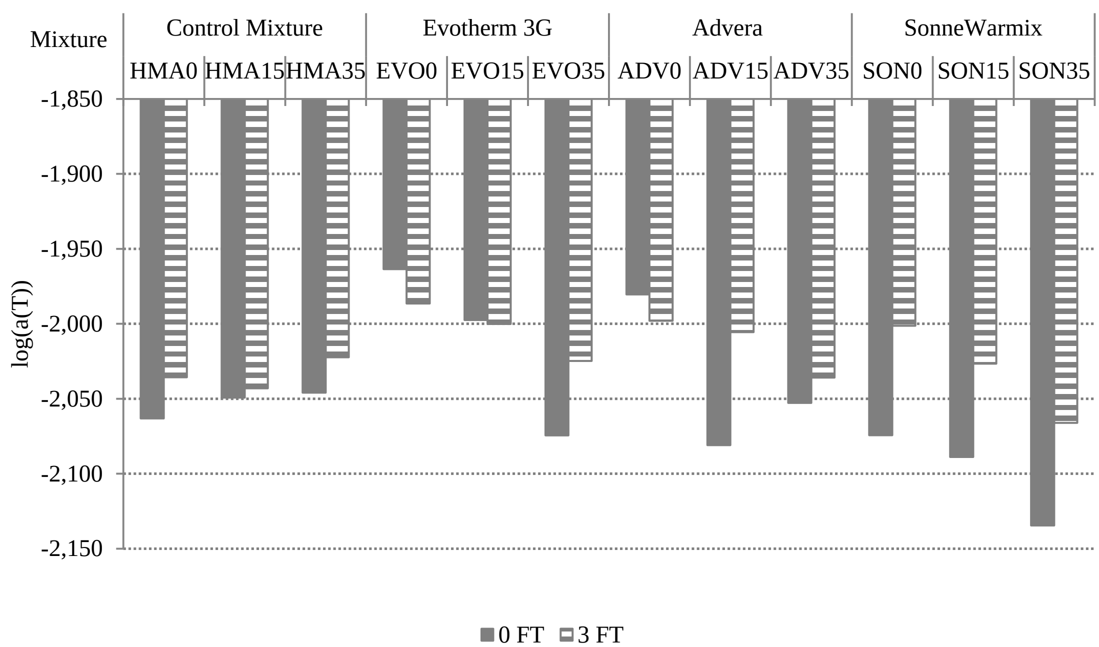

Figure 21: Log of Shift Factors 40C/21.1C used in Dynamic Modulus Master Curves

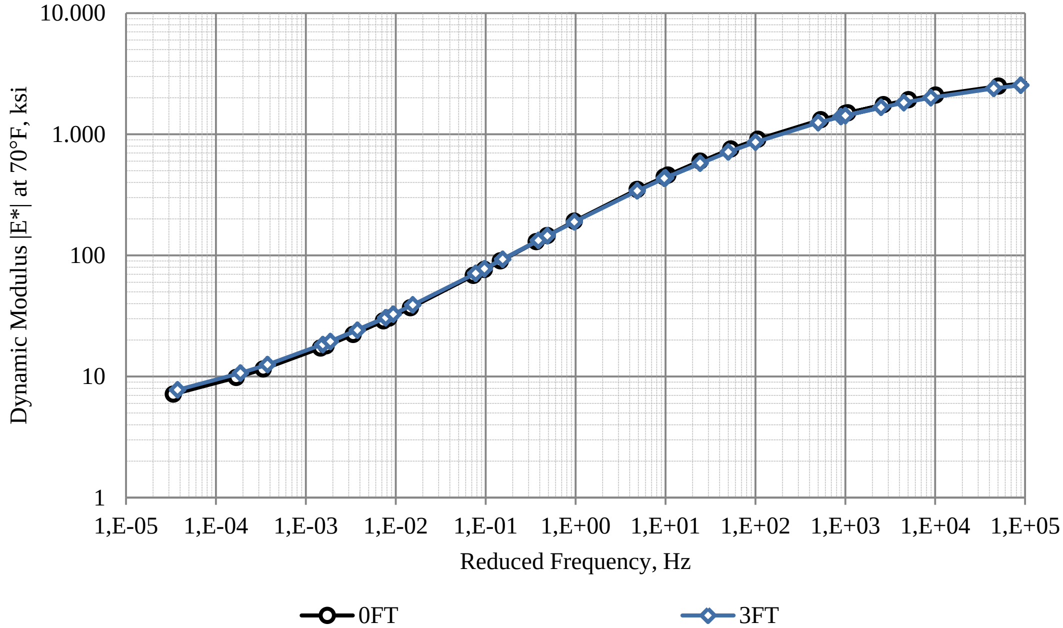

Figure 22: HMA0 Dynamic Modulus |E*| Master Curves after 0 and 1 Freeze/Thaw cycles

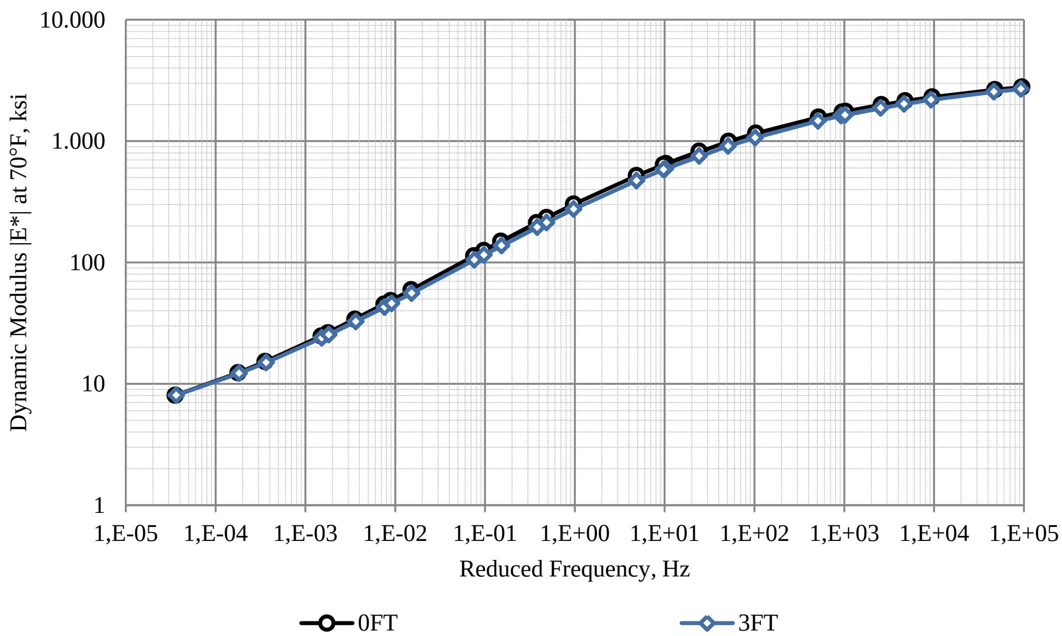

Figure 23: HMA15 Dynamic Modulus |E*| Master Curves after 0 and 1 Freeze/Thaw cycles

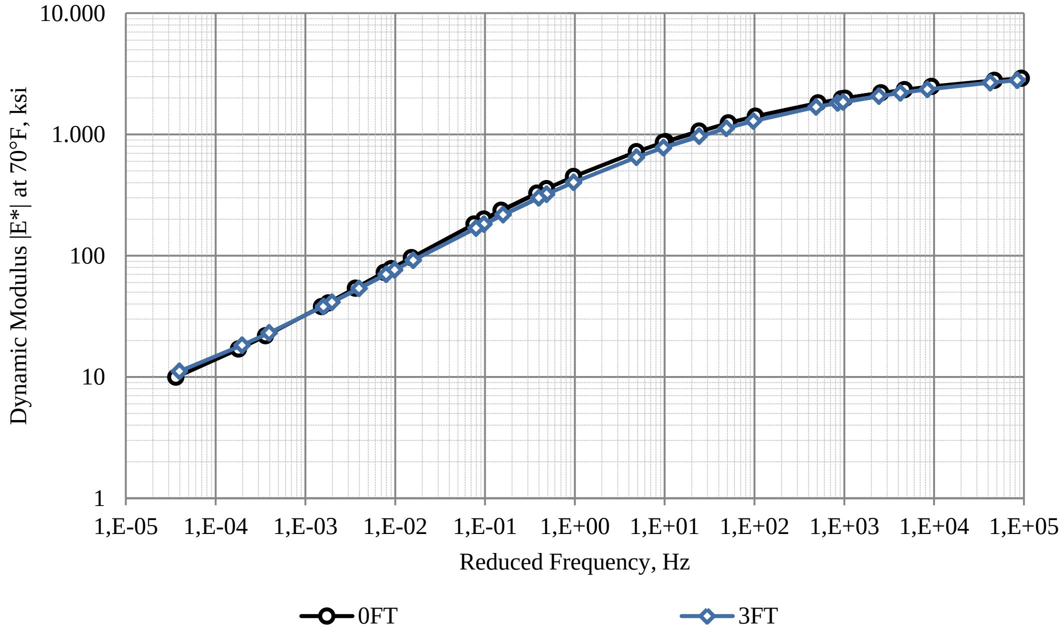

Figure 24: HMA35 Dynamic Modulus |E*| Master Curves after 0 and 1 Freeze/Thaw cycles

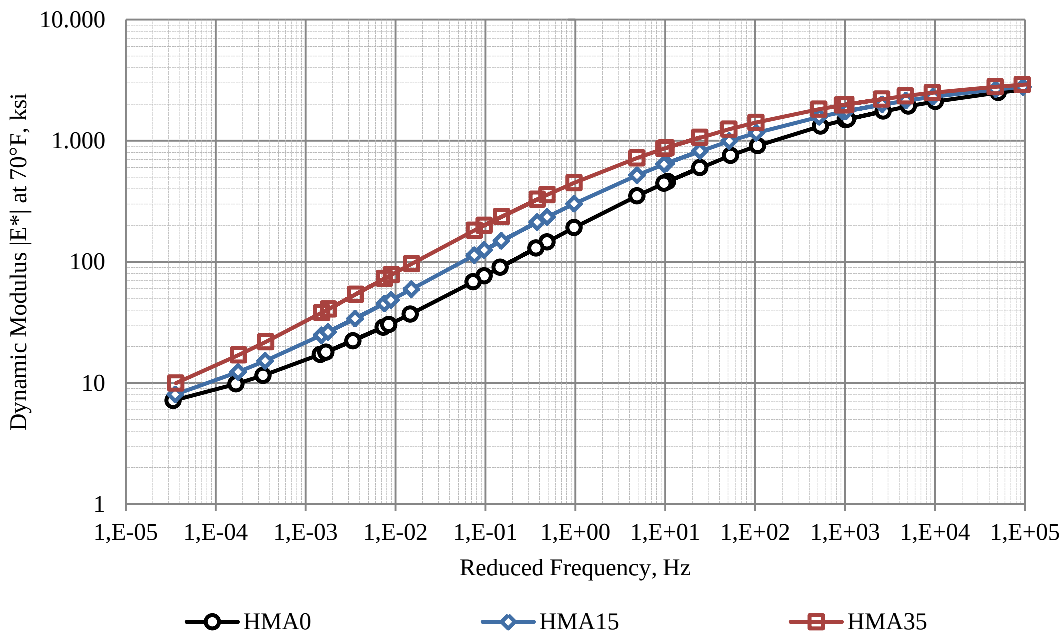

Figure 25: Dynamic Modulus |E*| Master Curves with 0 and two binder replacements after 0 Freeze/Thaw cycles for HMA-mixtures

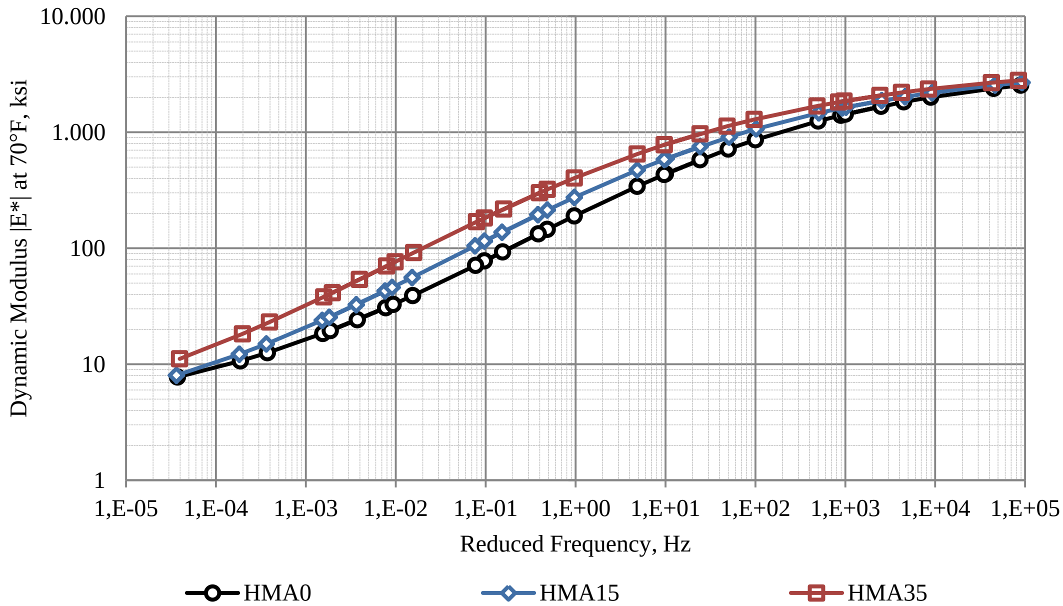

Figure 26: Dynamic Modulus |E*| Master Curves with 0 and two binder replacements after 3 Freeze/Thaw cycles for HMA-mixtures

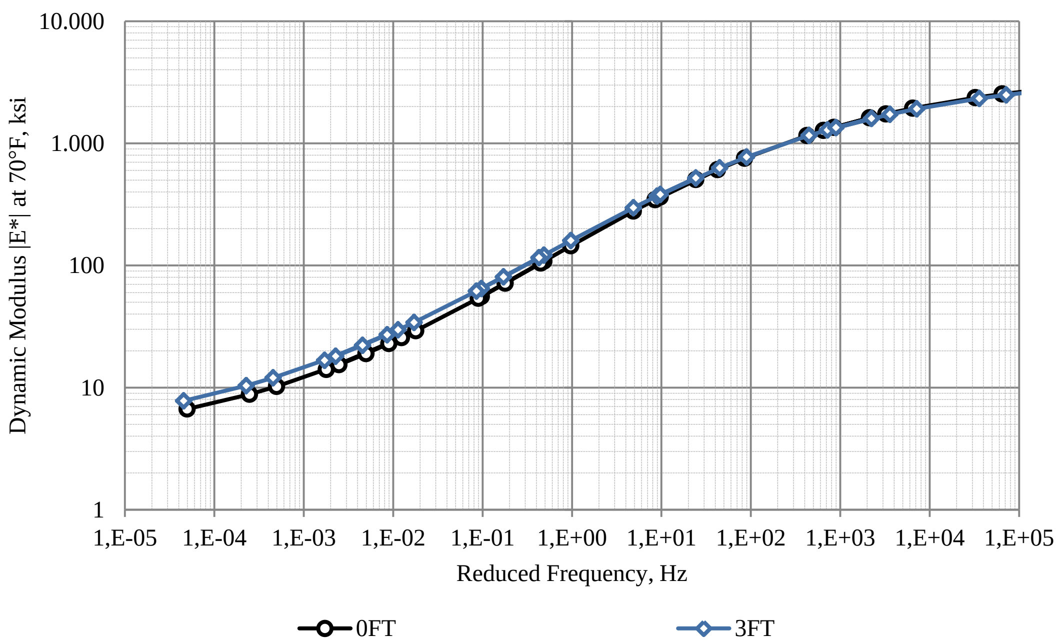

Figure 27: EVO0 Dynamic Modulus |E*| Master Curves after 0 and 1 Freeze/Thaw cycles

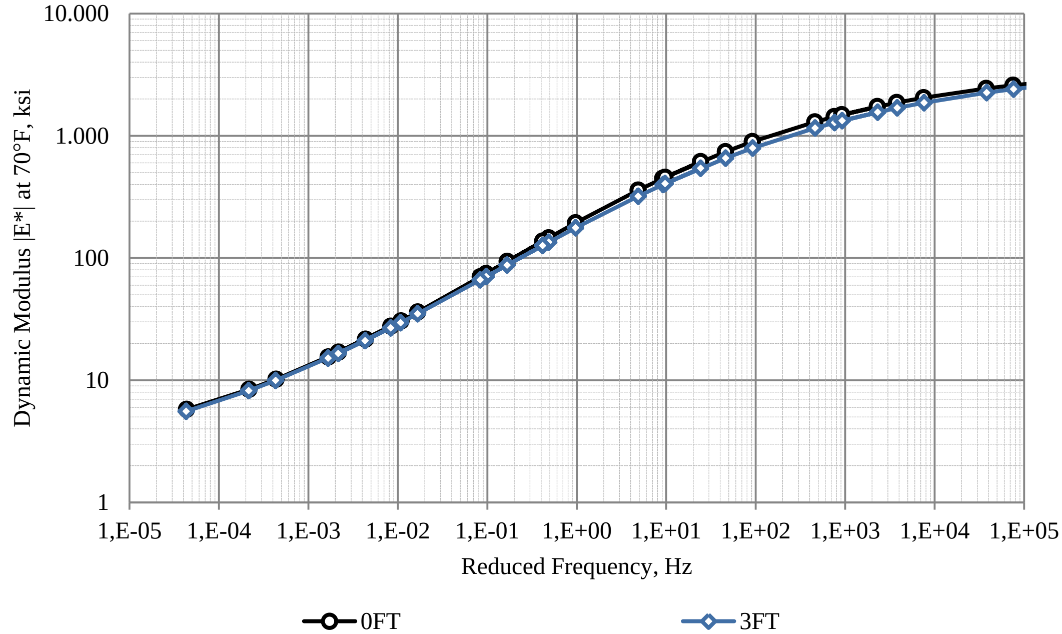

Figure 28: EVO15 Dynamic Modulus |E*| Master Curves after 0 and 1 Freeze/Thaw cycles

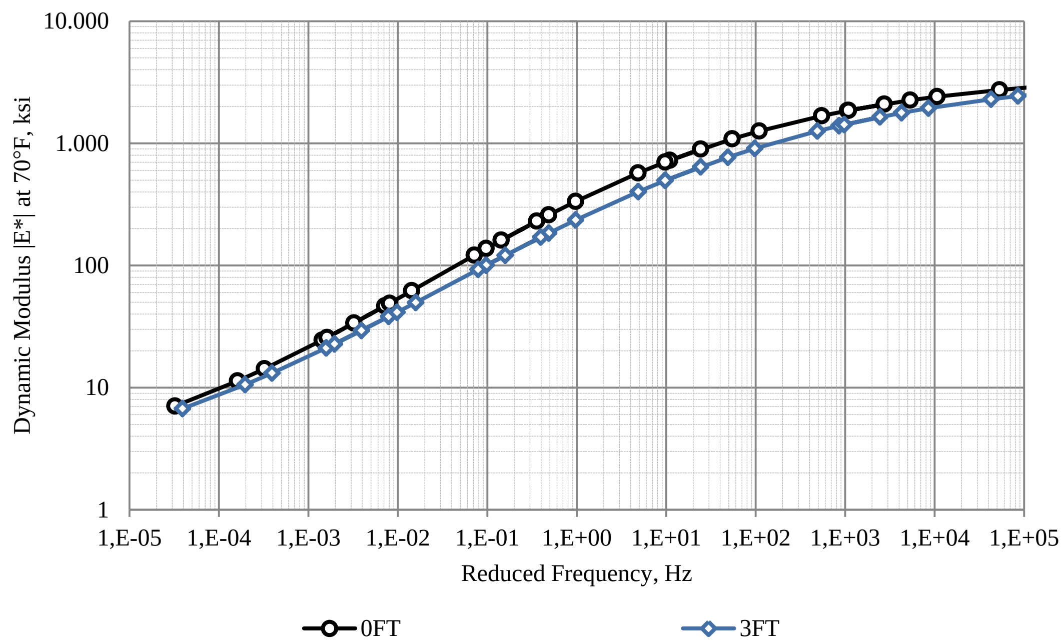

Figure 29: EVO35 Dynamic Modulus |E*| Master Curves after 0 and 1 Freeze/Thaw cycles

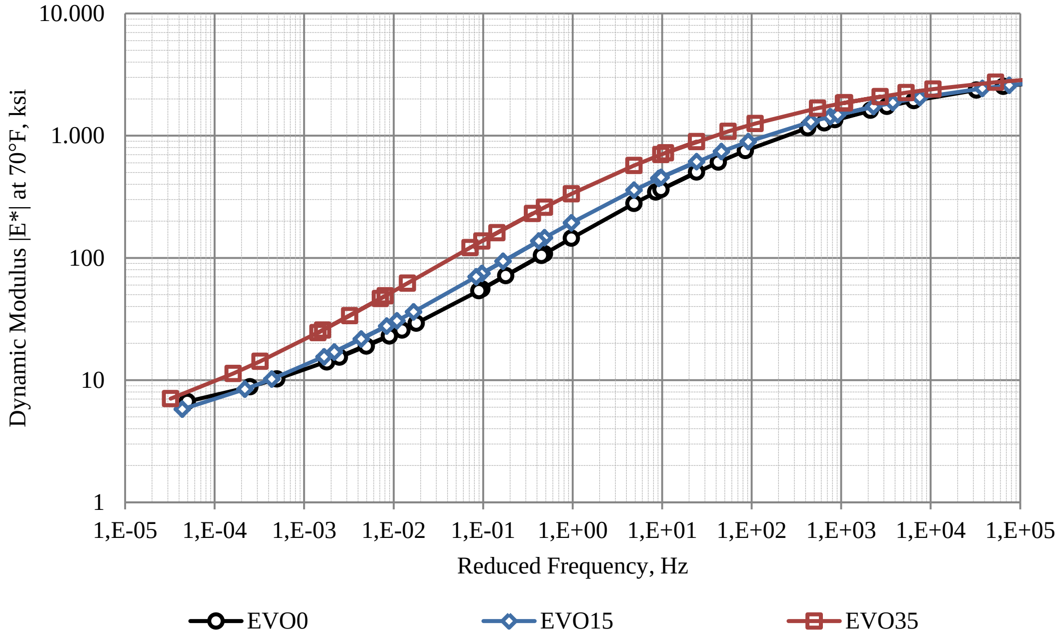

Figure 30: Dynamic Modulus |E*| Master Curves with 0 and two binder replacements after 0 Freeze/Thaw cycles for EVO-mixtures

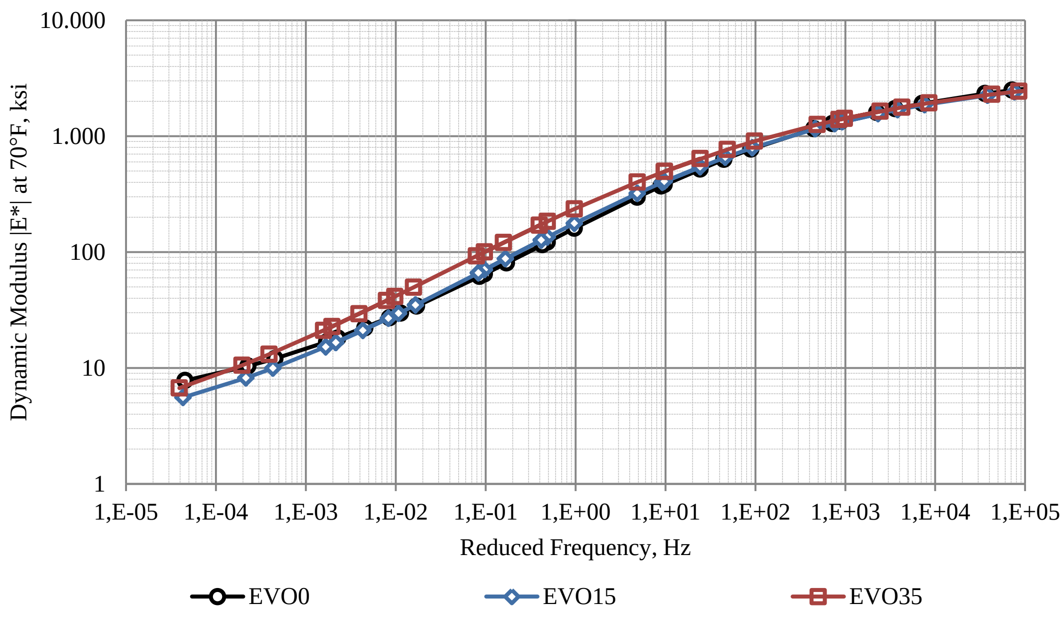

Figure 31: Dynamic Modulus |E*| Master Curves with 0 and two binder replacements after 3 Freeze/Thaw cycles for EVO-mixtures

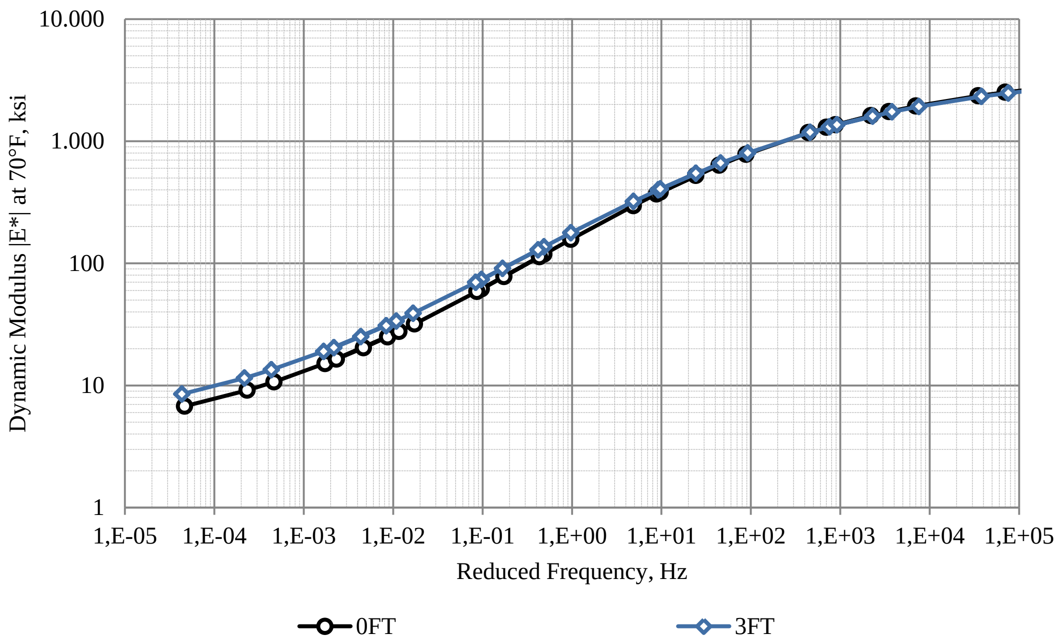

Figure 32: ADV0 Dynamic Modulus |E*| Master Curves after 0 and 1 Freeze/Thaw cycles

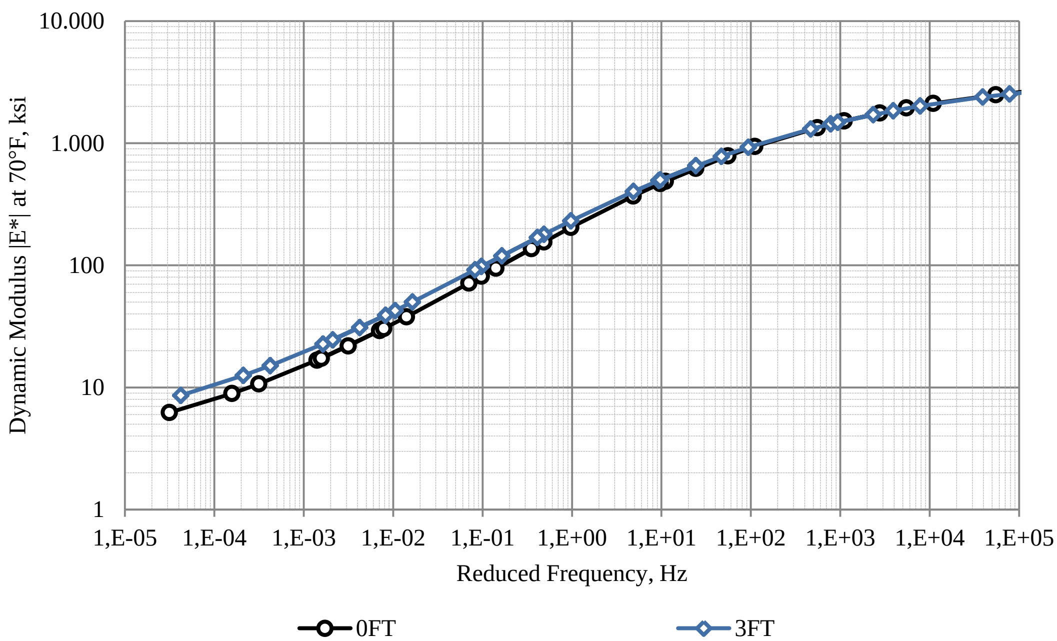

Figure 33: ADV15 Dynamic Modulus |E*| Master Curves after 0 and 1 Freeze/Thaw cycles

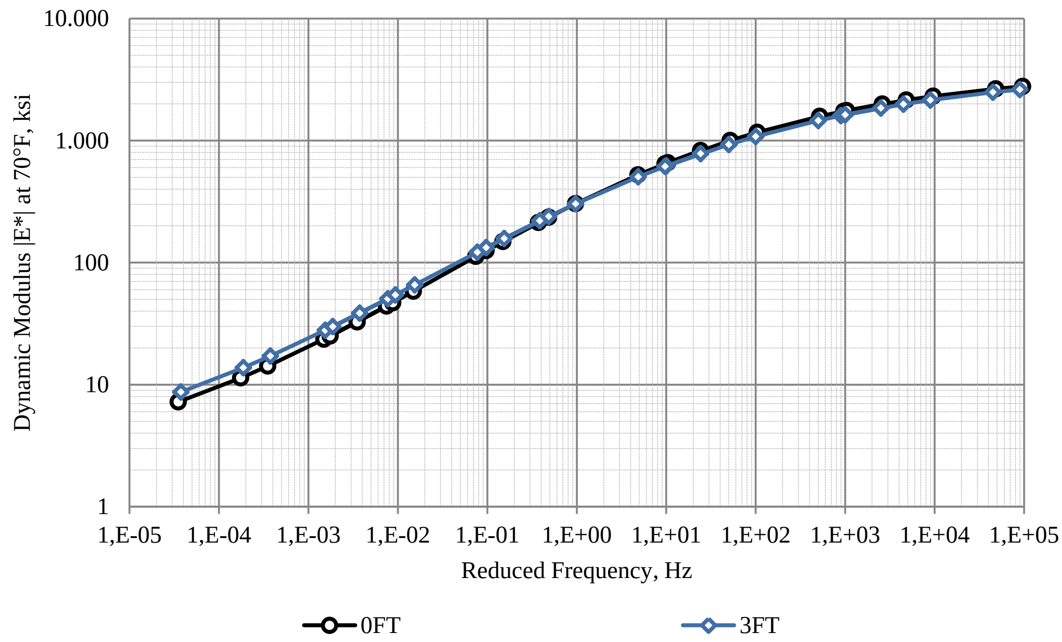

Figure 34: ADV35 Dynamic Modulus |E*| Master Curves after 0 and 1 Freeze/Thaw cycles

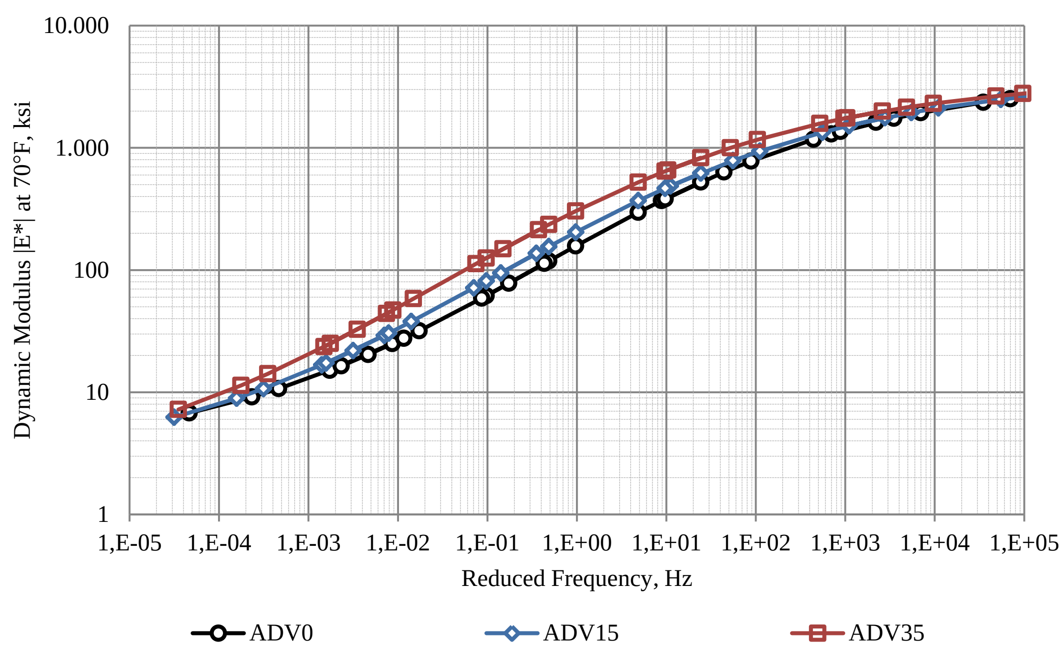

Figure 35: Dynamic Modulus |E*| Master Curves with 0 and two binder replacements after 0 Freeze/Thaw cycles for ADV-mixtures

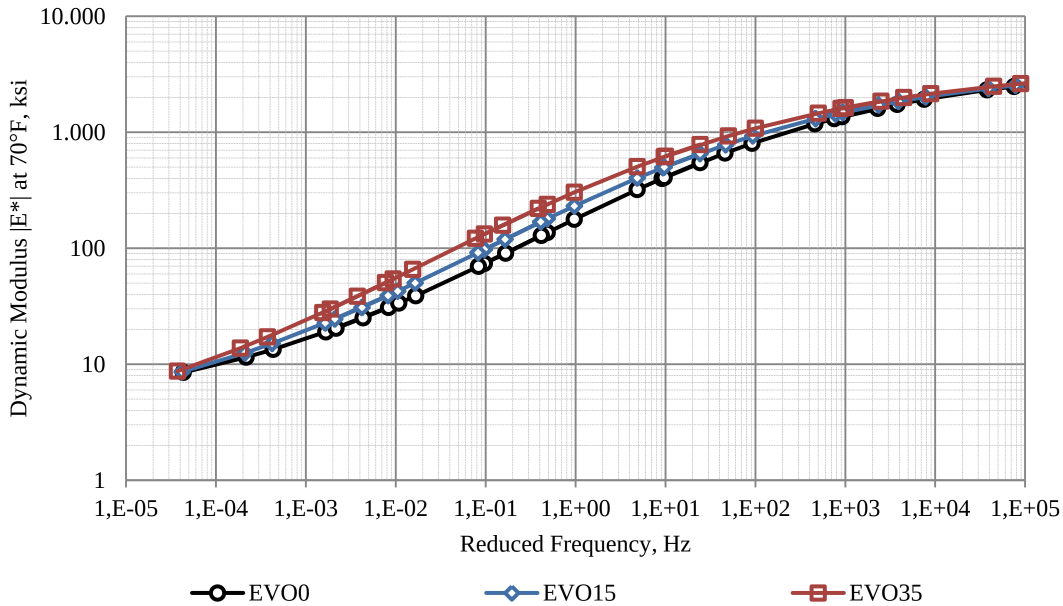

Figure 36: Dynamic Modulus |E*| Master Curves with 0 and two binder replacements after 3 Freeze/Thaw cycles for ADV-mixtures

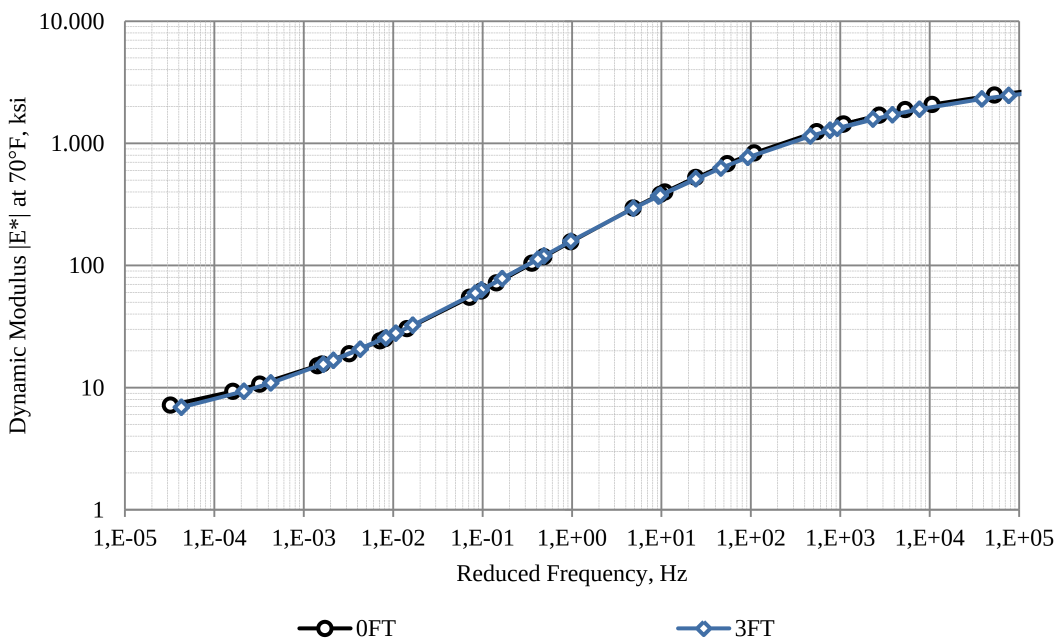

Figure 37: SON0 Dynamic Modulus |E*| Master Curves after 0 and 1 Freeze/Thaw cycles

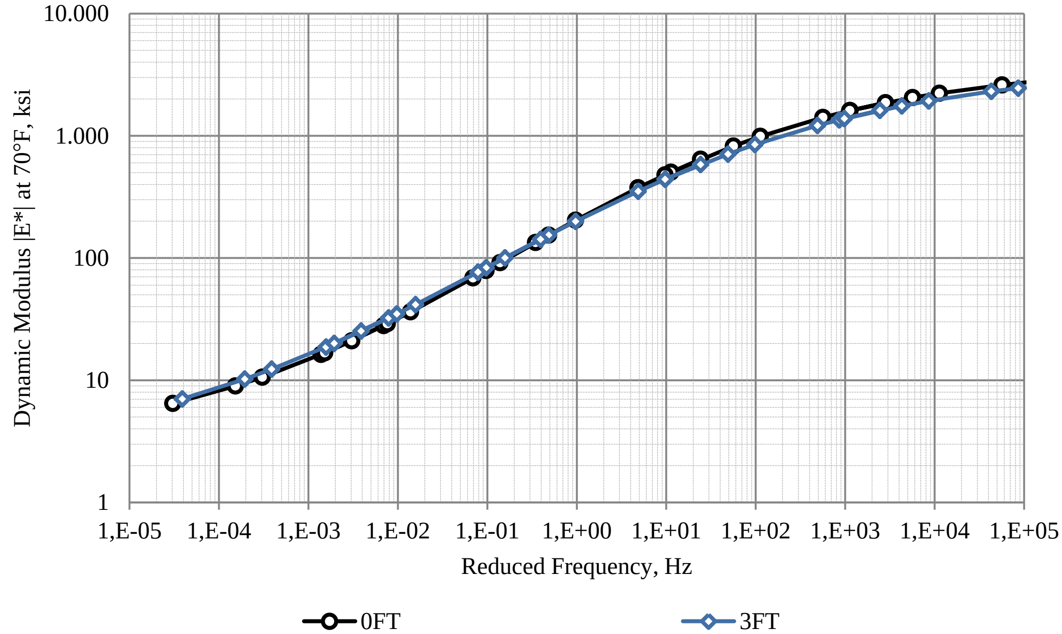

Figure 38: SON15 Dynamic Modulus |E*| Master Curves after 0 and 3 Freeze/Thaw cycles

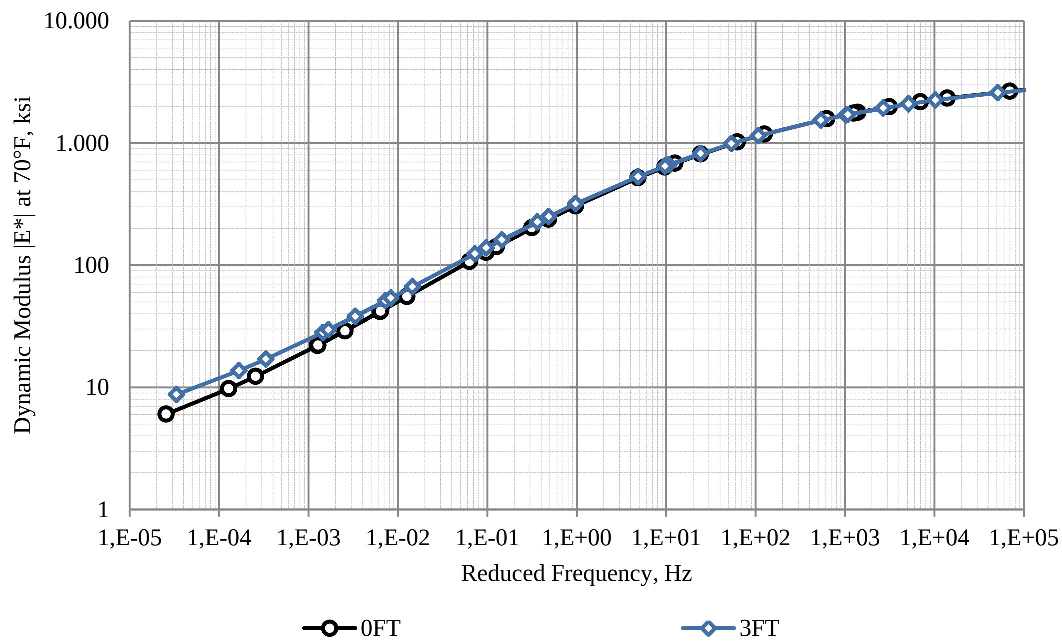

Figure 39: Dynamic Modulus |E*| Master Curves after 0 and 3 Freeze/Thaw cycles for SON35

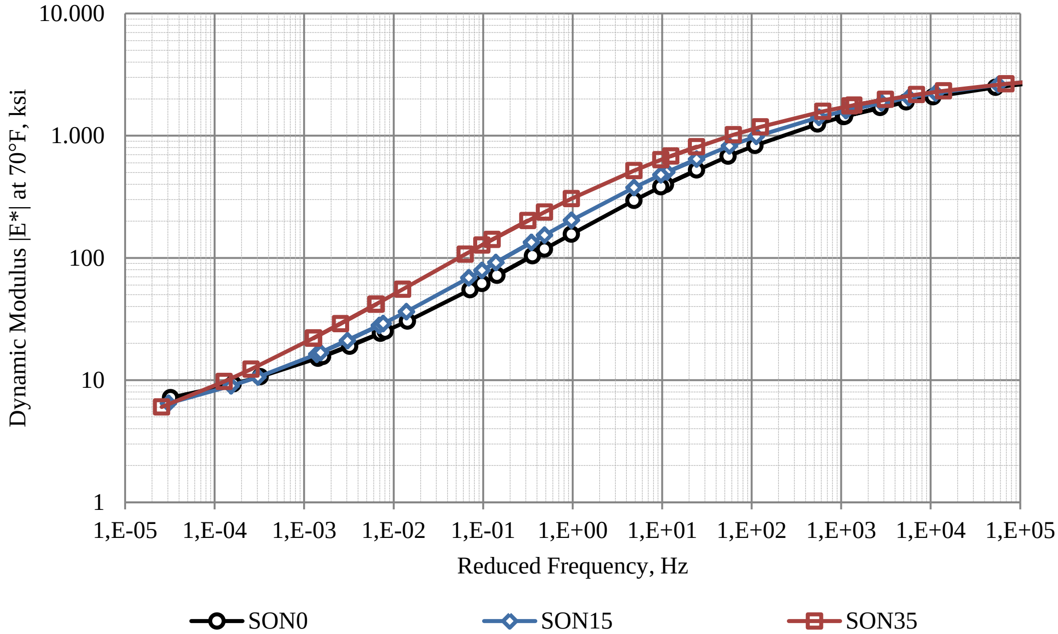

Figure 40: Dynamic Modulus |E*| Master Curves with 0 and two binder replacements after 0 Freeze/Thaw cycles for SON-Mixtures

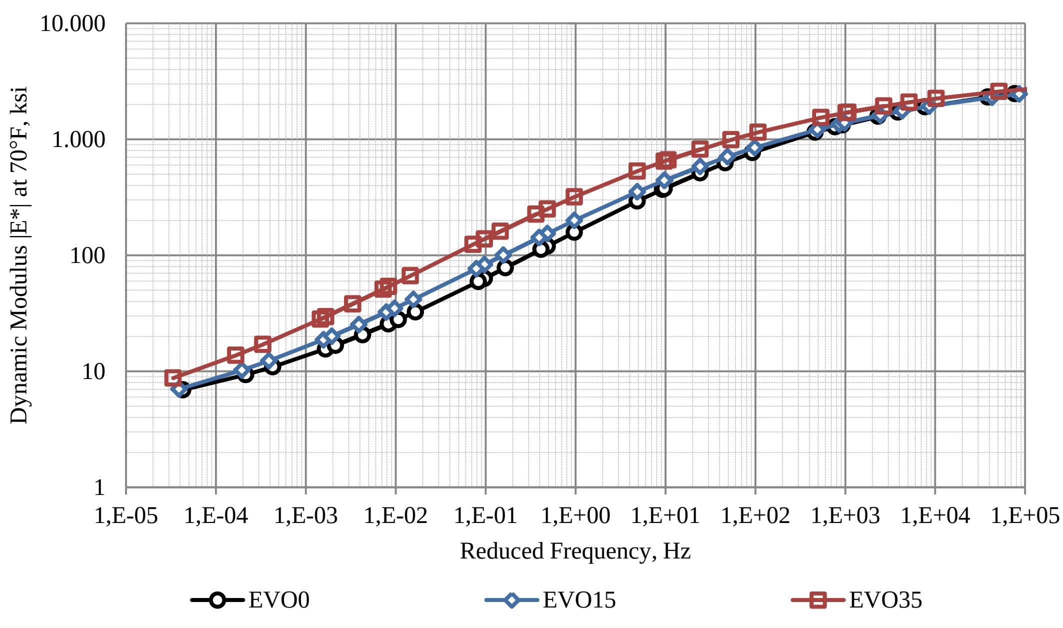

Figure 41: Dynamic Modulus |E*| Master Curves with 0 and two binder replacements after 0 Freeze/Thaw cycles for SON-Mixtures

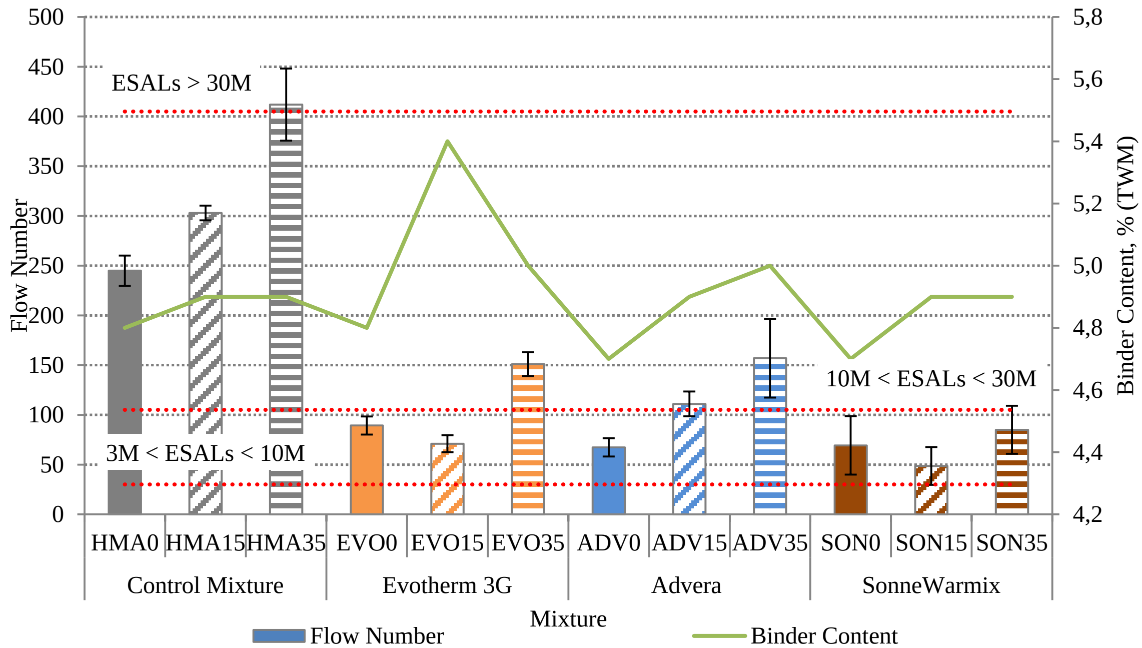

Figure 42: Flow Number of the various mixtures. (Bars represent 95% Confidence Interval)

APPENDIX: MORTAR EXPERIMENT DRAFT STANDARD

Standard Method of Test for

Estimating Effect of RAP and RAS on Blended Binder Performance Grade without Binder Extraction

AASHTO Designation: T XXX-12

1. SCOPE

1.1 This test method presents the procedure to estimate the effect of recycled asphalt pavement (RAP) or recycled asphalt shingles (RAS) on binder performance grade. The procedure measures the Superpave PG properties of asphalt binders and mortars in order to estimate the performance properties of the blended binder. Due to the use of mortars, extraction of binders from recycled materials is not required. In addition, the procedure provides blended binder performance properties at two levels of binder replacement and therefore can be used to construct a blending chart to estimate the change in performance grade as a function of percent binder replacement.

1.2 The values stated in SI units are to be regarded as the standard.

1.3 *This standard does not purport to address all of the safety problems, if any, associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. REFERENCED DOCUMENTS

2.1 *AASHTO Standards:*

AASHTO M320 – Standard Specification for Performance Graded Asphalt Binder

AASHTO T240 – Effect of Heat and Air on a Moving Film of Asphalt Binder (Rolling Thin Film Oven Test)

AASHTO R28 – Accelerated Aging of Asphalt Binder Using a Pressure Aging Vessel

AASHTO R29 – Practice for Grading or Verifying the Performance Grade of an Asphalt Binder

2.2 *ASTM Standards:*

ASTM D 7175-08 – Standard Test Method for Determine the Rheological Properties of Asphalt Binder Using the Dynamic Shear Rheometer.

ASTM D7643-10 – Determining the Continuous Grading Temperatures and Continuous Grades for PG Graded Asphalt Binders

ASTM D 6648-08 – Standard Test Method for Determining the Flexural Creep Stiffness of Asphalt Binder Using the Bending Beam Rheometer.

ASTM D6307 – Standard Test Method for Asphalt Contentof Hot Mix Asphalt by Ignition Method.

ASTM D8 – Standard Terminology Relating to Materials for Roads and Pavements

3. TERMINOLOGY

3.1 *General Definitions:*

3.1.1 General definitions of terms used in this practice are found in Terminology ASTM D 8 determined from common English usage, or combinations of both.

3.2 Procedural Definitions

3.2.1 *Binder Performance Grade Change Rate:* Change in blended binder continuous grade due to increasing binder replacement [°C/% RAP(S) binder replacement].

3.2.2 *Binder Replacement*: The percentage by weight of recycled binder present to the total weight of binder used to prepare the mortar.

3.2.3 *Blended Binder:* The effective asphalt binder (virgin binder blended with RAP(S) binder) in the mortar material. Blending does not need to be present to test the mortar materials in the proposed procedure. Complete blending may not occur if the mortar is prepared at low mixing temperatures, or the RAP(S) material is heavily oxidized.

3.2.4 *Burned Aggregates*: Remaining aggregates from RAP or RAS material after burning in the ignition oven according to ASTM D6307-05. This material is free from binder and consists only of the non-bituminous components of RAP or RAS.

3.2.5 *Asphalt Binder Continuous Grade:* The temperature at which an asphalt binder fails a given Superpave performance grading limit. To determine the continuous grade it is required to test at temperatures that correspond to the material passing and failing the specification limit. The performance vs. temperature relationship is used to determine the exact temperature at which the material fails by use of interpolation. Continuous grading is applicable to high, intermediate, and low grading temperatures.

3.2.6 *Virgin Asphalt Binder:* Conventional asphalt binder material used in construction of asphalt pavements. In application to this procedure virgin binder is subjected to standard SuperPave grading specified in AASHTO M320 and used for preparation and evaluation of mortar properties. Virgin binder can be modified or unmodified and used at various levels of artificial aging, as required by the test procedure.

3.2.7 *Mortar*: Laboratory produced mixture of virgin asphalt binder and R_{100} aggregates.

3.2.8 *R100 Aggregates*: Aggregates and other non-bituminous components from either RAP or RAS material that are passing sieve #50 (300μm) and retained on #100 (150μm).

Aggregates that do not meet this size requirement are discarded. Both aggregates sampled directly from RAP/RAS source and sampled after burning in ignition oven are used in this test procedure.

3.2.9 *RAP:* Recycled (Reclaimed) Asphalt Pavement.

3.2.10 *RAS*: Recycled (Reclaimed) Asphalt Shingles.

3.2.11 *Total Binder Content*: Weight percentage of binder (virgin binder + RAP/RAS binder) in mortar material.

4. SUMMARY OF METHOD

4.1 Evaluation of the impacts of RAP/RAS on performance requires testing of three different materials using SuperPave methods. The materials required include the virgin binder and two void-less mortar samples prepared with the virgin binder and a single sized aggregate gradation. The two mortar samples are prepared with identical gradation and total asphalt content using aggregates from recycled materials both before and after burning in the ignition oven. As a result, any difference in performance properties between the two mortars is attributed to the presence of recycled binder. The effect of the recycled binder on mortar performance is applied to estimate the performance of the blended binder.

5. SIGNIFICANCE AND USE

5.1 This test method provides an estimate of the impacts of binder replacement by recycled asphalt materials (RAP and/or RAS) on blended binder performance grade. The method is also used to define the change in binder continuous grade as a function of percent binder replacement. This relationship can be used to define the maximum allowable binder replacement before a change in virgin asphalt grade is necessary. The procedure uses mortars and thus eliminates the need, and variability associated with, chemical extraction and recovery. This test procedure is performance based and therefore the analysis is blind to RAP/RAS source, virgin binder source, and the use of binder modification.

6. SAMPLE PREPARATION

6.1 Select virgin binder, RAP, and RAS sources representative of materials used in the field for a user defined climate and geographical area.

6.2 Dry and sieve all RAP and/or RAS material. Arrange sieves to collect R_{100} material, defined as materials passing sieve #50 (300 μm) and retained on sieve #100 (150 μm). Discard all material passing the #100 sieve (P_{100}). The test procedure requires at least 500 g of R_{100} material. After sufficient material is collected split the R_{100} material.

6.3 Place at least 250 g of RAP or RAS material in the ignition oven and follow ASTM D6307-05 to determine the binder content of the R_{100} material. Save the aggregates that remain after burning for use in preparation of mortar samples.

6.4 The procedure requires preparation of two mortar types, these mortars are defined below. The quantity of mortar required and aging condition of the asphalt binder used in preparation of the mortar is user defined and depends on the performance property of interest. Guidelines are provided in Table 1.

6.4.1 *RAP/RAS Mortar:* Consists of R_{100} RAP or RAS material from (6.2) combined with virgin binder at a user-selected level of aging. Adjust binder content to ensure that a level of workability is achieved such that DSR samples and BBR beams can be cast free of voids. A minimum total binder content of 30% by weight is recommended as a guideline.

6.4.2 *Aggregate Mortar:* Consists of R_{100} burned aggregates from (6.3) mixed with virgin asphalt binder at a user defined aging condition and at the same total binder content as the RAP/RAS mortar prepared in (6.4. 1).

6.5 Select the aging condition of the virgin asphalt and quantity of the mortar based on the aging condition at and type of test required for performance evaluation.

Note 1 – Research has indicated the blending of virgin and RAP aged binder is a diffusion process in which time and conditioning temperature can greatly affect the amount (degree) of binder blending. To ensure sufficient blending, mortar samples should be conditioned at 135 °C for two hours. It is best to determine the field production temperature and conditioning time conditions in order to simulate the actual blending in the laboratory. If possible, use these conditions in the laboratory.

6.5.1 *Un-aged mortar performance:* Prepare the mortar with un-aged virgin asphalt binder.

6.5.2 *Short and Long Term aged mortar performance:* Short-term age virgin asphalt binder in the RTFO according to AASHTO T240 and use the aged binder to prepare the mortar.

6.5.3 *Recommended mortar quantities:* Un-aged and short-term aged properties require at least 60 grams of mortar for DSR testing. Evaluation of long term-aged properties requires at least 200 g of mortar to allow for both the DSR and BBR testing. Example calculations for preparation of the mortar are provided in Appendix X1.

6.6 Long term aging requires aging of mortars in the pressure aging vessel for 24 hours under the conditions specified in AASHTO R-28 prior to testing. Place sufficient mortar in each PAV pan to ensure 50 grams total of binder is present in the pan. For example, if the total binder content of the mortar is 40 percent, the required amount of mortar in each PAV pan will be 50g / (0.40 binder content) = 125 g mortar.

6.7 Short and long term age virgin binders as required to measure SuperPave PG properties. Conduct each specification test at a minimum of two temperatures.

The test samples required for a complete analysis procedure are summarized in Table 1.

Table 1 – Required Test Specimens for Complete Performance Grade Analysis

Performance	Low Temperature: BBR	Intermediate Temperature: DSR	High Temperature: DSR	
Asphalt Binder	**Same as PG Grading**			
RAP/RAS Mortar	PAV Aged: RTFO Binder + **RAP/RAS**	PAV Aged: RTFO Binder + **RAP/RAS**	RTFO Binder + **RAP/RAS**	Original Binder **+RAP/RAS**
Aggregate Mortar	PAV Aged: RTFO Binder + RAP/RAS **Aggregate**	PAV Aged: RTFO Binder + RAP/RAS **Aggregate**	RTFO Binder + RAP/RAS **Aggregate**	Original Binder + RAP/RAS **Aggregate**

7. TESTING PROCEDURE

7.1 *Virgin Asphalt Binder:* Conduct tests specified in AASHTO M320 to determine the SuperPave performance properties of the asphalt binder. Take measurements at two test temperatures. Select testing temperatures for a given performance property based recommendations provided in Table 3.

7.2 *RAP/RAS and Aggregate Mortars:* Measurement of the performance properties of mortars requires revisions to the AASHTO M320 procedure as detailed below. Evaluate each performance property at the same test temperatures as used for the virgin binder in Step 7.1.

7.2.1 *High Temperature Performance:* Prepare separate aggregate and RAP/RAS mortars with un-aged and RTFO aged virgin binders as detailed in Table 1. Test in the Dynamic Shear Rheometer (DSR) at a gap of 2mm at the prescribed test temperatures. All other testing conditions specified in AASHTO M320 remain unchanged.

7.2.2 *Intermediate Temperature Performance:* Prepare separate and Aggregate and RAP/RAS mortars with RTFO aged virgin binder. Long term age the mortars in the PAV according to (6.6). Evaluate the performance properties using the procedure in AASHTO M320 at the same test temperatures as the virgin binder tested in (7.1).

7.2.3 *Low Temperature Performance:* Prepare separate aggregate and RAP/RAS mortars with RTFO aged virgin binder. Long term age the mortars in the PAV according to (6.6). Prepare beams for testing in the bending beam rheometer (BBR) and test at the temperatures used for the virgin binder. Adjust test load in BBR based on test temperature selected as shown in Table 2. All other testing conditions as specified in AASHTO M320 remain unchanged.

Table 2 – Bending Beam Rheometer Test Loads in mN.

Test Temperature (°C)	PAV Binder	PAV Mortar
0	980	980
-6	980	1980
-12	980	2980
-18	980	3980
-24	980	4980

7.3 The testing procedures, evaluation parameters, recommended temperatures and deviations from AASHTO M320 are summarized in Table 3.

Table 3 – Summary of Test Procedure and Deviations from AASHTO M320 for Mortar Grading

<table>
<tr><th rowspan="2">Recommended Test Temperatures</th><th rowspan="2">Device and Test Parameters</th><th colspan="3">Deviations from AASHTO M320</th></tr>
<tr><th>Virgin Asphalt</th><th>Aggregate Mortar</th><th>RAP/RAS Mortar</th></tr>
<tr><td>Low Temperature PG LT+10°C, PG LT+16°C</td><td>Bending Beam Rheometer (BBR), S(60), m(60)</td><td rowspan="4">None</td><td colspan="2">Adjust test load for different test temperatures.</td></tr>
<tr><td>Intermediate Temperature PG IT, PG IT+3°C</td><td>Dynamic Shear Rheometer (DSR), G*sinδ</td><td colspan="2">None</td></tr>
<tr><td>High Temperature – Short Term Aged PG HT, PG HT+6°C</td><td>Dynamic Shear Rheometer (DSR), G*/sinδ</td><td colspan="2" rowspan="2">Increase DSR testing gap from 1mm to 2mm</td></tr>
<tr><td>High Temperature – Un-aged PG HT, PG HT+6°C</td><td>Dynamic Shear Rheometer (DSR), G*/sinδ</td></tr>
</table>

8. CALCULATION AND INTERPRETATION OF RESULTS

8.1 *Data Analysis*. The test data available after completion of Section 7 provides sufficient information to estimate the effect of recycled binders on virgin binder performance properties and to create a blending chart that establishes the relationship between %Recycled Binder replacement vs. change in binder continuous grade.

8.2 *Estimating the Impact of RAS/RAP on Virgin Binder Continuous Grade*

8.2.1 Determine the continuous grade of the virgin asphalt binder using the analysis procedure specified in ASTM D7643.

8.2.2 Compare the performance of the RAP/RAS and aggregate mortars at each testing temperature to estimate the performance properties of the recycled asphalt binder. The only difference between RAP/RAS aggregate mortars is the presence of the recycled binder, therefore any differences in performance are assumed to be due to the recycled binder. The difference in performance between the RAP/RAS and aggregate mortars for a given performance property at a given test temperature is defined as (δ_{Tx}). It is necessary to measure δ_{Tx} at both test temperatures.

Eq (1)

$$\delta_{\mathrm{Tx}} = \frac{\text{Property RAP/RAS Mortar}}{\text{Property Aggregate Mortar}}$$

Note: To maintain agreement with ASTM D7346 δ_{Tx} values must be calculated using a logarithmic scale for all properties except the BBR m-value. Use an arithmetic scale to calculate δ_{Tx} for the m-value.

8.2.3 Calculate the average change in performance across both testing temperatures (δ_{RAP}) using Equation 2.

Eq (2)

$$\delta_{RAP} = \frac{(\delta_{T1}) + (\delta_{T2})}{2}$$

Where,

δ_{RAP} = Average change in performance property due to the presence of recycled binder in the RAP/RAS mortar.

δ_{T1} = Change in performance property due to the presence recycled binder in the RAS/RAP mortar at PG testing temperature 1.

δ_{T2} = Change in performance property due to the presence of recycled binder in the RAS/RAP mortar at PG testing temperature 2.

8.2.4 Multiply the RAP shift factor (δ_{RAP}) by the virgin binder performance property of interest at each test temperature to determine the effect of blending of the reclaimed RAP or RAS binder with virgin binder. Calculate the blended binder continuous grade using the procedure detailed in ASTM D7643.

8.3 *Estimating the relationship between %Recycled Binder Replacement and Change in Blended Binder Continuous Grade*

8.3.1 The analysis method provides two points of reference for estimating the rate of change in continuous grade due to replacement of virgin binder with recycled binder, the continuous grade of the virgin binder (0% binder replacement) and the estimated continuous grade of the recycled binder blended with the virgin binder (x% binder replacement). The %binder replacement in the blended binder depends on the asphalt content of the recycled material used and the total binder content of the RAP/RAS mortar. Based on these points of reference the rate of change in continuous grade is represented by the slope of the line, provided in Equation 3.

Eq (3)

$$\textbf{Rate of Change in C.G.} = \frac{(\text{Est.Blended Binder C.G} - \text{Virgin Binder C.G.})}{\text{Recycled PBR}}$$

Where,

Rate of Change in C.G. = Rate of virgin binder grade change per percent binder replaced. [°C/% replacement]

Estimated Blended Binder C.G. = Estimated blended binder continuous grade [°C]
Virgin Binder C.G. = Virgin binder continuous grade [°C]
Recycled PBR: Percent binder replacement [%]

8.3.2 Equation 3 is applicable to any performance property in the high, intermediate, and low SuperPave grading temperature regimes. In addition the linear relationship is applicable to any quantity of recycled binder replacement through use of interpolation or extrapolation.

8.4 An example of this calculation is provided in Appendix X2.

9. REPORT

9.1 *Virgin Binder Properties*

9.1.1 SuperPave performance properties as measured according to AASHTO M320 at high, intermediate and low test temperatures.

9.1.2 Continuous grade of virgin binder as determined by ASTM D7643.

9.2 *Mortar Properties*

9.2.1 Reclaimed Binder Replacement (See Appendix X1.1) in mortar.

9.2.2 Blended binder continuous grade as determined by ASTM D7643.

9.2.3 Rate of change in continuous grade due to the presence of recycled binders (°C/%Binder Replacement).

10. PRECISION AND BIAS

10.1 Adherence to precision and bias statements of ASTM D 6648-08 – Standard Test Method for Determining the Flexural Creep Stiffness of Asphalt Binder Using the Bending Beam Rheometer and ASTM D 7175-08 – Standard Test Method for Determine the Rheological Properties of Asphalt Binder Using the Dynamic Shear Rheometer has been found to be acceptable. Ongoing testing is being completed to further characterize precision and bias of the proposed procedure.

APPENDIX

X1. EXAMPLE MORTAR MIXTURE DESIGN CALCULATIONS

X1.1 *Mortar Mix Design*. The following calculations are an example of mortar mixture design calculations performed to determine the appropriate material proportions for the mortar samples. However, other methods of design are possible.

RAP/RAS mortar samples are prepared according to (6.4.1). The following equations are valid for RAS materials and RAP/RAS material blends, with the only adjustment being the R_{100} asphalt content. The mortar total asphalt content AC_{total} and percent binder replacement AC_{RAP} are calculated from the following two equations:

$$Mortar\ Total\ Asphalt\ Content: AC_{totalRAP/RAS} = \left[\frac{(RAP_S * R100_{AC}) + VB}{RAP_S + VB}\right] * 100$$

$$Reclaimed\ Binder\ Replacement: AC_{RAP} = \left[\frac{RAP_S * R100_{AC}}{(RAP_S * R100_{AC}) + VB}\right] * 100$$

Where,
$AC_{total-RAP/RAS}$: RAP/RAS Mortar total asphalt content [%]
AC_{RAP}: Percent RAP/RAS binder replacement [%]
RAP_S: Sieved R_{100} RAP/RAS material quantity [g]
$R100_{AC}$: R100 RAP/RAS asphalt content [%]
VB: Virgin binder quantity at prescribed level of aging [g]

Aggregate mortar samples are prepared according to (6.4.2). Here, the user will only control the quantity of burned R_{100} aggregates, as the procedure requires that the total binder content matches that of the RAP/RAS mortar. To meet this requirement the following equation must hold:

$$VB = \frac{AC_{total-RAP/RAS} * RAP_{AM}}{1 - AC_{total-RAP/RAS}}$$

Where,
RAP_{AM}: Quantity of burned R_{100} RAP aggregates required for aggregate mortar [g]

The Aggregate Mortar total binder content $AC_{total-AM}$ is then be expressed as

$$AC_{total-AM} = \left(\frac{VB}{RAP_{AM} + VB}\right) * 100$$

And

$$AC_{total-AM} = AC_{total-RAP/RAS}$$

Note that for the previous equation to be true, the quantity of virgin binder required for the Aggregate Mortar will be greater than the quantity used in the RAP/RAS mortar as in this mortar a portion of the virgin binder (at the prescribed aging condition) is replaced with recycled binder.

X2. EXAMPLE BLENDED BINDER CONTINUOUS GRADE CALCULATIONS

X2.1 *Estimation of Blended Binder Continuous Grade*. The following calculations are an example of calculations used to estimate the blended binder continuous grade and the rate of change in continuous grade (rate of improvement) of the virgin binder. The calculations are shown for low temperature grading but the methodology remains unchanged for performance at other grading temperatures.

An example set of low temperature test results is given in Table X2.1.

Table X2.1 – Example Low Temperature Testing Results

Binder Replacement in RAP/RAS Mortar	25%			
Test Temperature, °C	-12		-18	
Test Specimen	Average BBR Parameter		Average BBR Parameter	
	Stiffness S(60) MPa	m-value	Stiffness S(60) MPa	m-value
PAV Aged Virgin Binder	160	0.343	283	0.316
RAP/RAS Mortar	1075	0.263	1790	0.233
Aggregate Mortar	675	0.324	1180	0.278

The low temperature stiffness continuous grade of the PAV aged virgin binder is first calculated following ASTM D 7643 as:

$$Tc = T_1 + \left(\frac{log_{10}(S_s) - log_{10}(S_1)}{log_{10}(S_2) - log_{10}(S_1)}\right)(T_2 - T_1) - 10$$

Where,
T_C: Continuous grading temperature, [°C]
T_1: Lower of the two test temperatures, [°C]
S_s: Specification requirement for stiffness; determined at the respective PG grading temperature [log MPa]
S_1: Test result for the stiffness at T_1 [log MPa]
S_2: Test result for the stiffness at T_2 [log MPa]
T_2: Higher of the two test temperatures, [°C]

The low temperature m-value continuous grade of the PAV aged virgin binder is then calculated following ASTM D 7643 as:

$$Tc = T_1 + \left(\frac{m_s - m_1}{m_2 - m_1}\right)(T_2 - T_1) - 10$$

Where,
m_s: Specification requirement for m-value; determined at the respective PG grading temperature
m_1: Test result for the m-value at T_1

m_2: Test result for the m-value at T_2

For the test results presented in Table X2.1, the virgin binder continuous grade is calculated to be:

Stiffness Continuous Grading Temperature -29 °C
m-value Continuous Grading Temperature -32 °C

Calculate δ_{RAP} according to 8.2.3. The calculation is shown in Table X.2.2 for the data set given above.

Table X2.2 – Calculation of δ_{RAP}

Test Temperature, °C	-12		-18	
Test Specimen	Average BBR Parameter		Average BBR Parameter	
	Stiffness S(60) MPa	m-value	Stiffness S(60) MPa	m-value
RAP/RAS Mortar	1075	0.263	1790	0.233
Aggregate Mortar	675	0.324	1180	0.278
δ_{Tx}	$\frac{\log(1075)}{\log(675)} = 1.07$	$\frac{0.263}{0.324} = 0.812$	$\frac{\log(1790)}{\log(1180)} = 1.06$	$\frac{0.233}{0.278} = 0.834$
δ_{RAP}	$\frac{1.07 + 1.06}{2} = 1.07$	$\frac{0.812 + 0.834}{2} = 0.823$		

The blended binder properties are then estimated by multiplying δ_{RAP} and the PAV aged virgin binder properties. Note that δ_{RAP} is multiplied by the logarithm of the stiffness but the arithmetic m-value. The results for the data set are shown below.

Table X2.3 – Calculation of Estimated Blended Binder Properties

Test Temperature, °C	-12		-18	
Test Specimen	Average BBR Parameter		Average BBR Parameter	
	Log Stiffness S(60) MPa	m-value	Stiffness S(60) MPa	m-value
PAV Aged Virgin Binder	2.20	0.343	2.45	0.316
Estimated Blended Binder Properties	2.35	0.282	2.62	0.260

The blended binder continuous grade is estimated using the formulas given in ASTM D 7643 and shown above using the estimated blended binder properties. For the test results presented above, the blended binder continuous grade is calculated to be:

Stiffness Continuous Grading Temperature -25 °C
m-value Continuous Grading Temperature -17.6 °C

The rate of change in continuous grade due to replacement of virgin binder with recycled binder. Is calculated according to 8.3.1 as:

$$0.16\ \frac{°C}{PBR} = \frac{(-25) - (-29)}{25\%}$$

And

$$0.58\ \frac{°C}{PBR} = \frac{(-17.6) - (-32)}{25\%}$$

The more conservative of the two rate of change numbers for design, in this case example, the 0.58°C/PBR for the m-value represents the most extreme change in properties, and thus controls binder replacement levels. An example of the blending chart that is generated from this analysis and the sensitivity of S(60) and m(60) to binder replacement is provided in Figure 1. In the figure, dashed lines represent portions of the chart that were extrapolated.

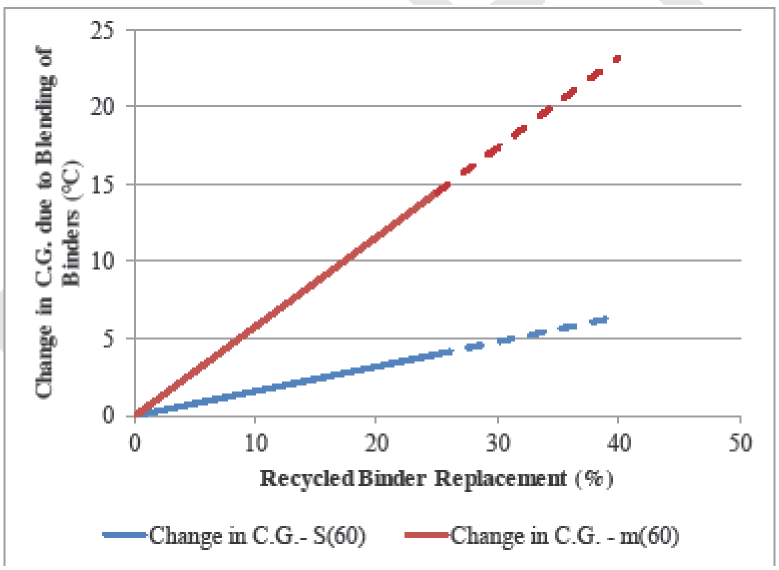

Figure X2.1 – Change in Continuous Grade based on S(60) and m(60) with Increasing Recycled Binder Replacement.