Fatigue characterization of Enrobés à Module Elevé with high RAP percentages

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ABSTRACT

To address the increasing traffic on Dutch highways, the application of high modulus asphalt mixtures such as EME, has proven to be an excellent concept. The use of a relative low pen binder combined with an increased binder content results in a mixture with improved stiffness, fatigue and rutting properties. In structural pavement design analysis, the improved properties result in a thinner total required asphalt thickness while maintaining the same lifetime expectancy compared to an conventional asphalt mixture.

As part for further improving this concept, a laboratory study was carried out to develop an understanding of the effect of different RAP percentages on mixture performance. Displacement controlled fatigue tests were performed at two strain levels using a four – point bending test setup. In this study one type of EME was examined containing RAP percentages of 0, 30, 50 and 70%. Additional to fatigue testing, stiffness measurements were also performed.

Results from fatigue tests were analyzed using the widely used phenomenological method and a more fundamental approach based on dissipated energy concepts. Results from both methods were used in an mechanistic – empirical design environment to calculate minimum required asphalt thicknesses for a chosen situation. In terms of mechanical behaviour, preliminary results indicate that the addition of RAP results in marginal differences between the studied mixtures.

The paper reports of the different steps and motivation behind choices. Laboratory findings are discussed and compared with existing models enabling an progress in rational pavement design.

Keywords: Functional specifications, Performance testing, Reclaimed asphalt pavement (RAP) Recycling, Structural fatigue

1. INTRODUCTION

In France in the early eighties an asphalt mixture was developed in order to reduce the pavement thickness with conservation of the good deformation- and fatigue-properties. This so-called enrobé à module élevé (EME), asphalt with high stiffness modulus, is nowadays applied on a large scale to realise thin, low-maintenance pavements. The good French experiences and the Dutch attention for sustainable development and innovation were the reason that Dura Vermeer Infrastructuur introduced this mixture in the Netherlands (Naus 1998). The first application was in 1998 at the east carriageway of the A6 (part Muiderberg - Almere Stad West). A reduction in thickness of 25 % with respect to the design with conventional asphalt was applied.
Since then EME was applied successfully in the Netherlands on larger scale. Additionally, the application of this mixture was found suitable for strengthening purposes where limits for the underpass at crossovers is critical. The thickness reduction of 25% results in lower emissions at the asphalt plant, transport and processing per square meter since the amount of material to be produced and processed is lower.
To further improve this concept, the effect of introducing reclaimed asphalt pavements, hereafter RAP, was studied for different percentages. With the introduction of higher RAP percentages the amount of to be added new materials reduces and therefore an overall lower impact on the environment.
The aim of this study was to develop an understanding of the effect of different RAP percentages on EME. For this purpose laboratory tests were performed, two different fatigue approaches were utilized and pavement design calculations were performed under Dutch conditions.

2. MATERIALDS AND METHODS

2.1 Mixture design

In this study one type of EME mixture was analyzed containing different RAP percentages. The target design curve and type of RAP was kept constant for all mixtures. To quantify the effects of different RAP percentages on mixture properties and its consequences in pavement design analysis, an reference base layer, AC 22 Base 50% RAP, was also studied.
The type of stone and sand was kept constant for all the mixtures. In the EME mixtures a limestone filler was used while in the reference mixture a composite filler was used containing hydrated lime. The total bitumen content of the EME mixtures was kept constant while the reference mixture contained a lower bitumen content of 1% by mass. The added new bitumen was a pen 15/25 type for the EME mixtures while for the reference mixture a pen 70/100 bitumen was used. Due to time constraints little attention was paid to fully characterize the different bitumen's and respectively blends. The amount of new bitumen was calculated using the so called pen rule:

$$\log pen_{mix} = A \cdot \log(pen_{RAP}) + B \cdot \log\left(pen_{Virgin}\right)$$

Where
pen_{mix} = penetration of the bitumen blend (dmm);
pen_{rap} = penetration of the RAP bitumen (dmm);
pen_{virgin} = penetration of the virgin bitumen (dmm);
A, B = individual share (%); A+ B = 1;

The conventional properties of the different bitumen's are summarized in table 1.

Table 1: Conventional properties of used bitumen

ID	Penetration (dmm)	Tr&b (°C)	P.I.
Pen 70/100	80	47	-0,8
Pen 15/25	20	65	0
RAP bitumen	24	62	-0,2

It is interesting to observe the minimal difference between the field aged bitumen, recovered from the RAP, and the virgin pen 15/25 bitumen. The conventional results indicate that the "hardness" of the low pen binder and the RAP binder is almost identical. The low pen binder results from tailor made refinery processes while the RAP bitumen is assumed to be a product from "conventional" refinery processes. As a result their mechanical behavior might differ.

2.2 Sample preparation and testing

All the mixtures were prepared and tested within the scope of the so called "Type testing" according to European standard EN 13108 – 20. The sequence of adding the virgin materials and mixing time was kept constant in the laboratory. After adding the stone and sand fraction, the filler is added followed by the RAP. Hereafter the mixer is turned on and the bitumen is added gradually. The mixing time was kept constant for 4 minutes. Target mixing temperatures were set to 185°C for the EME mixtures respectively 160°C for the reference AC 22 Base mixture. The RAP was pre – heated for a maximum of 3 hours at the chosen mixing temperatures. Mixing was performed using a laboratory mixer containing two in different directions operating mixing arms and compaction was performed using a steel roller sector compaction device, see figure 1.

Figure 1: Used mixing (left) and compaction (right) equipment

After compaction, rectangular beams (50*50*450mm) were prepared out of the compacted slabs. Hereafter all the beams were stored in climate chamber at a temperature between 10 – 15 °C for 14 days and humidity around 70%. In this study the overall determined air voids varied between 2 and 3% while the average sample density of all the mixtures was 2385 kg/m³. Stiffness and fatigue tests were performed in a strain – controlled mode using a four – point bending test setup at one temperature of 20°C. For stiffness measurements, a strain amplitude of 50 µm/m was adapted at frequencies of 0.1, 0.2, 0.5, 1, 2, 5, 8, 10, 20 and 30 Hz. Fatigue tests were performed at a frequency of 30 Hz. Table 2 gives an overview of the used strain levels for fatigue testing and the number of beams tested for each case.

Table 2: Applied strain levels in fatigue tests (UL= upper level, ML= mid level, LL= lower level)

ID	UL	ML	LL
EME 0% RAP	267 (6)		155 (3)
EME 30% RAP	267 (6)		155 (4)
EME 50% RAP	262 (6)	217 (6)	156 (6)
EME 70% RAP	267 (6)		155 (5)
AC 22 Base 50% RAP	255 (6)	166 (6)	109 (5)

2.3 Fatigue models

Fatigue test results were analyzed using the widely used phenomenological method and a more rational based energy approach. The phenomenological approach relates the stress or strain in the asphalt layer to the number of load repetitions that causes failure (Pell, 1962). It is a combination of the phenomenological observation and the laboratory-tabulated data derived from designated fatigue tests. This approach has been widely used together with Miner's (1945) linear law of cumulative damage in structural pavement design because of its simplicity. Results from fatigue tests are typically formulated as follows depending on the mode:

$$N_f = A(\frac{1}{\epsilon})^b \qquad \text{for strain – controlled tests}$$

$$N_f = C(\frac{1}{\sigma})^d \qquad \text{for stress - controlled tests}$$

Where

N_f = number of load application to failure;
ε, σ = tensile strain or stress repeatedly applied;
A, b, C, d = material coefficients, derived of fitting the data

Although widely used, the traditional phenomenological approach does not provide a mechanism of damage accumulation in the mixture under the repetitive load. Furthermore, the accumulated damage is treated as linear in the strain-fatigue life relationship, which has been found incorrect at low strain/damage condition (Carpenter et al., 2003). Because it is material and loading mode dependent, this approach cannot be applied directly to the complex loading scenarios that are actually common to in-service pavements. The traditional approach also does not account for the complexity of asphalt mixture mechanisms such as healing and stress redistributions, which are known to have significant effect on the fatigue behavior of asphalt mixtures. Nevertheless it should be noted that over 50 years of experience has been build up with this approach and its application in pavement design analysis. Therefore from practical point of view when studying non traditional mixtures and/or materials, the phenomenological approach can serve as an excellent starting point to develop an estimation.

In addition to the phenomenological method, a dissipated energy approach was utilized to analyze fatigue data. Applying a load to a material, the area under the stress-strain curve represents the energy being put into the material. During the loading-unloading process, if the unloading curves do not coincide with the loading but follow different paths, an energy loss has occurred within the material. Part of the energy is dissipated out of the material system due to the external work, in the form of mechanical work, heat generation, or damage. The following equation can be used to calculate the dissipated energy:

$$DE_i = \pi \sigma_i \varepsilon_i \sin\delta_i$$

Where

DE_i = dissipated energy in cycle i;
σ_i = stress level in cycle i;
ε_i = strain level in cycle i;
δ_i = phase angle between σ and ε in cycle i;

The relative change in the amount of the dissipated energy is directly related to damage accumulation. A low amount of relative change in energy dissipation can be found either in high fatigue resistance materials, low external loading amplitudes, or both. Such relative change in dissipated energy represents the total effect of fatigue damage without the necessity of considering material type, loading modes and severity separately, Shen and Carpenter (2007).
This concept was first initiated by Carpenter and Jansen (1997) who suggested using the change in dissipated energy to relate damage accumulation and fatigue life. The work was refined and expanded by Ghuzlan and Carpenter (2000), and then well applied and verified by Carpenter et al. (2003) who used the ratio of dissipated energy change (RDEC) as an energy parameter to describe asphalt fatigue damage. This ratio can be represented as:

$$RDEC_a = \frac{DE_a - DE_b}{DE_a\,(b-a)}$$

Where

$RDEC_a$ = the average ratio of dissipated energy change at load cycle a, compared to the next cycle b;
a, b = load cycle a and b;
DE_a, DE_b= the dissipated energy produced in load cycle a and b respectively;

RDEC eliminates the energy that is dissipating in other forms without producing damage. This provides a true indication of the damage being done to the mixture from one cycle to another by comparing the previous cycle's energy level and determining how much of it caused damage.
Introduced by Ghuzlan (2001) and Carpenter et al. (2003, 2006), the damage curve represented by RDEC vs. loading cycle's shows three stages: a rapid decrease, followed by a plateau stage (stage II) for the majority of the fatigue cycles.

The plateau stage (stage II), an indication of a period where there is a relatively constant percentage of input energy being turned into damage, will extend throughout the main service life until a dramatic increase in RDEC, which gives a sign of true fatigue failure (stage III), see figure 2.

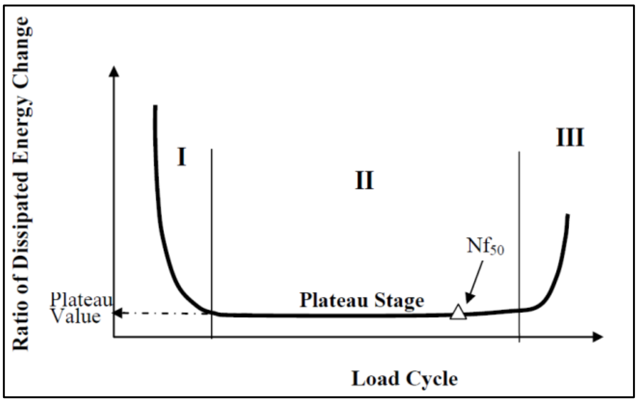

Figure 2: RDEC principle (Shen 2007)

Extensive testing (Shen 2007) showed that the failure point, 50% stiffness reduction criterion, is generally located in stage 2 of the RDEC –N curve. By plotting the corresponding RDEC value, hereafter referred to as plateau value or PV, versus the number of cycles at failure, a unique relationship was developed between the PV and number of cycle. This relationship was validated being independent of loading magnitude, material type and test method. In this study an alternative PV – N relationship was determined based on test results from the studied mixtures. The obtained relationship was further used in the pavement design analysis.

2.4 Structural pavement design

For pavement design calculations, use was made of the linear elastic multi – layer software BISAR. Each layer represents a construction layer of the pavement which is described by the Poisson constant and mixture stiffness. Calculations were performed for a heavily loaded primary road. Parameters to calculate the amounts of traffic were adapted from Dutch design guidelines. The factors which dominate the amount of traffic are summarized in table 3.
The damage factor represents the distribution of axle load groups (20 – 200 kN), the average number of axles per truck (4) and the difference in type of tire. A single tire, super single and dual wheel set with their individual wheel load and contact pressure. For detailed information, reference is made to (CROW 2012).

In the pavement design calculations, the surface layer and binder layer were fixed with an individual thickness of 50mm. The surface layer was assumed to be porous asphalt with a fixed stiffness of 3000 MPa. The binder layer was fixed with a stiffness value of 7500 MPa. Similar the foundation was assumed to be a 250mm thick crushed concrete layer with a stiffness of 600 MPa. The subsoil was assumed to contain a stiffness of 120 MPa. By fixing all these layers and their properties, the remaining required asphalt thickness was calculated.
Further on stiffness values were not corrected for traffic speed and design temperature. The measured values from the laboratory, at 20°C and 8 Hz, were used as direct input for design calculations. Being aware of the time – temperature superposition ability of bituminous materials, the authors believe that the values used, do resemble a moment in the pavement lifetime were temperature and traffic speed are such that mixture properties reach laboratory measured values.

Table 3: Composition of the studied mixtures

ID	Value
No. of trucks per day	10000
No. of working days	270
No. lanes per direction	0,9
Design life (years)	25
Growth per year (%)	2,5
Damage factor	1,7

The calculated traffic was corrected with a so – called "shift factor" to compensate for the difference between continuous loading in the lab, and rest periods between axle passages in practice. For this purpose use was made of the empirical relationship adapted from the Dutch design guidelines;

$$SF = 1 + 0{,}0000419 \cdot {V_b}^{1{,}06} \cdot pen^{2{,}45}, with\ SF \leq 4$$

Where
SF = shift factor;
Vb = total mass percentage of bitumen (%m/m 'in');
pen = penetration of the bitumen or bitumen blend (dmm);

For the phenomenological approach, allowable strain levels were calculated knowing the N-ε relationship for each individual mixture. For the RDEC approach, first the PV – N relationship was determined. After knowing the allowable PV the allowable strain can be calculated if the PV – ε relationship is known for each individual mixture.

3. RESULTS AND DISCUSSION

3.1 Stiffness and fatigue

As stated in the introduction, the objective of this study was to develop an understanding on the effect of different RAP percentages on the fatigue performance. To compare the performance of the different mixtures use was also made of the up till now gathered practical knowledge regarding beam testing to distinguish between real differences between mixtures and differences caused by scatter in experimental data.

Table 4 summarizes the measured stiffness values and fatigue performance of the different mixtures. It can be observed from the results that the addition of 30% RAP increases the stiffness with approximately 10%. This increase is not observed when stiffness values of the 30% and 70% RAP mixtures are compared. When the standard deviation for the individual mixtures is also considered, the results indicate that the difference between the EME mixtures is marginal.

The difference in stiffness values between the 50% RAP mixtures is believed to be the result of the difference in type of virgin bitumen. The EME 50% RAP mixture has a bitumen content which is 1% higher then the AC 22 mixture. Since the RAP is constant, the difference can only be explained by the difference in type of bitumen. With the addition of RAP, a lesser amount of the pen 15/25 bitumen is added to the mixtures and therefore the final performance of the mixture is believed to be dominated by the properties of the RAP bitumen.

Since the RAP bitumen and pen 15/25 contain identical conventional properties, the marginal differences in mixture stiffness values indicate that both binders and/or their blends show similar mechanical behavior. No additional tests were performed to confirm this.

Table 4: Measured mixture characteristics

ID	Smix 20°C; 8Hz (MPa)	ε6 (µm/m)
EME 0% RAP	10979 ± 882	165
EME 30% RAP	12318 ± 265	163
EME 50% RAP	11919 ± 713	166
EME 70% RAP	11863 ± 342	160
AC 22 Base 50% RAP	8512 ± 457	119

Identical as the stiffness results, ε6 values do not differ which imply identical fatigue performance. Further on figure 3 illustrates the marginal differences in fatigue lines between the different mixtures. However the effect of difference in slope values can best be described by the EME 0% RAP mixture. When compared with the remaining EME mixtures, it can be observed that the allowable number of cycles at the higher strain levels is the highest for this mixture and the lowest at the lower strain levels. Although Smix and ε6 values indicate similar behavior between the EME mixtures, in structural design analysis the phenomenological approach might result in under or over estimated asphalt thicknesses.

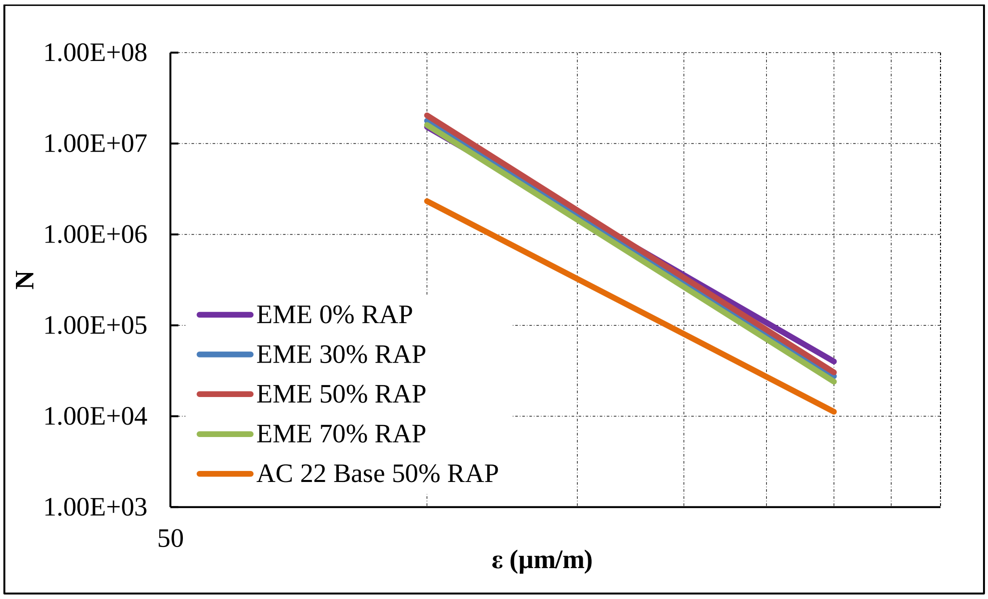

Figure 3: Fatigue results phenomenological approach

In figure 4 an example is given of the RDEC versus number of cycles plot for the EME mixture with 0% RAP. In this experiment the plateau value, PV, was determined by calculating the average RDEC value within a fixed interval. For this purpose the reduction in mixture stiffness was used as an indicator. For all the studied mixtures, the PV was consequently calculated in an arbitrary chosen interval between 80 and 65%, see figure 4.

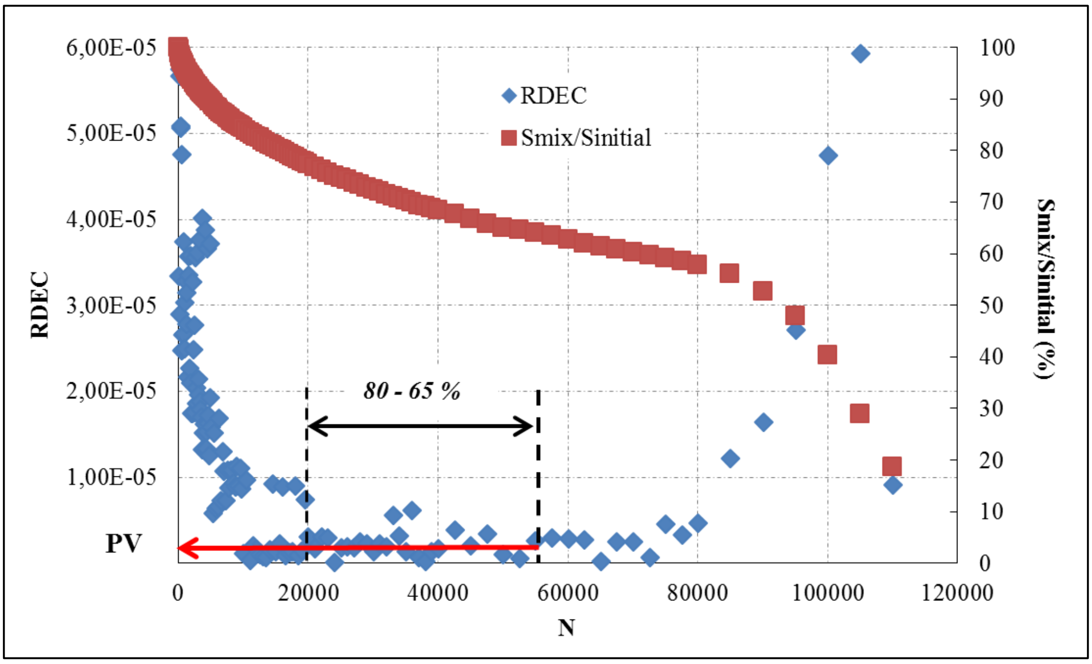

Figure 4: Calculation principle of PV (example: EME 0% PR)

In identical manner the PV was calculated for all the tested beams. Outliers were identified by using Peirce's criterion (Ross 2003).

The final results of the PV calculations are summarized in the figures 5 and 6. When average values are compared, a clear trend is observed with increasing RAP percentages. This increase in plateau value would indicate a decrease in fatigue performance. When the scatter is taken also taken into consideration, no difference between mixtures is observed. At the lower strain level, figure 6, the same trend is observed.

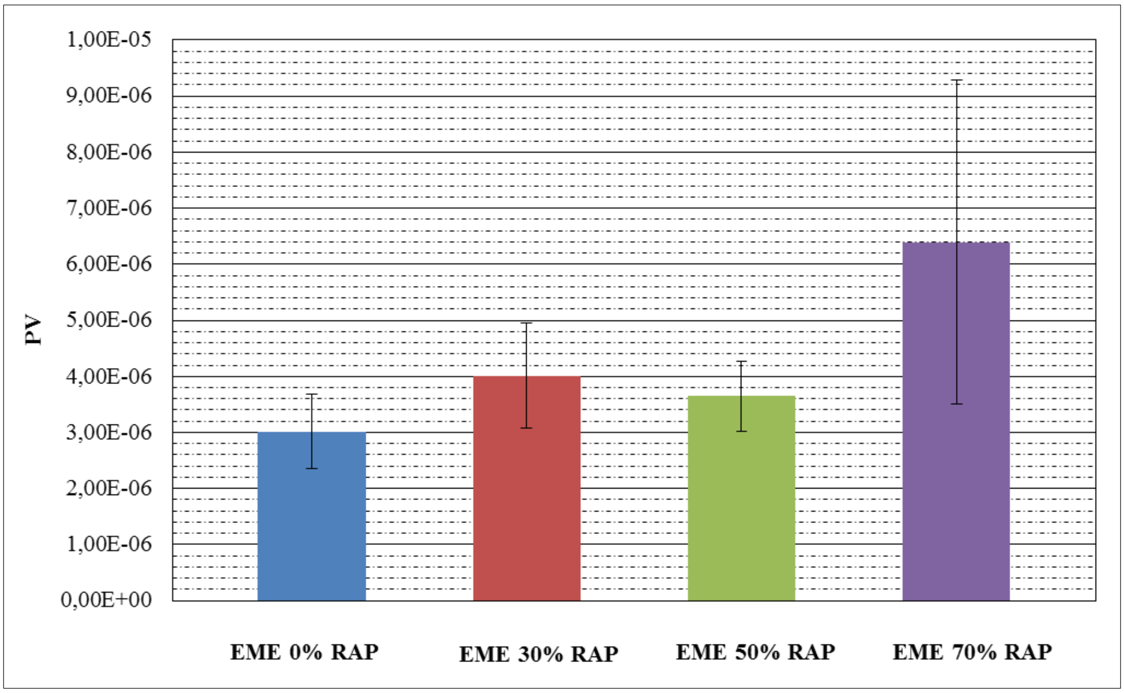

Figure 5: Fatigue results, RDEC approach (upper strain level)

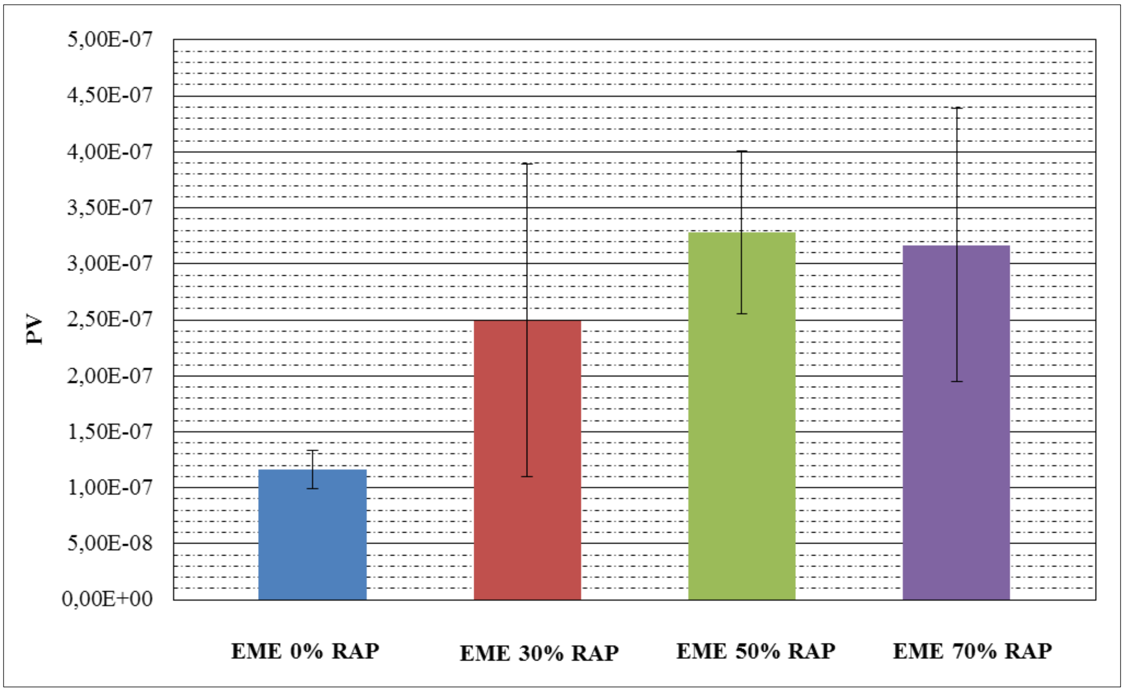

Figure 6: Fatigue results, RDEC approach (lower strain level)

In figure 7 the obtained PV-N relationship in this experiment is compared with the unique PV – N relationship found in literature (Shen 2007). It can be observed from figure 7 that some difference exists between the two models. Although the measured model is based on limited results, the authors believe that the differences are caused by the difference in test methods, and data processing techniques. Nevertheless the obtained results confirm the unique relationship between the plateau value and the number of loading cycles.

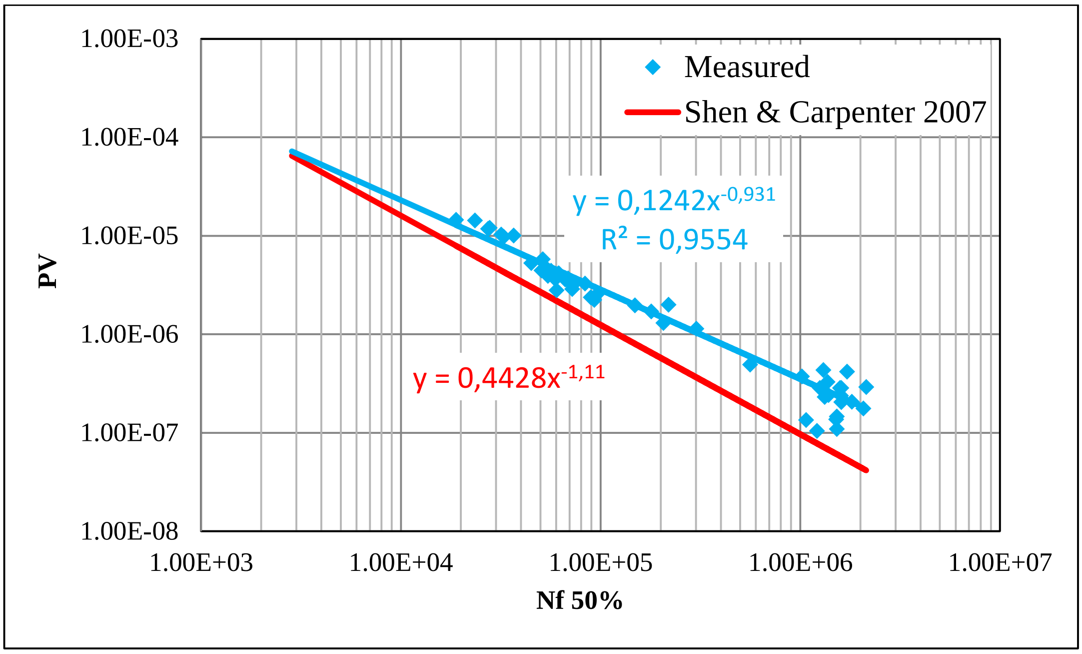

Figure 7: Comparison of PV – N relationships

After determining the PV-N relationship, an attempt was made to use the model in a mechanistic – empirical design environment. For this purpose use was made of the method proposed in the NCHRP Project 9-44. Calculations were performed for a chosen situation using input from both the phenomenological approach and the energy approach. The following section discusses the design considerations.

3.2 Pavement design analysis

Pavement design calculations were performed for a heavily loaded primary road. Occurring strains at the bottom of the base asphalt layer were calculated using BISAR. As already shown in table 4 and figure 3, the fatigue results described by the phenomenological approach did not differ significantly between the EME mixtures.
For the energy approach, relationships between the PV and the applied strain were derived from the data in order to calculate allowable strain levels at the bottom of the asphalt layer.
A power function was fitted in the test data.
Figure 8 and table 5 compare the obtained PV – ε relation of the different mixtures.

Table 5: Obtained model parameters PV – ε relationship

ID	A	B	R^2
EME 0% RAP	9E-21	5,9793	0,98
EME 30% RAP	6E-19	5,2834	0,94
EME 50% RAP	2E-17	4,6646	0,98
EME 70% RAP	3E-19	5,4692	0,94
AC 22 Base 50% RAP	4E-17	4,7726	0,98

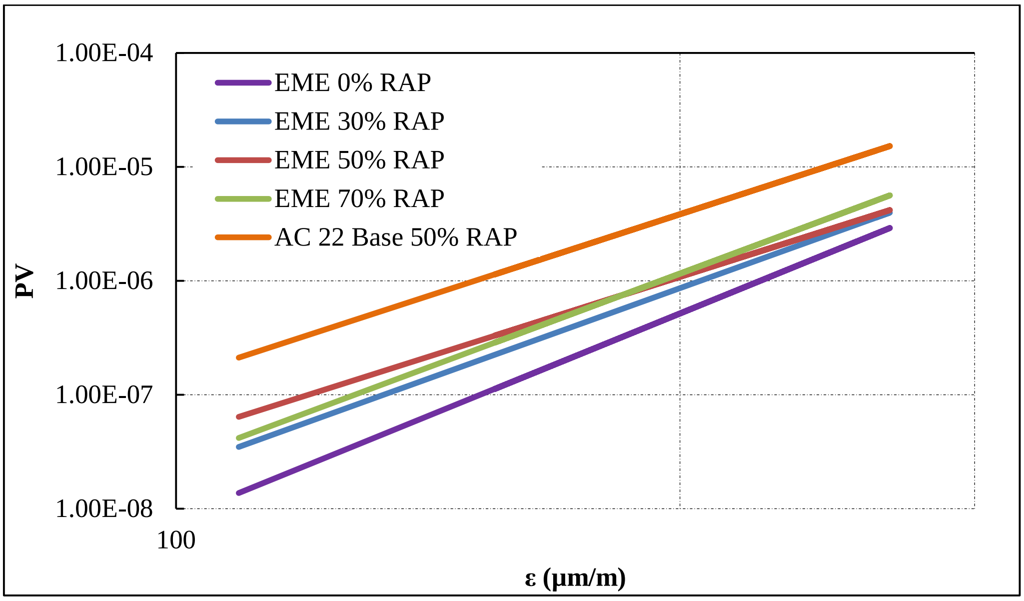

Figure 8: Comparison of PV – ε relationships

It is pointed out that the obtained relationships in figure 8 are based on limited testing and therefore should be considered with care. Here the obtained relationships are used for comparison purposes.
Knowing the PV – N relationship, the allowable PV was determined for the chosen traffic level. Substituting this value in the obtained relationships from figure 8, the allowable strain levels and thus the minimum required asphalt thicknesses were calculated, see figure 9
It can be concluded from figure 9 that the total required asphalt thicknesses do not vary significant between the different EME mixtures. The observed differences fall in between the "natural" variation of layer thicknesses in practice. This observation holds for both the phenomenological and the RDEC approach. Between the two approaches some difference is observed. The authors believe that additional testing is required to explain these differences. Finally it can be concluded from figure 9 that a widely use layer reduction of 25 – 30% remains valid for all the studied EME mixtures.

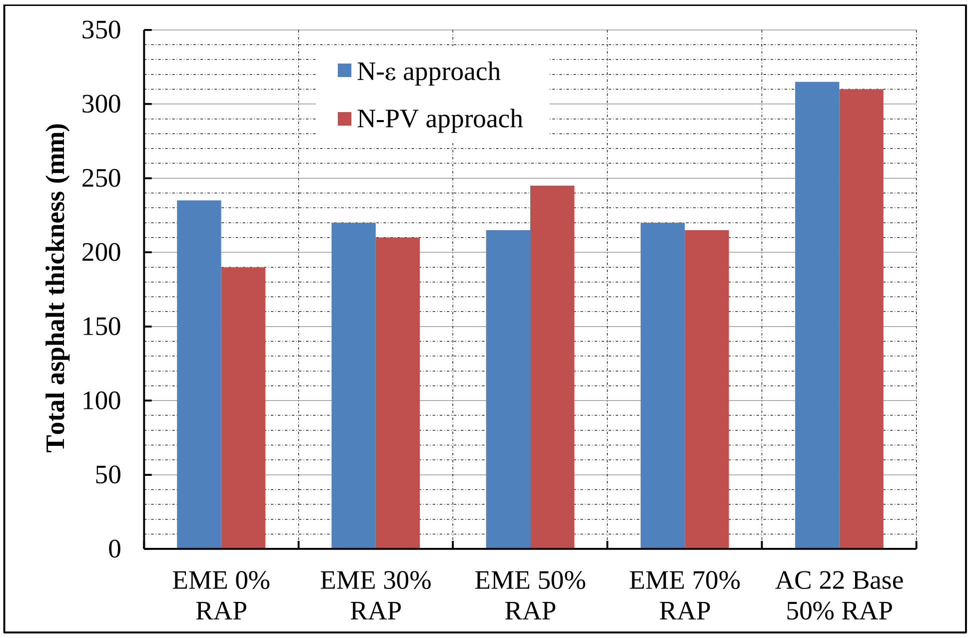

Figure 9: Comparison of layer thicknesses

4. CONCLUSIONS AND RECOMMENDATIONS

The effect of increasing RAP percentages on mixture stiffness and fatigue performance was studied for one type of EME. For this purpose use was made of a four point bending test setup. Results from fatigue tests were analyzed using two different approaches and finally differences were quantified by means of total required asphalt thicknesses for a chosen situation. On the basis of all the obtained results it is concluded that higher RAP percentages do not affect mixture performance in a negative way. Marginal differences observed are believed to be a result of data processing techniques and the natural occurring variance present in laboratory testing. Additional testing is recommended to further prove this.

The RDEC approach is a promising method to describe the fatigue of asphalt mixtures. Its applicability in structural pavement design analysis was demonstrated. For this purpose a relationship between the applied strain level and plateau value needs to be established for the mixture in consideration. It is recommended to further explore if this relation is independent of the test method.

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