Evaluation of Warm Mix Asphalt with Reclaimed Asphalt Pavement in Rhode Island

Neha Shrestha, Kang-Won Wayne Lee, and George Veyera

Abstract The potential for using Warm Mix Asphalt (WMA) in reclaimed asphalt pavement (RAP) is becoming more interesting topic nowadays due to economic and environmental benefits. WMA technology allows reductions in production and compaction temperatures guaranteeing relevant environmental cost saving benefits. The objectives of the present study were to investigate and evaluate the performance of a typical additive in WMA pavement with RAP on rutting, fatigue cracking and thermal cracking resistance, which was used on Rhode Island (RI) Route 102. The asphalt binder was tested at different dosages of WMA additive using Dynamic Shear Rheometer (DSR), Rolling Thin Film Oven (RTFO), Pressure Aging Vessel (PAV), Multiple Stress Creep Recovery (MSCR), and Bending Beam Rheometer (BBR). It was found that 0.70% additive would lessen pavement damage in case of rutting, fatigue cracking and thermal cracking. After that, two HMA specimens, with and without RAP and two WMA specimen with and without RAP were prepared using Superpave Gyratory Compactor (SGC). These specimens were tested with the Asphalt Mixture Performance Tester (AMPT) and developed master curves for each specimen. It was observed that WMA mixtures with RAP, could perform better in fatigue resistance but expected to have poor rutting performance than HMA and HMA-RAP. This study indicates that addition of WMA additives performs better in fatigue resistance.

Keywords Warm mix asphalt · Reclaimed asphalt pavement · Dynamic modulus · Superpave gyratory compactor · Master curve · Fatigue

1 Introduction

Increasing truck traffic loads and traffic volumes have led to high demand for more durable and sustainable pavements. It has been estimated that truck traffic on US highways will continue to increase, surpassing all other modes of freight shipment

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soon. Tractor trailers and heavy vehicles account for much of the damage done to highways [1]. Thus, a University of Rhode Island (URI) research team has been developing a durable pavement structure based on performance analyses as shown in Fig. 1. In addition, an asphalt mixture that will have high thermal cracking (TC) resistance and top-down fatigue cracking (TDFC) resistance was researched for wearing of surface course. Another mixture was evaluated for intermediate course that will have high fatigue cracking resistance in the asphalt layer in the present study. The final step is to develop an asphalt mixture that will have high bottom-up fatigue cracking resistance (BUFC) for asphalt base layer.

Since there is routinely a need for considerable pavement rehabilitation, substantial volumes of removed asphalt concrete, also called reclaimed asphalt pavement (RAP), are typically accumulated. The disposal of this material consumes a considerable volume of land and creates the potential for environmental pollution. These excess materials, in turn, create the opportunity for greater amounts of old asphalt pavements to be re-used and reclaimed, reducing the need for virgin materials, and providing an environmentally friendly alternative. Additionally, accumulated RAP materials and growing traffic volumes have resulted in the desire to find more sustainable pavement strategies and techniques.

RAP consists of granular pavement materials containing a mixture of bitumen and aggregates produced after the removal of the asphalt pavement by partial or full depth milling. Asphalt pavement rehabilitation involves milling and resurfacing of the existing asphalt pavement to mitigate rutting, cracking, potholes, and other distresses. Natural resources are conserved by using RAP, which also reduces costs, and is environment friendly. Millions of tons of RAP are stockpiled in the northeastern United States. Several studies indicating that RAP materials could be effectively used to resist rutting, thermal cracking, and temperature susceptibility using different recycling techniques [2–5]. One of the advantages of RAP utilization is that it does not have to be heated to a high temperature to prevent further aging (of its already aged binder), and the ability for rapid pavement restoration. In fact, to prevent RAP from getting heated to a very high temperature (very high referring to conventional virgin mix production temperature, e.g., 150 °C; recommended ranges of generally

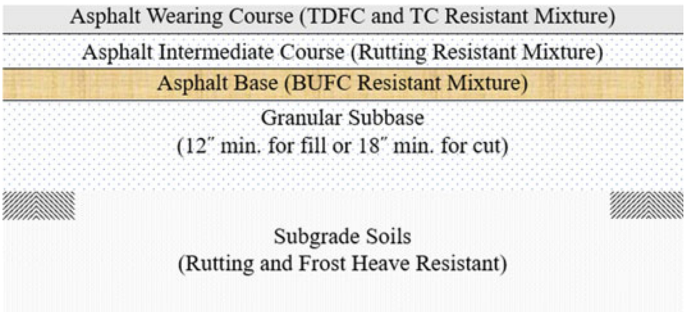

Fig. 1 Proposed durable asphalt pavement section for New England

accepted RAP temperatures are 110–135 °C), the material will be introduced in a separate spot in the recycling drum plant, so that it does not encounter the heater flame, but instead is heated by contact with the superheated (190 °C) virgin aggregates [6].

Warm Mix Asphalt (WMA) is a recent broad category of technology in the asphalt industry that is aimed at reducing energy consumption in the production and construction of asphalt pavements [7, 8]. WMA is produced at temperatures of about 15–40 °C, lower than those required to produce hot mixed asphalt (HMA). This technology provides a method of attaining low viscosity in the asphalt at relatively low temperatures [7]. Energy savings provide two major benefits: lower construction costs, and reduced emissions, which makes asphalt pavement construction more affordable and more environmentally conscious [8]. Incorporating WMA additives in RAP mixtures can increase the sustainability benefits, and also enhance asphalt pavement performance against rutting [5]. In addition, WMA allows high proportions of RAP to be used in asphalt mixtures [5]. WMA uses additives (organics/chemicals) or a water-based foaming process to reduce the temperature needed to produce asphalt materials [9, 10].

The Rhode Island Department of Transportation (RIDOT) has done several trial projects using different types of WMA additives to build stronger, more durable asphalt roads, with reduced physical maintenance requirements, providing reduced costs. Route 102 in Coventry, RI is one of the trial sections conducted by RIDOT using warm mix chemical additives. Typically, RI regular HMA asphalt pavements are designed for a 20-year life, and generally consist of four layers (subgrade soils, granular subbase, asphalt base, and asphalt surface). The routine maintenance and rehabilitation of pavement requires a effective strategy to meet design life requirement and beyond. Thus, a new pavement structure has been proposed as shown in Fig. 1. However, at this time, there are no specific layer materials to use based on performance. The present study was an initial exploration of suitable asphalt mixtures for each layer based on performance with and without RAP.

2 Objective of the Study

The primary objective of this study was to develop WMA mixtures with RAP in RI durable pavement structures for improved performance, and to investigate the feasibility of using a typical WMA additive in the asphalt layer. Specific research objectives were as follows:

1. Compare rheological properties of asphalt binder contained WMA additive with the asphalt binder,
2. Explore longer lasting asphalt mixtures including WMA additive with and without RAP,
3. Determine the dynamic moduli of asphalt specimens to develop Master Curves, and

Table 1 Unmodified and modified asphalt binders

Specimen name	Description
0-0	PG 58-28 without WMA additive
0-5	PG 58-28 with 0.5% WMA additive
0-7	PG 58-28 with 0.7% WMA additive
0-9	PG 58-28 with 0.9% WMA additive

4. Predict the rutting and cracking performance of pavement structures with WMA additive containing RAP.

3 Materials and Specimen Preparation

3.1 Asphalt Binder

Rhode Island typically uses PG 58-28 as the base asphalt binder when mixtures contain more than 15–25% of RAP as per RIDOT standard specifications [11]. For this study, a PG 58-28 asphalt binder without any polymer modification was acquired from a local distributor in Providence.

3.2 WMA Additive

A new generation chemical WMA additive, that has been commonly used by the asphalt paving industry in New England States, provides a temperature reduction in the range of 50–75 °C lower than typical HMA. Inclusion of the WMA additive helps to integrate recycled asphalt materials more easily in asphalt mixtures [12]. A typical chemical WMA additive has been used for this study, and specimens were prepared based on the manufacturer's recommendations. Table 1 shows both the unmodified and modified asphalt binder.

3.3 Mineral Aggregates

It is rare to obtain a desired aggregate gradation from a single aggregate stockpile. Therefore, Superpave mix designs usually draw upon several different aggregate stockpiles and blend them together in a ratio that will produce an acceptable final blended gradation. Superpave mix designs typically use 3 or 4 different aggregate stockpiles. The present study used four types of aggregates sources and one RAP source. Within the Superpave mix design requirements, gradations are identified based upon NMAS (Nominal Maximum Aggregate Size), which is defined as one

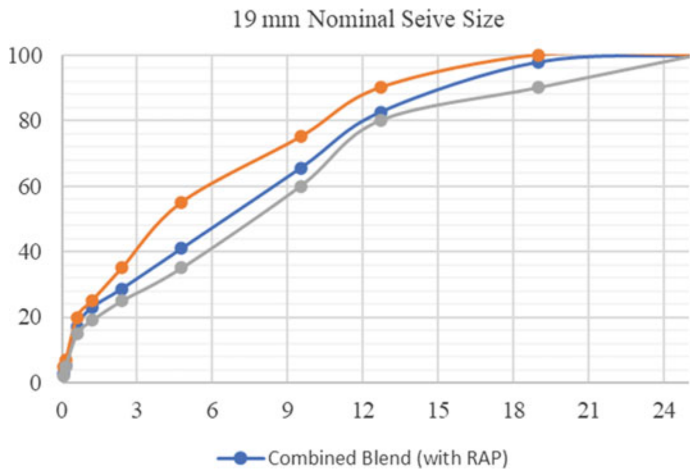

Fig. 2 Blend of aggregates passed through RIDOT specification

sieve size larger than the first sieve size that retains more than 10%. In accordance with RIDOT specifications, mixes containing RAP should have 19 mm NMAS. Figure 2 shows the blended aggregates with percentages that meet the RIDOT specification.

Specimen Preparation

WMA binders were prepared using a high shear mixer (i.e., mechanical stirrer with attached heater). Asphalt was heated to 138 °C (280°F) in the oven until the asphalt binder became liquid. Then, asphalt binder was removed from the oven, and the typical WMA additive was slowly introduced at a low mixing speed. The mixture was then sheared at a rotation speed of 150 rpm for about 30 min to ensure a homogenous blended mix.

4 Laboratory Testing Methods

4.1 Dynamic Shear Rheometer (DSR) Test

The DSR test was used to evaluate the viscous and elastic behaviors of asphalt binders at temperatures starting from medium to high. The DSR measures a specimen's complex shear modulus (G*) and phase angle (δ). The complex shear modulus (G*) can be the specimen's total resistance to deformation when repeatedly sheared, while the phase angle (δ), is the lag between the applied shear stress and the resulting shear strain. In Superpave, high temperature grade has been defined as the temperature where $G*/\sin\delta$ of bitumen before and after aging is higher than 1.0 and 2.2 kPa, respectively.

4.2 Rolling Thin-Film Oven (RTFO) Test

The RTFO test was used to produce short-term aged asphalt binder for physical property testing following AASHTO T 240 [13]. Asphalt binder is exposed to oven temperatures at 163 °C (325°F) for 85 min to simulate manufacturing and placement aging. This aging is expected to represent typical conditions where the asphalt is considerably aged from the first exposure to the plant burner and contact with hot aggregates, throughout hauling and paving, until the final compaction takes place. Research leading to the development of the RTFO test indicated that an aging time of 85 min produces aging effects comparable to average field conditions.

4.3 Multiple Stress Creep Recovery (MSCR) Test

The MSCR test is the most recently method for binders to evaluate their non-recoverable creep compliance (J_{nr}), which is the ratio of non-recoverable strain to applied stress. Field loading conditions were simulated following AASTHO M332 procedures, which is why J_{nr} can be considered as a rutting parameter [13]. This approach provides a high temperature binder specification that more accurately represent the asphalt binder rutting performance. A major benefit of the MSCR test is that it eliminates the need to run additional tests such as elastic recovery, toughness and tenacity, and force ductility. This test is specifically designed to indicate the effects of polymer modification additives on the asphalt binder behavior. The MSCR test uses the well-established creep and recovery test concept to evaluate the binder's potential for permanent deformation.

4.4 Bending Beam Rheometer (BBR) Test

The BBR test provides a measure of the low temperature stiffness and relaxation properties of asphalt binders. These parameters give an indication of an asphalt binder's ability to resist low temperature cracking. PAV residue was tested at low temperature using the BBR to measure creep stiffness (S) and logarithmic creep rate (m) at 60 s as per AASTHO T313 [13]. The specification requires that the stiffness of the testing specimen (S) should be less than 300 MPa, and the value of m should be greater than 0.300 to resist low temperature cracking.

4.5 Indirect Tensile Test

The tensile properties of bituminous mixtures are evaluated in the Indirect Tensile test where a cylindrical specimen is loaded by a compressive load along a diametric plane at a constant rate acting parallel to and along the vertical diametrical plane of the specimen through two opposing loading strips. This loading configuration develops a relatively uniform tensile stress perpendicular to the direction of the applied load, and along the vertical diametrical plane, ultimately causing the specimen to be tested in tension by splitting along the vertical diameter. The Superpave mix design method was used to prepare the specimen for this test. HMA and WMA specimens with 20% RAP, were prepared at the optimum binder content (OBC) at a temperature of 25 °C for a period of 2 h in air. These specimens were then mounted on a conventional Marshall testing apparatus, loaded at a deformation rate of 51 mm/min with the load at failure recorded for each case. A 13 mm (0.5 in) wide loading strip was used for the 101 mm (4 in) diameter specimens to provide a uniform loading that produces a nearly uniform stress distribution. The static indirect tensile strength of specimens was determined using the procedure outlined in ASTM D 6931 [14].

4.6 Dynamic Modulus

For linear viscoelastic materials such as HMA, the stress–strain relationship under a continuous sinusoidal loading is defined by its complex dynamic modulus ($|E^*|$). This is a complex number that relates stress and strain for linear viscoelastic materials subjected to continuously applied sinusoidal loading in the frequency domain. The complex modulus is defined as the ratio of the amplitude of the sinusoidal stress (at any given time, t, and angular load frequency, ω), $\sigma = \sigma_o \sin(\omega t)$, and the amplitude of the sinusoidal strain $\varepsilon = \varepsilon_o \sin(\omega t - \delta)$, at the same time and frequency, that results in a steady state condition.

$$|E*| = \sigma/\varepsilon = \sigma_o \sin \omega t / \varepsilon_o \sin(\omega t - \delta) \tag{1}$$

where,

σ_o = peak (maximum) stress,
ε_o = peak (maximum) strain,
δ = phase angle (deg),
ω = angular velocity,
t = time (s), and
i = imaginary component of the complex modulus.

Stiffness (dynamic modulus) is a key material property that determines strains and displacements in pavement structures. Stiffness data for a HMA mix as obtained from the $|E^*|$ test provide very important information about the linear viscoelastic behavior of that particular mix over a wide range of temperatures and loading frequencies.

4.7 Master Curves

The mechanical behavior of viscoelastic materials such as asphalt composites is dependent on the temperature and the loading frequency at which the material is tested [15]. To compare test results of various mixes, it is useful to normalize one of these variables. Data collected at different temperatures can be "shifted" relative to the time of loading, so that the various curves can be aligned to form a single master curve. Master curves are constructed using the principle of time–temperature superposition. The data at various temperatures are shifted with respect to time until the curves merge into a single smooth function. The master curve for dynamic modulus as a function of time developed in this manner describes the time dependency of the material. The amount of shifting at each temperature required to form the master curve describes the temperature dependency of the material. The greater the shift factor, the greater the temperature dependency (temperature susceptibility) of the mixture.

Time–Temperature Superposition of |E*|—In the Mechanistic-Empirical Pavement Design Guide (MEPDG) the stiffness of HMA at all temperature levels and time rate of loading, is determined from a master curve constructed at a reference temperature (generally taken as 70°F) [16]. Master curves are constructed using the principle of time–temperature superposition [17]. The data at various temperatures are shifted with respect to time until the curves merge into single smooth function. The master curve of the modulus as a function of time formed in this manner describes the time dependency of the material. The amount of shifting at each temperature required to form the master curve describes the temperature dependency of the material. In general, the master modulus curve can be mathematically modeled by a sigmoidal function described by:

$$\text{Log}\left|E^{*}\right| = \delta + \frac{\alpha}{1 + e^{\beta+\gamma(\log t^{r})}} \tag{2}$$

where,

t_r = reduced time of loading at reference temperature,
δ = minimum value of $|E^*|$,
$\delta + \alpha$ = maximum value of $|E^*|$, and
β, γ = parameters describing the shape of the sigmoidal function.
The shift factor can be shown in the following form:

$$a(T) = \frac{t}{t_r} \text{ or } \log\,(f_r) = \log\,(f) - \log\,[a(T)] \tag{3}$$

where,

$a(T)$ = shift factor as a function of temperature,
t = time of loading at desired temperature,
t_r = reduced time of loading at reference temperature, and
T = temperature of interest.

For improved precision, a 2nd order polynomial relationship between the logarithm of the shift factor, i.e., Log $a(T_i)$and the temperature in °F is used. The relationship can be expressed as follows:

$$\text{Log a}(T_i) = aT_i^2 + bT_i + c \tag{4}$$

where,

$a(T_i)$ = shift factor as a function of temperature T_i,
T_i = temperature of interest, °F, and
a, b and *c* = coefficients of the second order polynomial.

|E*|

$|E^*|_{Measurements}$ ***from Master Curves***—Following the new MEPDG's input Level-1 approach, $|E^*|$ master curves of all mixtures were constructed for a reference temperature of 70°F using the principle of time–temperature superposition [17]. The data at various temperatures were shifted with respect to time until the curves merged into a single sigmoidal function representing the master curve using a 2nd order polynomial relationship between the logarithm of the shift factors, log $a(T_i)$ and the temperature. The time–temperature superposition was done by simultaneously solving for the four coefficients of the sigmoidal function (δ, α, β, and γ) as described in Eq. 2, and the three coefficients of the 2nd order polynomial (*a*, *b*, and *c*) as described in Eq. 4. The Microsoft Excel™ "Solver" function was used to conduct the nonlinear optimization for simultaneously solving these seven parameters.

5 Testing Results and Analysis

5.1 Rutting Resistance

Rutting is a wheel path surface depression resulting from plastic or permanent deformation in each pavement layer. AASHTOWare Pavement ME Design (PMED) computes the rut depths within the HMA, unbound aggregate layers, and subgrade soils. Since the New England States have strong subgrade soils, and use good quality granular subbase materials, the present study concentrated on investigating (intermediate course) HMA permanent deformations only. To resist rutting, an asphalt binder should be stiff, and it should be elastic. Therefore, the complex shear modulus elastic portion, $G^*/\sin\delta$ should be high. When rutting is of greatest concern, a minimum value for the elastic component of the complex shear modulus is specified. Intuitively, the higher the G^* value the stiffer the asphalt binder is, and the lower the δ value, the greater the elastic portion of G^* is. Therefore, the higher the rutting parameter, the better the performance against rutting.

From DSR results of unaged binder as shown in Fig. 3, asphalt binder containing 0.7 and 0.9% additive has the highest rutting parameter values compared to other binders. This suggests that that (0.7–0.9) % WMA binder has better performance against rutting.

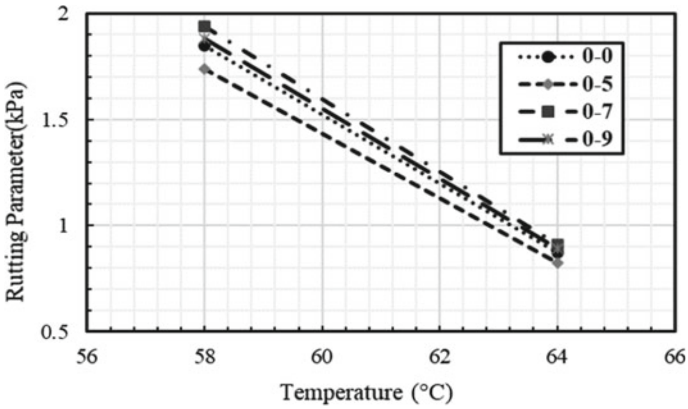

Fig. 3 Rutting parameter of asphalt binder and asphalt binder contained WMA additive

In the MSCR test, two separate parameters can be determined during each loading cycle: non-recoverable creep compliance (J_{nr}), and percentage of recovery (R). In addition to determining J_{nr}, the MSCR test can be used to determine the amount of recovery in an asphalt binder during the creep-recovery testing. MSCR Recovery provides an indication of the delayed elastic response of the asphalt binder. A high delayed elastic response indicates the asphalt binder has a significant elastic component at the test temperature. If the asphalt binder meets the appropriate J_{nr} specification, then it would be expected that the binder will minimize its contribution to rutting.

For standard traffic loading, J_{nr} (at 3.2 kPa) is required to have a maximum value of 4.0 kPa^{-1}. Also, AASHTO MP19 maintains a requirement that the difference in J_{nr} values between 0.1 and 3.2 kPa shear stress should not exceed a ratio of 0.75, and Table 2 shows that the results agree with these conditions. Also, from the MSCR test results (Table 2), asphalt binder containing 0.9% additives showed the higher recovery percentage, and lower non-recoverable creep compliance as compared with the regular asphalt binder.

Table 2 Results of MSCR tests for unmodified and modified binders

Asphalt binder	$R_{0.1}$(%)	$R_{3.2}$(%)	R_{DIFF}(%)	J_{NR}0.1(KPA^{-1})	J_{NR}3.2(KPA^{-1})	J_{NR}diff (%)
0-0	20.79	11.53	44.53	0.680	0.787	15.80
0-5	16.36	6.65	59.37	1.090	1.290	18.42
0-7	18.09	8.53	52.87	0.883	1.040	17.54
0-9	25.34	13.41	47.06	0.599	0.725	21.06

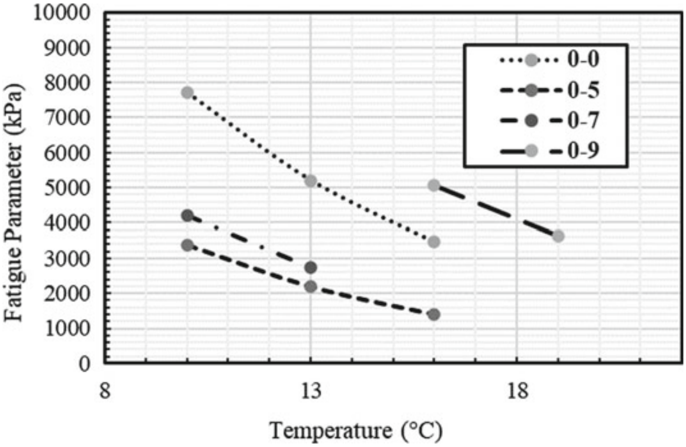

Fig. 4 Fatigue resistance of asphalt binder and asphalt binder with WMA additive

5.2 Fatigue Resistance

Fatigue damage in asphalt pavements is a complex phenomenon occurring from repeated bending resulting in asphalt pavement micro-damage. This micro-damage is a competitive process between micro-cracking and healing, manifested as a reduction in asphalt pavement stiffness, degrading the load-carrying capacity and the ability to resist further damage. Eventually, micro-cracks coalesce into macro-cracks that appear in the wheel path. To resist fatigue cracking, an asphalt binder should be elastic (able to dissipate energy by rebounding and not cracking), but not too stiff (excessively stiff substances will crack rather than deform-then-rebound). Therefore, the complex shear modulus viscous portion, $G^{*}\sin\delta$ should be a minimum. When fatigue cracking is of greatest concern (late in a HMA pavement's life), a maximum value for the viscous component of the complex shear modulus is specified. Figure 4 shows that 0.5 and 0.7% have the lowest $G^{*}\sin\delta$ values compared with the unmodified asphalt binder, which indicates that (0.5–0.7) % WMA binders showed better performance against fatigue cracking.

5.3 Resistance Against Low Temperature Cracking

The BBR test is used to determine an asphalt binder's creep stiffness as a function of time. Because low temperature cracking is a phenomenon found mostly in older pavements, the test is run on the long-term aged residue from the PAV. As temperature becomes low, the asphalt binders start to become stiffer. Stiffness is the extent to which an object resists deformation in response to an applied force. Also, creep stiffness describes the strain in response to stress at low temperature. And, if this creep stiffness is too high, cracking will occur. Since a higher creep stiffness value indicates higher thermal stresses, a maximum creep stiffness value (300 MPa) was specified (Fig. 5). Since a lower *m*-value indicates a reduced ability to for stress relaxation, a minimum *m*-value of 0.300 was specified. From the BBR test results as shown in Fig. 6, it was found that 0.5% WMA binder has less stiffness and higher relaxation parameters (*m*-value) compared with the unmodified asphalt binder, and 0.7% WMA binder has nearly the same stiffness and same relaxation parameter (*m*-value) compared with the unmodified asphalt binder. Therefore, it can be concluded that the (0.5–0.7) % WMA additives containing binders indicated better performance against lower temperature cracking. Also, from the above results in terms of rutting, fatigue cracking and thermal cracking, it can be concluded that 0.7% WMA additives was the optimal dosage rate for further study.

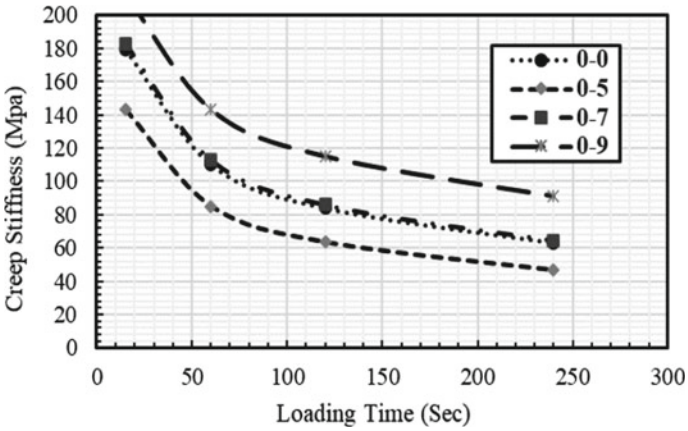

Fig. 5 Creep stiffness of asphalt binder and asphalt binder contained WMA additive

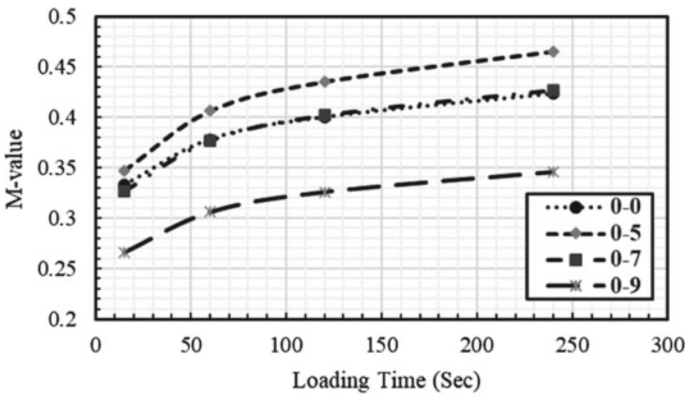

Fig. 6 m-value of asphalt binder and asphalt binder contained WMA additive

5.4 Preparation of Asphalt Mixtures

A Superpave gyratory compactor (SGC) is used as compaction devices and the compaction effort in mix design is based on expected traffic. It is used to prepare the SuperPave mix design and to find out the volumetric properties that relates to the performance of asphalt pavement. Once the binder and aggregates were selected for the asphalt mixture, then they were combined to produce the optimum mixture properties. The SGC is intended to simulate the rolling effect of field paving on the mixture, and it is more consistent with the field stress state of the mixture, being closer to the actual field conditions. In accordance with guidelines to make the specimens for specified traffic loading, less than 0.3 million ESALs were used. Several trial blends were evaluated to determine the optimum binder content (OBC).

5.5 Density and Voids Calculations

The following two measures of densities were determined:

Bulk specific gravity (*Gmb*), and Theoretical maximum specific gravity (*TMD*, *Gmm*).

5.6 OBC Selection

The OBC was determined at 4% air voids. From Fig. 7, the OBC for HMA with RAP was at 5.3, and 5.6% for WMA (0.7% additive) with RAP.

5.7 Indirect Tensile Test Results

Eight specimens were prepared and tested for indirect tensile strength. Results for the HMA and WMA mixtures at unconditioned OBC specimens are given in Table 3. The test results showed that the average tensile strength of HMA mixtures with 20% RAP is 677.5 kPa with a standard deviation of 299.1 kPa, whereas WMA is 588.6 kPa with a standard deviation 110.9 kPa. The test results showed a variation in strength, which could be attributed to specimen preparation, material handling, instrument handling, and other potential sources of error. However, it was found that HMA mixtures have high average tensile strengths compared with WMA mixtures.

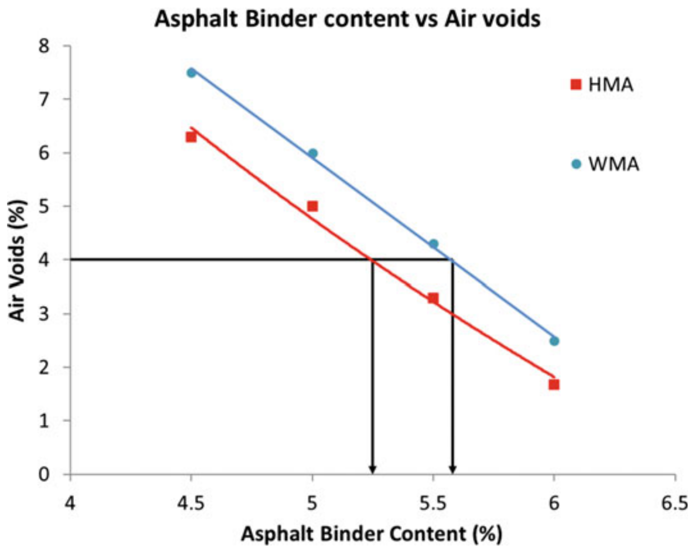

Fig. 7 Air void effect on different asphalt binder content

Table 3 Results of indirect tensile strength tests of WMA and HMA specimens

Specimen #	Specimen thickness (mm)	Specimen diameter (mm)	Maximum load (N)	IDT strength (kPa)
HMA #1	55.9	150	11,676.8	873.3
HMA #2	48.3	150	10,123.8	876.7
HMA #3	48.1	150	5224.3	453.6
HMA #4	50.8	150	6154.4	506.3
WMA #1	42.0	150	5255.5	523.6
WMA #2	50.8	150	6434.7	529.4
WMA #3	49.5	150	6483.7	547.1
WMA #4	48.3	150	8708.7	754.3

5.8 Dynamic Modulus Test Results

Four specimens were prepared and tested using the Asphalt Mixtures Performance Tester (AMPT). Testing details are shown in Table 4, and AMPT results are given in. Figure 8 Results indicate that the dynamic modulus of all WMA and HMA mixtures decreases significantly with the increase of temperature and exhibits frequency dependency under the same temperature conditions. Higher loading frequency means the higher dynamic modulus for asphalt mixtures, which illustrates the viscoelastic properties of that asphalt mixtures [18].

Table 4 Dynamic modulus of HMA and WMA specimens

Specimen A	Asphalt binder PG 58-28	HMA mixtures for Base with 20% RAP
Specimen B	Asphalt binder PG 58-28 with 0.7% Evotherm	WMA mixtures for base with 20% RAP
Specimen C	Asphalt binder PG 64-28	HMA mixtures for base without RAP
Specimen D	Asphalt binder PG 64-28 with 0.7% Evotherm	WMA mixtures for base without RAP

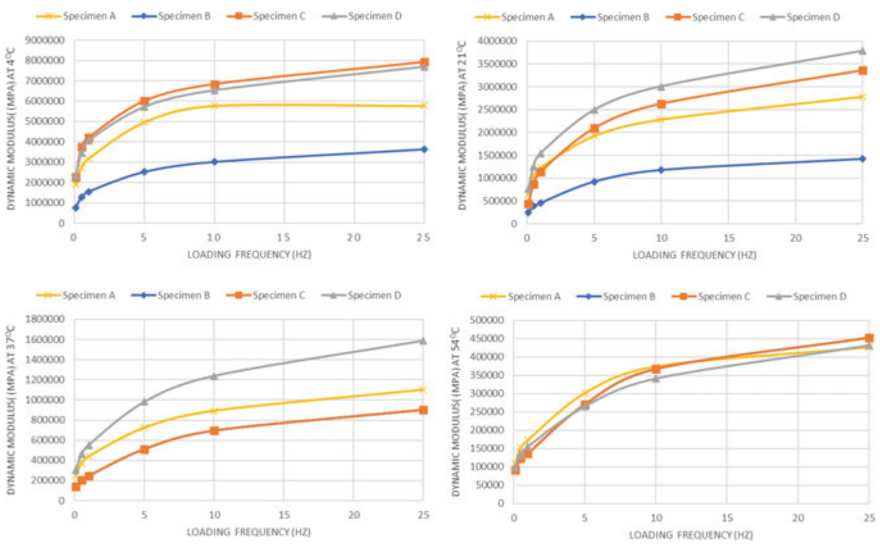

Fig. 8 Dynamic modulus of HMA and WMA specimens

Four specimens were prepared according to the AASTHO T 342-1 specification [13]. Specimens were tested in the AMPT machine to determine the dynamic modulus of the respective mixtures. Dynamic modulus testing requires a 150 mm high by 100 mm diameter specimen at a target air void content, cored from a 175 mm high by 150 mm diameter specimen.

In this study, 21 °C (70°F) is taken as the reference temperature and $|E^*|$ master curves of all mixes were developed at the reference temperature following the principle of time–temperature superposition. The data at various temperatures were shifted in line with frequency until the curve merges into a single sigmoidal function, representing the master curve (Fig. 9). The least-squares method is used for determining the coefficients of the sigmoidal functions (α, β, γ and δ) (Table 5). The shift factor is a function of temperature and is independent of strain level so it can be used within the linear viscoelastic range to be predict material behavior at any strain level.

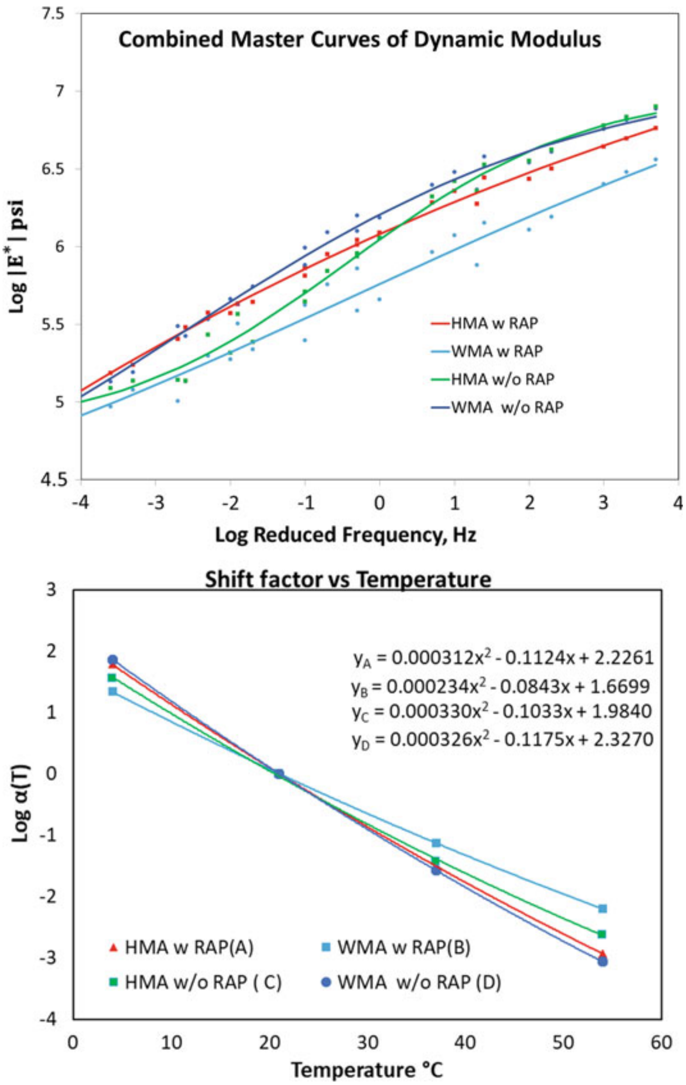

Fig. 9 Master curves and shift factor plots of HMA and WMA

Table 5 Master curve parameters

Specimen	δ	α	β	γ
HMA with RAP	4.568099	2.490947	−0.445790	−0.468820
WMA with RAP	4.629264	2.267807	0.0602610	−0.645040
HMA without RAP	4.84628	2.190182	−0.155180	−0.820180
WMA without RAP	4.088297	3.098425	−0.768200	−0.417560

Figure 9 shows combined master curve for four different mixes plotted together. This figure showed that at high temperature and low frequency, HMA-RAP indicates the highest reduced modulus, and WMA without RAP has second highest modulus compared to other mixes. These results suggest that WMA without RAP can be expected to improve performance against rutting compared with HMA without RAP and WMA-RAP [19]. Again, at high frequency and low temperature, WMA-RAP shows the lowest reduced modulus compared with all other mixes. Results for WMA-RAP mixtures can also be expected to improve the performance against fatigue cracking compared with other mixtures but was found to have poor rutting resistance compared with HMA and HMA-RAP.

6 Conclusions and Recommendations

Based on the laboratory test results and analyses from this study, the following conclusions can be made:

1. The addition of WMA additive enhanced the resistance properties of the asphalt binder against rutting, fatigue, and temperature cracking.
2. Combined Master Curves showed that WMA-RAP performed better in fatigue cracking resistance than HMA and HMA-RAP.
3. It was found that there was an average 20% drop oi tensile strength in WMA-RAP as compared to HMA-RAP mixtures.
4. There is need for further investigation and improvement regarding rutting performance using WMA additive.

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