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Fatigue performance assessment of recycled tire rubber modified asphalt mixtures using viscoelastic continuum damage analysis and AASHTOWare pavement ME design

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HIGHLIGHTS

- RTR mixtures provided better fatigue lives than neat and softer asphalt mixtures.
- DVR mixes outperformed at high strain level, implying better for heavy traffic roads.
- PP-VECD analysis and Pavement ME results showed similar fatigue life trend.
- Initiation of pavement rehabilitation was retarded 15 times with RTR modification.

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ABSTRACT

Improving fatigue cracking resistance of asphalt mixtures is an important issue that concerns roadway agencies, public and industry. There are various, well-established methods for improving the material characteristics, such as polymer modification methods. Recycling/reusing scrap tire rubber in asphalt pavements has also been an interest of engineers. As such, many different methods of scrap tire rubber (herein called crumb rubber) modification alternatives are being developed. This study focused on a comparative evaluation of the fatigue cracking resistance of various base and recycled tire rubber modified asphalt mixtures, including the relatively new and innovative devulcanized rubber (DVR), crumb rubber terminally blend (CRTB), crumb rubber wet process (CRWET), original/base PG58-28 and softer PG58-34 binders, by using Push-Pull Viscoelastic Continuum Damage (PP-VECD) analysis and AASHTOWare Pavement Mechanistic Empirical Design (Pavement ME) software. The results of the study revealed that rubberized asphalt mixtures provided better fatigue lives than the base and softer binders did. Especially, DVR mixtures outperformed at high strain levels, implying they can be used for heavy traffic roads to enhance the fatigue life of asphalt pavements. Both PP-VECD and Pavement ME produced results that showed similar trends.

1. Introduction

Pavement cracking caused by repeated bending distresses has been a concern for engineers since the late 1940s [1]. Repeated application of the wheel loads results in the fatigue failure of the asphalt surface and base courses. Flexible pavements are loaded each time a vehicle passes on them. While the strains induced by each loading below the fatigue endurance limit (FEL) are not believed to cause damage, those above FEL will result in a significant loss of durability of the pavement structure [2,3].

The fatigue failure of asphalt pavements occurs as short longitudinal cracks in the wheel path where the maximum tensile stresses and strains exist and propagate into interconnected cracks taking the shape of alligator skin pattern [4]. Two types of fatigue cracking can develop on asphalt pavements depending on the location where the cracks initiate. Bottom-up fatigue cracking is commonly observed in comparatively thin asphalt layers because of flexural bending and top-down fatigue cracking can be encountered on the wheel path of thick asphalt layers due to highly aged asphalt binder and high localized tensile stresses created by tire-pavement interaction [5]. Since the fatigue life of the asphalt pavements has been a growing concern for the agencies involved, the

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development of fatigue resistant pavements has been the focus of recent studies over the past couple of decades.

Researchers have been carrying out studies to enhance both the fatigue life and other properties of the asphalt pavements. Modification of the bituminous materials using polymer and rubber-based modifiers has been one of the strategies to accomplish it [6–8]. The use of discarded tires from cars and trucks in the form of recycled tire rubber (RTR) has also been researched [9–12]. Both the economic and environmental benefits of using recycled tire rubber in the pavement construction make it a favorable modifier compared to other alternatives [13–18]. Additionally, the public take advantage of increased service life and minimized preventive maintenance cycles, which need to be conducted on a regular basis, as a result of rubber modified asphalt pavements [19].

Crumb rubber (CR) modification of the asphalt binders is typically performed using a method called the wet process. In wet process, CR is added into hot liquid asphalt binder at temperatures around 325°F- 400°F and in varying amounts [8,20–21]. The CRTB process is similar to the wet process. The main difference is the use of other additives to maintain the CR particles suspended in CRTB binder. Moreover, the amount of the CR integrated in CRTB process is around 10% to 12% by weight of binder, which is lower than the amount used in wet process [8]. Devulcanized rubber (DVR) modification process is another rubber modification technology using the wet process. In the DVR process, the "so-called" devulcanized rubber particles are mixed with liquid asphalt binder at elevated temperatures. DVR particles can be delivered in pelletized or liquid form, latter from makes it more economic. The main advantage of DVR modification is that the rubber particles completely dissolve within the binder allowing the use of current performance grading system [16,22]. Hence, the gradation of DVR particles does not affect the properties as observed in CR modification.

For this study, the fatigue life predictions of asphalt mixtures prepared with the base binder PG58-28, soft binder PG58-34 and recycled tire rubber (RTR) modified asphalt binders CRWET, CRTB, and DVR were examined and compared using both PP-VECD analysis and the Pavement ME software. The descriptions and production/preparation procedures of RTR modified asphalt binders, asphalt mixtures, the laboratory performance tests and test samples, analysis model and the software used were included.

2. Objectives and scope

The objectives of this research can be summarized as follows;

- To investigate and compare the fatigue lives of asphalt pavements manufactured using original, soft and rubber-modified asphalt binders.
- To evaluate the impact of various asphalt binder rubber modification technologies on the fatigue life.
- To determine and compare the fatigue lives of asphalt pavements obtained by PP-VECD analysis and Pavement ME design software.
- To analyse the preventive maintenance initiation times for thin and thick layered asphalt pavements constructed using hot mix asphalt mixtures of this study for a pre-determined threshold value.

In order to achieve the objectives, asphalt mixtures were produced using a single mix design with five different binder types. While two of them were original PG58-28 and soft PG58-34 binders, other three were manufactured using crumb rubber terminally blend (CRTB), devulcanized rubber (DVR) and crumb rubber wet (CRWET) technologies.

Fatigue lives of the asphalt mixtures produced using different binders were analyzed using Pavement ME software and PP-VECD analysis. Pavement ME is a well-known method for designing and analyzing asphalt pavements, which applies layered elastic theory to compute the mechanical responses and empirical models to predict the distresses while PP-VECD analysis uses another method to characterize the fatigue life of asphalt pavements by using the linear viscoelastic behavior and stiffness reduction due to microcracking. In PP-VECD analysis, the so-called damage characteristics curve of a mixture is computed from the cycles-peak stress-peak strain data. Once the damage characteristics curve is obtained, the fatigue life at any temperature and frequency for the desired strain level can be predicted.

3. Materials and methods

3.1. Asphalt mix design

The hot mix asphalts (HMA) prepared in this research study had a 12.5-mm nominal maximum aggregate size (NMAS) with 40% recycled asphalt pavement (RAP) content. Design of the HMA followed the Michigan Department of Transportation (MDOT) specifications for top or leveling course to withstand 1 million ESAL traffic load [23]. MDOT specifications showed slight variations compared to Superpave mix design specifications to consider the locally available materials.

The HMA was designed with PG58–28 asphalt binder at 4.80% binder content and verified with other binder types to ensure that the same mix design could be used with all binder types. Although there were slight changes in the volumetrics, they were still within the specification limits, allowing the use of the same mix design for all binder types. The design aggregate gradation, specification limits, binder content of RAP, specific gravity of aggregates and other design specific information are provided in Table 1.

The RAP used in this study was selected according to the mix design requirements. It was obtained from a single source with the RAP aggregate having an NMAS of 12.5-mm. In order to minimize the possible gradation and binder content variations due to high amount, the RAP was divided into fine and coarse stockpiles

Table 1
12.5-mm MDOT superpave mix design gradation and specification limits.

Sieve Size	Percentage Passing	Superpave Specs
19 mm (3/4″)	100.0	100 min
12.5 mm (1/2″)	90.6	90–100
9.5 mm (3/8″)	81.3	90 max
4.75 mm (#4)	63.5	–
2.36 mm (#8)	48.6	28–58
1.18 mm (#16)	34.6	–
0.6 mm (#30)	22.4	–
0.3 mm (#50)	10.9	–
0.15 mm (#100)	5.8	–
0.075 mm (#200)	3.9	2–10
% G_{mm} at (N_i) (maximum)	90.5%	
Number of Gyrations	$N_{initial}$	7
	N_{design}	76
	N_{max}	117
Specific VFA (%) at N_d		65–78
%G_{mm} at Design Number of Gyrations (N_d)		96%
%G_{mm} at the Maximum Number of Gyrations, (N_m)		98%
VMA min % at N_d (based on aggregate bulk specific gravity, (G_{sb})		14
Fines to effective asphalt binder ratio ($P_{\#200}/P_{be}$)		0.6–1.2
Binder Content		4.8%
RAP Content		40%
Binder Content of RAP		4.53%
Bulk Specific Gravity, (G_{sb})		2.668

by passing though #4 sieve based on the procedure outline in [24,25].

3.2. Preparation of asphalt binders

There were five different binder types selected to investigate the effect of RTR modification on the fatigue life of asphalt mixtures. The first binder was performance grade (PG) 58–28, commonly used in the midwest asphalt pavement projects. The same PG58-28 base binder was used to produce all RTR modified binders in order to minimize the effect of binder source. The second binder type was a softer binder with PG58-34, which was selected according to the blending chart analysis for high RAP asphalt pavements [25]. The other three types (CRTB, DVR, CRWET) were RTR modified binders. Although RTR modified binders were manufactured using the basis of wet process, rubber types and production technologies among the rubberized asphalt binders were completely different. RTR used in this study was in the form of either crumb rubber (CR) #20-mesh size or de-vulcanized rubber. CRTB binder had a CR content of 12% by weight of binder (bwb) and manufactured at an asphalt binder terminal using the same base PG58-28 binder. CRWET and DVR binders were prepared in-house using a high shear mixer. CRWET was produced using 12% CR bwb. Since there were not any uniform guidelines for the production of rubber modified asphalt binders in the literature, the following specifications were developed and applied in this study.

For the CRWET binder, the asphalt cement was heated up to 190°C and CR particles were added. The blend was mixed around 700 revolutions per minute (rpm) for 35 min before storing. For the DVR binder, the de-vulcanized rubber particles at 9% bwb were soaked into asphalt cement at mixing temperature for two hours. Following the soaking, the rubber-binder mix was blended at 3000 rpm for 15 min and the temperature was raised to 190°C. Once the temperature was stable, 1% styrene–butadienestyrene was added into the blend and mixed for another 45 min at 3000 rpm until the homogenous mix was obtained. As the last step, 0.2% bwb liquid Sulphur as a cross-linking agent was incorporated into the blend and mixed for another 15 min before storing.

3.3. Preparation of performance test samples

The Superpave gyratory compactor (SGC) was used to compact the performance test specimens. After batching and mixing, the loose mixtures were short term aged for 4 h at 135°C in the forced-draft oven. The mixing and compaction temperatures were determined based on the viscosity test results of the asphalt binders. While the mixing temperature ranged between 148°C and 173°C for base and laboratory produced RTR modified binders, compaction temperatures were in between 132°C and 163°C. The compaction and mixing temperatures for the CRTB binder followed the manufacturer's recommendation. To provide uniform conditioning of the mixture, loose mixtures were stirred each hour. After short term aging, mixtures were compacted to produce cylindrical samples having 150-mm diameter and 180-mm height. The compacted samples were allowed to cool down to the room temperature overnight. The next day, 100-mm by 150-mm cylindrical samples were cut and cored from them. Only the samples having 7% ± 0.5% were accepted for final performance testing as per AASHTO standard specifications requirements [26].

3.4. Dynamic modulus (|E*|) tests

Dynamic modulus, |E*|, theoretically a non-destructive |E*| test run to characterize the basic linear viscoelastic properties of asphalt mixtures. The tests were conducted using an asphalt mixture performance tester (AMPT) on 100-mm by 150-mm cylindrical HMA samples. The AMPT was capable of applying uniaxial haversine compressive stress at various temperatures and loading frequencies. The data acquisition system recorded |E*| and phase angle values at each temperature-frequency combination as the test runs. The sample preparation, testing and data analysis were performed according to AASHTO standards [26]. While the testing temperatures were −10°C, 10°C, 21°C, 37°C, 54°C, the loading frequencies at each temperature were selected as 25 Hz, 10 Hz, 5 Hz, 1 Hz, 0.5 Hz and 0.1 Hz. A minimum of three replicates were tested for each mixture type.

Once the tests were completed successfully, the |E*| master curve was established using the time–temperature superposition (TTS) principle by following the steps explained below and in [27].

1- A single |E*| master curve is obtained by shifting the |E*| data obtained at different temperatures horizontally in a log–log plot of |E*| versus frequency. The parameter in the horizontal axis is called reduced frequency (f_R) following the shifting and it is defined as:

$$f_R = f\ a_T(T) \tag{1}$$

where f is the frequency of the load and
aT(T) is the shift factor coefficient for a given temperature T.

2- The shift factor coefficient $a_T(T)$, which is the amount of horizontal shift for each temperature, is computed. Then typically a second-order polynomial is fit to the data to acquire the polynomial coefficients a1 and a2 in the following equation:

$$a_T(T) = 10^{a_1\,(T^2 - T_{ref}{}^2) + a_2(T - T_{ref})} \tag{2}$$

where T_{ref} is the reference temperature.

3- The shift factors (a_T(T)) at each temperature are varied until a good sigmoid fit to the |E*| data of all temperatures is obtained. Typically, the following sigmoid function is used:

$$\log(|E*|) = b_1 + \frac{b_2}{1 + \exp(-\,b_3 - b_4 \log(f_R))} \tag{3}$$

where b_1, b_2, b_3, and b_4 are the sigmoid coefficients, and f_R is the reduced frequency.

3.5. Push-Pull fatigue tests

Push-Pull (PP) is a uniaxial compression-tension fatigue test performed on the same or similar size cylindrical samples as |E*| testing. The benefit of running the PP test is the ability to use the same |E*| samples. Since the |E*| tests are non-destructive, the same samples can be used for PP fatigue tests as well. In this study, however, a different set of samples were tested for |E*| and PP fatigue tests in order to accelerate the testing time by running both tests simultaneously. Preparation of the PP test samples required paying special attention. Attaching the bottom and top plates were done using an in-house manufactured gluing jig. If the plates were not glued parallel to each other, the end failure of the samples was unavoidable due to the eccentricity. Furthermore, improper gluing might lead to the detachment of plates during the first couple of loading cycles. The cylindrical samples were equipped with steel tabs around the circumference to accommodate the linear variable differential transmitters (LVDT), which recorded the linear displacement induced. The material testing system (MTS) performed the PP tests at controlled-actuator displacement mode by straining an average of 200-microstrains on-specimen LVDTs. The PP testing frequencies were selected as 1 Hz or 5 Hz and the testing temperatures ranged between 10°C and 23°C.

4. Results and discussions

4.1. Linear viscoelastic characterization of asphalt mixtures

Linear viscoelastic characterization of asphalt mixtures is obtained from dynamic modulus master curves. Fig. 1 illustrates the comparison of master curves of asphalt mixtures in a log–log scale at the reference temperature of 21°C.

As shown in Fig. 1, at high temperatures/low frequencies, CRTB had the highest stiffness, followed by the PG58-28 and CRWET. The PG58-34 and DVR mixtures had very similar stiffness values and lower than the rest of the mixtures. At low temperatures/high frequencies, CRTB again had the highest stiffness, followed by the PG58-28 mixture. The behavior of the CRTB mixture is somewhat unexpected since the CR modified mixtures are usually softer at low-temperatures. This may be attributed to and explained by the other ingredients that exist within the CRTB binder, which are not disclosed to the authors because of their proprietary nature, and the existence of RAP binder. Since the mixtures included 40% RAP, the RAP binder might have hindered the properties of the CRTB binder. The CRWET, DVR, and PG58-34 mixtures all had less stiffness than the CRTB and PG58-28. It is noted that at lower frequencies and higher temperatures, stiffer mixtures typically exhibit better rutting resistance. On the other hand, soft mixtures at low/intermediate temperatures are expected to perform better in fatigue and low temperature cracking. Table 2 shows the master curve coefficients of the asphalt mixtures at the reference temperature of 21°C.

$|E^*|$ tests and mastercurves of the mixtures computed here are used in the very first steps of PP-VECD.

4.2. Viscoelastic continuum damage (VECD) analysis of push–pull (PP) tests

Laboratory PP fatigue test results were analyzed using the VECD model in order to acquire and compare the fatigue lives of HMA mixtures prepared at different strain levels and temperatures [28]. VECD model computes the "so-called" damage characteristics curve of a mixture from cycles-peak stress-peak strain data. The model can predict the fatigue life of asphalt mixtures at any temperature-frequency-strain level combination once the mixture's damage characteristic curve is established. In this study, PP-VECD software, which is an implementation of the VECD formulation given in [29], was utilized to perform VECD analysis [28]. As part of the analysis of the uniaxial cyclic push–pull tests using the VECD model, the mixture |E*| master curve is needed. Once the |E*| master curve is known, a single push–pull test (either stress- or strain-controlled) at a selected frequency and temperature is sufficient to calibrate the VECD model (i.e., C versus S relationship). The VECD-based analysis used in this study is explained in [27,30] and summarized in Fig. 2. The descriptions of the parameters in the equations in Fig. 2 are listed below:

- In step #4,

  m is the maximum slope of the relaxation modulus versus time graph in log–log scale.

- In step #6,

  $|E^*|_{LVE}$ is the linear viscoelastic (undamaged) dynamic modulus,

  ε_0^N is the peak strain, $|E^*|_N$ is the dynamic modulus measured in N^{th} cycle,

  ε_N^R is the peak pseudostrain at N^{th} cycle,

  S_N and $S_{N+\Delta N}$ are the damage parameters in N^{th} and $(N + \Delta N)^{th}$ cycles,

  C_N is the pseudostiffness in N^{th} cycle,

  ε_N^R is the peak pseudostrain computed in N^{th} cycle,

  f is the frequency and

  I is sample-to-sample variability parameter and calculated as $I = |E^*|_{N=1} / |E^*|_{LVE}$ where $|E^*|_{N=1}$ is the dynamic modulus in push–pull test at first cycle

- In step #8,

  N_f = the number of cycles to failure,

  N_S = the number of discrete intervals of S up to the S_f where

  S_f = damage parameter at failure (or when C = 0.5 value),

  ε_0= the selected strain level,

  $\frac{dC}{dS}\Big|_{at\ S_i}$= the rate of change of C with respect to S (slope of C vs. S curve) at a given S_i

  f = the reduced frequency,

$$\Delta S_i = S_{i+1} - S_i$$

Steps 1 through 8 in Fig. 2 are repeated for each HMA mixture and the asphalt mixtures are ranked based on N_f.

Fig. 3 illustrates the number of repetitions to failure (N_f) values for each mixture computed at 200, 500 and 800 microstrains at 20°C using the PP-VECD model program. As shown, the RTR modified asphalt mixtures performed better than the PG58-28 and PG58-34 mixtures in all strain levels. The fatigue life of CRWET was almost ten times more than PG58-28 and six times more than PG58-34 mixtures on average. While the DVR behaved very similar to PG58-34 at low strain levels, it outperformed all the mixtures at higher strains. It can be concluded that the CRWET and DVR technologies are better than the PG58-34 mixture; therefore, the PG58-34 binder can be substituted with the CRWET or DVR technologies. It appears that the benefit of the CRTB can be observed at relatively low strain levels. At high strain levels, the improvement in fatigue life due to the use of CRTB technology was minimal.

It should be noted that the fatigue performance of DVR showed a better behavior as the microstrain level increased. While DVR was ranked as the third-best fatigue-resistant mixture at 200-microstrain, it was the best one at 800-microstrain. This implies asphalt roadways having heavy traffic loads can be constructed using DVR to enhance the fatigue life.

4.3. Fatigue performance evaluation using AASHTOWare pavement me design software

The fatigue performance of the mixtures was also analyzed using AASHTOWare Pavement ME Design (version 2.3) software. Since the Pavement ME uses the equations developed for a four-

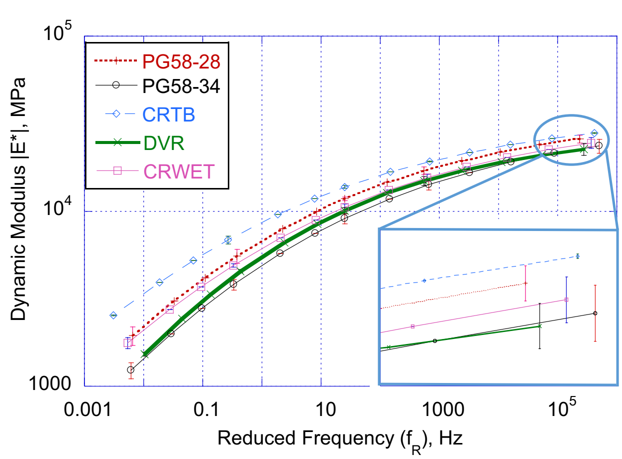

Fig. 1. Dynamic modulus (|E*|) master curves in a log–log scale at reference temperature of 21°C.

Table 2
|E*| master curve coefficients of the asphalt mixtures at reference temperature of 21 °C.

Coefficient	PG58-34	CRWET	CRTB	CRDEV	PG58-28
a_1	4.61E-04	3.66E-04	2.76E-04	4.35E-04	3.09E-04
a_2	−1.44E-01	−1.39E-01	−1.39E-01	−1.35E-01	−1.32E-01
b_1	−0.28	0.19	0.90	0.22	0.54
b_2	4.82	4.37	3.67	4.25	4.01
b_3	1.55	1.55	1.57	1.59	1.53
b_4	0.32	0.31	0.32	0.37	0.34

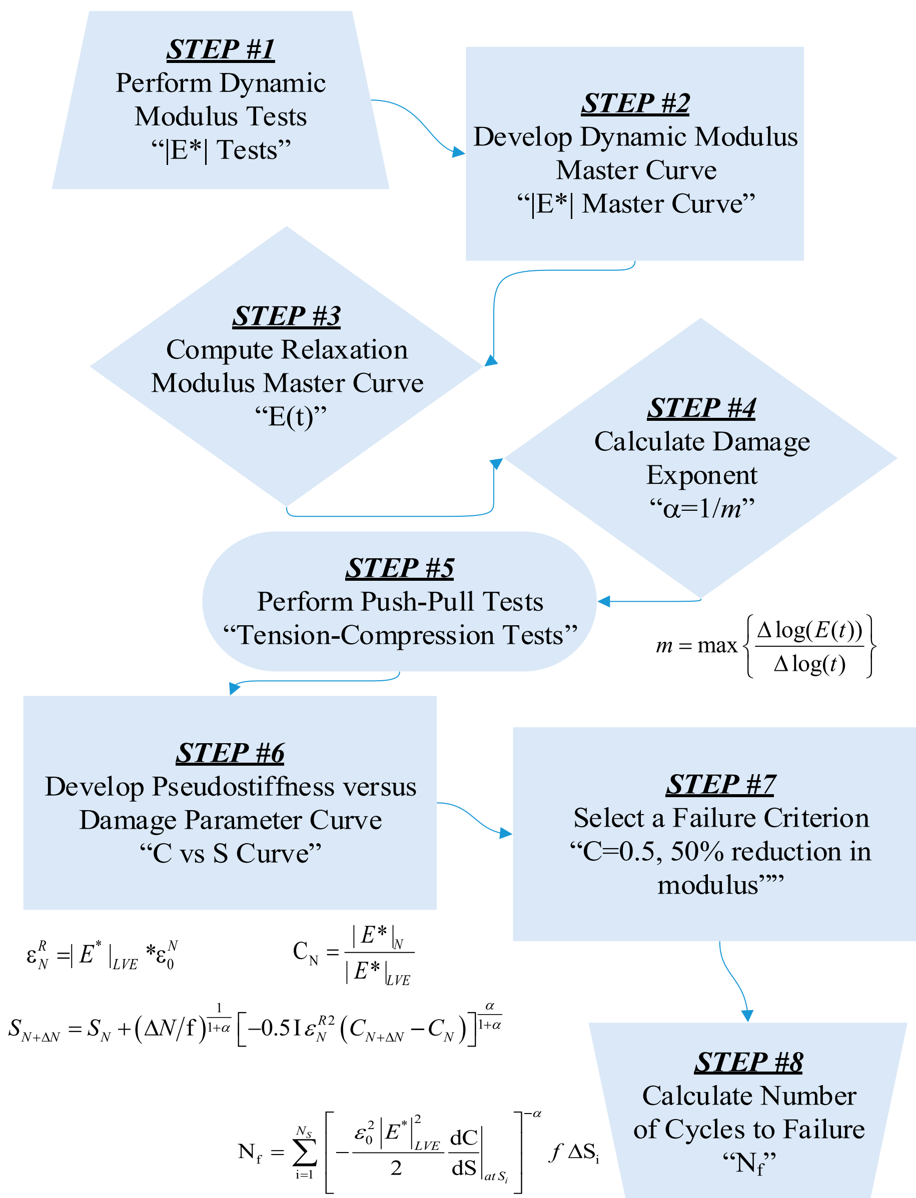

Fig. 2. Steps to perform viscoelastic continuum damage analysis.

point bending beam (FPBB) laboratory fatigue test, the first step was the calibration of parameters between FPBB and viscoelastic continuum damage (VECD) analyzed Push-Pull (PP) tests.

4.3.1. Calibration of fatigue model parameters between FPBB and VECD analyzed PP fatigue tests

The fatigue model used by the Pavement ME software was originally developed by using data obtained from FPBB tests performed at different temperatures and strain levels. Eq. (4) shows the original material model:

$$N_f = k_1 \left(\frac{1}{\varepsilon_t}\right)^{k_2} \left(\frac{1}{E}\right)^{k_3} \tag{4}$$

where N_f is the number of cycles to failure and k_1, k_2, and k_3 are the empirical constants.

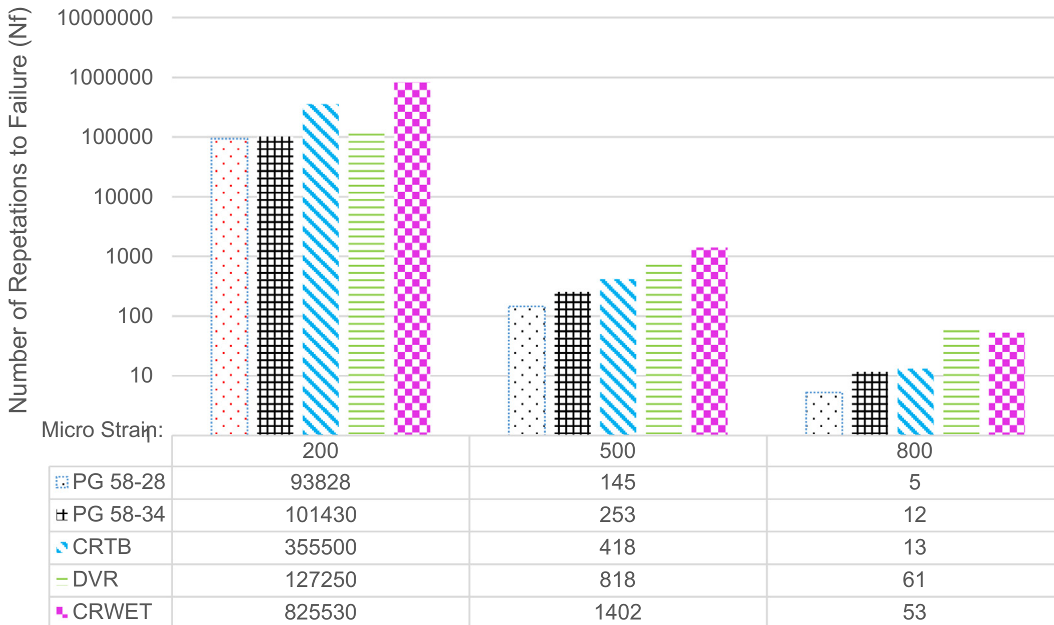

Fig. 3. PP-VECD fatigue analysis results for different microstrain levels at 20°C.

The Pavement ME software uses a modified version of Eq. (4), as listed in Table 3.

Modifications were made to better reflect the field conditions and allow calibration of the models with calibration factors β_{f1}, β_{f2}, and β_{f3}. Table 4 provides the material calibration factors for the mixtures tested.

Since the PP fatigue test was conducted in lieu of FPBB test in this study, the correlation between number of cycles to failure (Nf) for FPBB and PP given in [27] was used for conversion. Once the Nf values are calculated, the Pavement ME software uses damage accumulation models and transfer functions to calculate the actual top-down and bottom-up fatigue distresses. Table 5

Table 3
Material level fatigue damage model in pavement ME for asphalt concrete.

Distress	Revised material model in Pavement ME	Calibration factors [(1)]		Variable definitions
Bottom-up fatigue (AC)	$N_{f-bu} = \beta_{f1} k_{f1} C C_{H-bu} \left(\frac{1}{\varepsilon_{t-bu}}\right)^{\beta_{f1} k_{f2}} \left(\frac{1}{E}\right)^{\beta_{f3} k_{f3}}$	CH-bu	C, $\beta f1$, $\beta f2$ and $\beta f3$	N_{f-bu} = N_f for bottom-up cracksN_{f-td} = N_f for top-down cracksε_{t-bu} = tensile strain at the bottomε_{t-td} = tensile strain at the surfaceE = modulus/stiffness
Top-down fatigue (AC)	$N_{f-td} = \beta_{f1} k_{f1} C C_{H-td} \left(\frac{1}{\varepsilon_{t-td}}\right)^{\beta_{f1} k_{f2}} \left(\frac{1}{E}\right)^{\beta_{f3} k_{f3}}$	CH-td		

Notes: [(1)] $C = 10^{4.84\left(\frac{V_{be}}{V_a+V_{be}}-0.69\right)}$ where Vbe = effective asphalt content by volume, Va= percent air voids in the HMA mixture. $C_{H-bu} = \left(b_{bu1} + \frac{b_{bu2}}{1+e^{(b_{bu3}-b_{bu4}h_{ac})}}\right)^{-1}$
where hac = height of the AC layer and bbu1 = 0.000398, bbu2 = 0.003602, bbu3 = 11.02, bbu4 = 3.49. $C_{H-td} = \left(b_{td1} + \frac{b_{td2}}{1+e^{(b_{td3}-b_{td4}h_{ac})}}\right)^{-1}$
where hac = height of the AC layer and btd1 = 0.01, btd2 = 12, btd3 = 15.676, btd4 = 2.8186

Table 4
Material calibration factors for mixtures.

	PG58-28	PG58-34	CRTB	DVR	CRWET
$\beta f1$	0.16442	1.16565	0.168	0.166	0.17329
$\beta f2$	0.95251	0.87578	1.0021	0.9322	1.00541
$\beta f3$	0.86021	0.83272	0.9473	0.7833	0.95776

Table 5
Fatigue transfer functions and distress equations in pavement ME.

Distress	Damage Accumulation Model	Transfer Function	Field Calibration Factors
Bottom-up fatigue(AC)	$D_{bu} = \sum_{i=1}^{TP} \frac{n_i}{N_{f-bu-i}}$	$FC_{bu} = \left(\frac{1}{60}\right)\left(\frac{C_{4-bu}C_4^*}{1+e^{C_{1-bu}C_1^*+C_{2-bu}C_2^*\log D_{bu}}}\right)$	$C_{1\text{-bu}}$, $C_{2\text{-bu}}$, $C_{4\text{-bu}}$
Top-down fatigue(AC)	$D_{td} = \sum_{i=1}^{TP} \frac{n_i}{N_{f-td-i}}$	$FC_{td} = \left(\frac{1}{60}\right)\left(\frac{C_{4-td}C_4^*}{1+e^{C_{1-td}C_1^*+C_{2-td}C_2^*\log D_{td}}}\right)$	$C_{1\text{-td}}$, $C_{2\text{-td}}$, $C_{4\text{-td}}$

D_{bu} = Bottom-up crack damage, N_{f-bu-i}= N_f for bottom-up cracks for period I,
D_{td} = Top-down crack damage, N_{f-td-i}= N_f for top-down cracks for period i
TP = number of periods, n_i = traffic cycles in a period. $C_1^* = -2C_2^*$, $C_2^* = -2.40874 - 39.748(1+h_{ac})^{-2.85609}$

illustrates the damage accumulation models and the transfer functions used in the program.

4.3.2. Traffic, climate and pavement structure inputs in pavement ME

The traffic, climate, and pavement structure inputs for Pavement ME software were selected based on a possible construction project. The most feasible mixture in this research study was evaluated to be constructed on one of the county roads in the City of Lansing, MI. In this effort, the actual design traffic information was gathered to be entered into the software. The initial two-way average annual daily truck traffic (AADTT) was calculated as 8200. The construction offered only one-lane in the design direction. While the trucks in design direction was expected at 50%, trucks in the design lane were 100%. The operational speed was 50 km per hour (km/h) and the roadway was designed for 20 years. The climate data were obtained from station #14863 (Lansing, MI). Other information/inputs were maintained at their default values.

Bottom-up (BU) and top-down (TD) fatigue cracking susceptibility of thin and thick pavement structures were analyzed. Both structures included a 150 mm (6″) thick unbound base (AASHTO soil classification: A-1-a) overlain by a semi-infinite subgrade layer. The AC layer thicknesses for the thick pavement structure were selected as 50-mm surface course, 50-mm intermediate course and 150-mm for the surface course. The AC layer thicknesses for the thin-layered structure were 50-mm and 50-mm for the surface and base courses, respectively.

4.3.3. Mixture and binder level-1 data inputs

Dynamic Modulus, $|E^*|$, values of the asphalt mixtures and Complex Shear Modulus, $|G^*|$, and phase angle, δ values of the asphalt binders were input as level-1 for flexible pavement analysis using Pavement ME software. $|E^*|$ of each mixture was obtained by averaging at least 3 replicates tested at 5-temperature and 6-frequency levels. Binder's rheological parameters $|G^*|$ and δ are obtained by performing temperature-frequency sweep tests. In order to cover a wide range of temperatures, tests were performed by using 8-mm and 25-mm parallel plate geometries. Although $|G^*|$ and δ are acquired for a range of frequencies at various temperature levels, only the values at 1.59 Hz (10 rad/sec) at each temperature entered as inputs into the software. Table 6 shows the PG58-28 mixture $|E^*|$ values and binder $|G^*|$ and δ used as level-1 input in Pavement ME software.

4.3.4. Asphalt concrete (AC) bottom-up (BU) and top-down (TD) cracking results

The AC bottom-up (BU) cracking is one of the most common distress types that occurs on the flexible pavement over time. It is also known as classical fatigue cracking or alligator cracking, and it is a detrimental crack type, especially for thin-layered asphalt pavement. Thin AC structure easily develops flexural bending at the bottom of the pavement, where the pavement experiences high tensile bending stresses. As a result, the BU cracks initiate and propagate to the surface as longitudinal cracks. Over time, the longitudinal cracks connect and form an alligator skin pattern. Further deterioration of the pavement surface results in potholes.

Asphalt concrete top-down (TD) or longitudinal fatigue cracking is another type of pavement distress commonly observed on flexible pavements. This type of crack is generally observed on thick pavement layers due to high-localized tensile stresses between tire and pavement interaction.

Both the BU and the TD fatigue cracking of the thin- and thick-layered asphalt pavements were studied. The target distress reliability was set at 90% for both crack types and pavement structures. While Fig. 4(a) shows % BU cracking for thin pavements layers, Fig. 4(b) illustrates the results for thick-pavement structures.

The results are provided for total percent cracking for 20 years (240 months) design life. The results revealed that all mixtures for both structures performed well based on the selected reliability level. The RTR modified mixtures were better than the non-modified mixtures, especially for thick pavement structure. While the DVR outperformed all other mixtures for thin-layered pavements, the CRTB mixture showed the best fatigue resistance by accumulating only 2.34% BU cracking in thick-layered structure.

Since the AC-BU cracking is a more critical distress type for thin pavement sections, using DVR mixtures can enhance the fatigue life of asphalt pavements. This conclusion and mixtures' fatigue performance are very similar to the results obtained using the PP-VECD model.

Fig. 4(c) and Fig. 4(d) demonstrate the asphalt pavements top-down cracking for thin- and thick-layered analysis, respectively. In the thin-layered analysis, PG58-34 mixtures failed based on 90% distress reliability. It took only 96 months to reach the TD threshold value of 387.8 m/km. PG58-28 mixture barely achieved the threshold value by accumulating 374.1 m/km at the end of 240 months. For the thick-layer analysis, PG58-34 was the only mixture failed at the end of 168 months. RTR modified mixtures showed better TD cracking resistance as they did for BU cracking resistance. CRTB was the superior mixture for both layer analyses.

4.3.5. Fatigue life comparison of the mixtures by using pavement ME design

The impact of mixture type on the fatigue lives is investigated by using Pavement ME analyses for thin- and thick-layered pave-

Table 6
PG58-28 mixture $|E^*|$ and binder G^*-phase angle data used in pavement ME.

Dynamic Modulus $\|E^*\|$ Data in (Pa)						
Temperature (°C)	Frequency (Hz)					
	0.1	0.5	1	5	10	25
−10	18022.5	21608.5	22793.5	24,870	25,690	26,673
10	8247.5	10457.5	11,488	13,959	15,140	16627.5
21	4058	5546	6310	8408	9396	10,933
37	1148	1906.5	2293.5	3624.5	4301.5	5227
54	271.85	492.25	630.05	1142	1469.5	1953
Complex Shear Modulus Temperature (°C)	G*-Shear Modulus (Pa)			- Phase Angle (deg)		
15	3,133,539			55.4		
30	735,688			59.12		
46	107,333			58.36		
60	28,181			58.67		
70	7218			59.6		

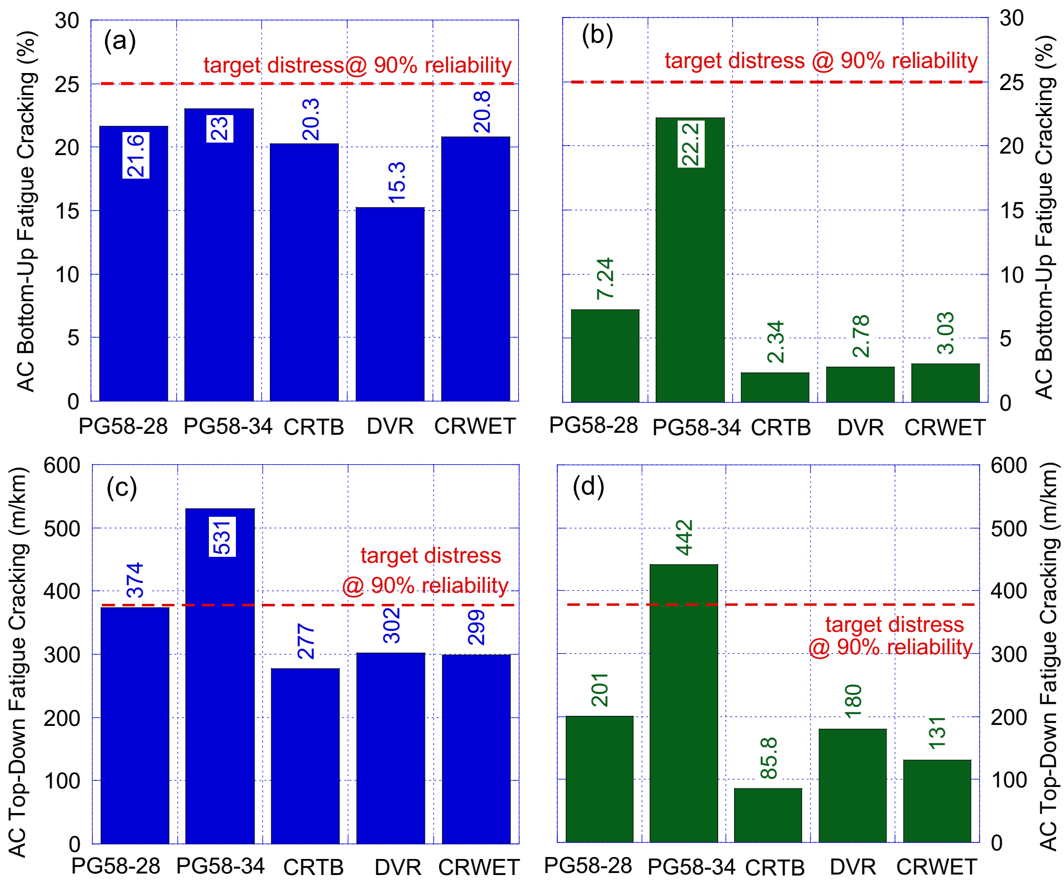

Fig. 4. (a) AC-BU cracking for thin- (b) AC-BU cracking for thick- (c) AC-TD cracking for thin- (d) AC-TD cracking for thick-layered pavement structures.

ment structures. The threshold (T) values of 10% for BU cracking, 200 m/km for TD cracking are selected to initiate the pavement rehabilitation in thin-layered pavement structures. These values are adjusted as 2% and 100 m/km for thick-layered pavement structure, respectively. In other words, the T-values for thick-layered pavement analysis are selected to be 5 times lesser for TD cracking and 2 times lesser for BU cracking than the T-values of thin-layered pavement structures. The T-values are randomly chosen to provide a comparison between mixtures and can be adjusted by considering various parameters, such as the availability of funds for rehabilitation and priority of the pavement sections/locations.

Fig. 5 shows the (a) bottom-up and (b) top-down fatigue cracking change of mixtures over design life for thin-layered asphalt concrete pavement structures. Based on the data provided and selected T-values of 10% and 200 m/km for BU and TD cracking, respectively, TD cracking was critical and it governed the pavement maintenance activities. While the PG58-34 suggested pavement rehabilitation take place just 8.5 months after the initial construction, CRTB required the pavement maintenance approximately 29 more months later than the PG58-34, which offered about 4 times better TD fatigue life. Moreover, the BU fatigue life difference between PG58-34 and CRTB mixtures was around 6 times the selected threshold value.

Fig. 5(c) and Fig. 5(d) illustrate the change in bottom-up and top-down fatigue cracking for thick-layered asphalt pavement structures over the design period, respectively. The specified T-values for both fatigue types are shown as well. The TD fatigue was again the governing cracking type for pavement rehabilitation initiation based on the selected T-values other than CRWET, for which the BU fatigue cracking was dominant. Rubber modified mixtures outperformed for both TD and BU cracking. The similar ranking of the mixtures observed in thin pavement analysis for fatigue life was also acquired in thick pavement analysis.

The results revealed that the maintenance initiation for PG58-34 was 15 times earlier than that of CRTB. While the PG58-38 pavement required maintenance just after 10.5 months, CRTB pavements required maintenance after 154.8 months. Even though the pricing information of the rubberized asphalt binders and mixtures are not stable to draw a conclusion at the moment and subject to change depending on various parameters, the fatigue life differences of the rubber-modified and base/original asphalt mixtures can provide a selection criterion for the authorities involved.

Table 7 summarizes the information related to the preventive maintenance initiation times for both fatigue cracking and pavement layer types based on the data presented in Fig. 5. Ultimately, in the fatigue life comparison of the mixtures using the Pavement ME program, the PG58-34 mixture showed the least favorable fatigue cracking resistance for both cracking types and pavement structures. RTR modified mixtures for thin and thick pavements showed superior fatigue cracking resistance. They lasted as little as 3 times and as much as 15 times longer than the PG58-28 and PG58-34 mixtures. CRTB mixture showed the best fatigue resistance behavior. It did not even accumulate the required TD fatigue cracking to initiate the pavement maintenance at the end of design life, which suggested no rehabilitation cost.

The fatigue cracking outcomes of both PP-VECD and Pavement ME programs are very similar even though there are slight order

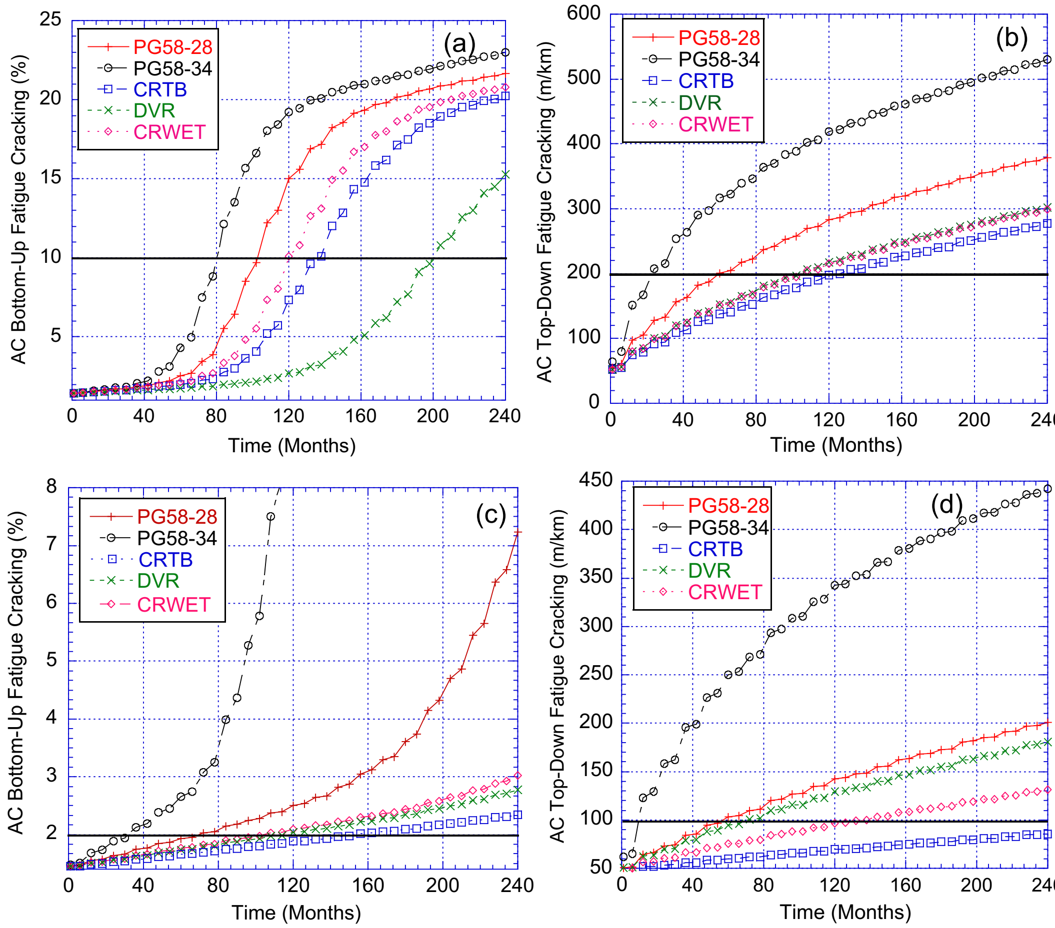

Fig. 5. Comparison of mixtures (a-c) Bottom-Up and (b-d) Top-Down fatigue cracking over time for thin- and thick-layered pavement structures, respectively.

Table 7
TD and BU fatigue lives (in months) for thin- and thick-layered asphalt pavement structures.

		PG58-28	PG58-34	CRTB	DVR	CRWET
THIN	Top-Down ("T = 10%")	13.2	8.5	32.8	23.7	24
	Bottom-Up ("T = 200 m/km")	59.7	22.8	126	104.8	106
THICK	Top-Down ("T = 2%")	58.5	10.5	154.8	71.5	141
	Bottom-Up ("T = 100 m/km")	70.6	33.8	N/A	116.6	106

changes within RTR mixtures. Both programs provide the results support the hypothesis that rubber modification improves the fatigue cracking of asphalt pavements for the selected conditions.

5. Conclusions

This paper reported an analysis of fatigue cracking performances of hot mix asphalt samples prepared with one unmodified (PG58-28), one modified to bump low temperature grade (PG58-34) and three recycled tire rubber modified asphalt binders. The fatigue behavior of the mixtures was analyzed and compared using PP-VECD and Pavement ME programs. Based on the results of the tests and analyses, the following conclusions can be made.

- Dynamic modulus test results showed that CRTB had the highest stiffness at high temperature and low frequency combinations. PG58-28 and CRWET followed the CRTB mixtures. DVR and PG58-34 had almost the same stiffness values. At low temperatures and high frequencies, again CRTB had the highest stiffness, which was followed by PG58-28. The stiffness values of CRWET, DVR and PG58-34 mixtures were the lowest and not significantly different from each other. The behaviour of the CRTB mixture at low temperatures and high frequencies is somewhat unexpected since the CR modified mixtures are usually softer at low-temperatures. This type of behaviour may be attributed to the other ingredients that exist within the CRTB binder, which are not disclosed due to their proprietary nature.
- Fatigue resistances of recycled tire rubber modified mixtures were better than the PG58-28 and PG58-34 mixtures based on the analyses results of PP-VECD and Pavement ME programs.
- PP-VECD analyses revealed that the CRWET mixture showed the best fatigue resistance at low to medium strain levels while the DVR mixture outperformed at high strain levels. This implies that DVR mixtures can be used for heavy traffic roads to enhance the fatigue life of asphalt pavements
- Pavement ME results were similar with PP-VECD analysis results at higher strain levels. They showed that both thin and thick-layered asphalt pavements constructed with RTR

technologies performed better than asphalt mixtures prepared with base/original binders for bottom-up and top down fatigue cracking.

- Overall, RTR modified asphalt technologies can be used to improve the fatigue life of asphalt pavements, especially with high RAP amounts. This can be attributed to the softening effect of the rubber modification.

CRediT authorship contribution statement

S. Kocak: Conceptualization, Methodology, Software, Formal analysis, Investigation, Resources, Data curation, Writing - review & editing, Visualization. **M.E. Kutay:** Software, Writing - review & editing, Supervision.

Declaration of Competing Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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