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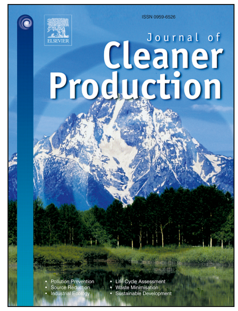

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Full-scale validation of bio-recycled asphalt mixtures for road pavements

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Abstract: Recycling of asphalt has become a well-established practice in many countries, however the road pavement industry remains a bulk consumer of extracted raw materials. Novel solutions that find root in circular economy concepts and life-cycle approaches are needed in order to enable optimisation of infrastructure resource efficiency, starting from the design stage and spanning the whole value chain in the construction sector. Itis within this framework that the present study presents a full-scale validation of asphalt mixtures specifically designed to ensure durability of flexible road pavements and at the same time enabling the reuse of reclaimed asphalt pavement (RAP) through the incorporation of bio-materials as recycling agent. These bio-recycled asphalt mixtures have been first designed in laboratory and subsequently validated in a real scale experiment conducted at the accelerated pavement testing facilities at IFSTTAR. Four pavement sections were evaluated: three test sections with innovative bio-materials, and a reference section with a conventional, high modulus asphalt mix (EME2). Two tests were realized: a rutting test and a fatigue test and for each of them the evolution of bio-recycled asphalt mixtures properties as well as the pavement deteriorations were recorded and studied. Evolution of the bio-asphalt mixtures was monitored for a 5 months period after paving by a bespoke nondestructive micro-coring, extracting and recovering methodology developed at the Western Research Institute (WRI). The structural health of the pavement sections was monitored through periodic falling weight deflectometer (FWD) as well as with strain gages and temperature sensors. As a result the three tailored bio-asphalt mixtures performed similarly or better than the control mixture, both in terms of property evolutions and durability.

Keywords: Bio-materials; Reclaimed asphalt; Accelerated pavement test; Monitoring; Rutting test; Fatigue test; Non-destructive binder evaluation

Full-scale validation of bio-recycled asphalt mixtures for road pavements

1 Introduction

The infrastructure construction industry is a bulk consumer of extracted raw materials. Due to geological and societal constraints, the supply of these materials is diminishing and their prices are on the rise. The limited availability of certain mineral aggregates and crude oil that are fit for the production of bitumen has become a major concern to the industry with the effect that recycling and reuse have become well established practices in many countries. Increasing reclaimed asphalt pavement (RAP) content is highly environmentally beneficial since it could avoid long distance aggregate transport and it generally allows fresh bitumen content reduction in mixes (Wayman 2012, Santos 2014, Santos 2017, Santos 2018). One of the key technical issues is to take advantages of the remaining bitumen in RAP, even if its physical state is too brittle and stiff to meet virgin binder criteria (Lo Presti 2016, Del Barco Carrion 2015). The problematic is then to find appropriate new binders and/or additives, able to re-activate the aged binder to achieve desirable viscoelastic behavior.

Another option considered by asphalt technologists to reduce the dependence from the oil industry is the introduction of alternative binders and/or other surrogate such as bio-materials.

In laboratory, it has been shown that high amount of RAP could be added as soon as the virgin binder is chosen to restore or improve the asphalt performances and the mixing procedure is optimized (Yan et al., 2019, Yan et al., 2017 and Ma et al., 2016b).

Many studies have been done on the effect of bio or conventional rejuvenators on performances of high reclaimed asphalt (RA) mixtures. Rejuvenators generally improved cracking performances (fatigue and low temperature behaviour) but as to be used carefully regarding rutting performance. The main characteristic for the rejuvenator for a good diffusion in the bituminous mixture is the viscosity (Ma et al., 2015). Bio-rejuvenators appear to act in the mixture like petroleum rejuvenators. It has also been shown in laboratory that fatigue life could be improve by increasing the RAP content (Ma et al., 2016a, Ma et al., 2017).

Nevertheless, all these studies are restricted to laboratory evaluations; no data on accelerated pavement testing evaluation are available (Zaumanis et al. 2014, Tran et al., 2017, Mogawer et al., 2013, Ding et al., 2019).

Concerning bio-binder, study made by Jimenez et al. (2017), with a bio-binder as the only neat binder in a mix with 50% of RA shown that the bio-binder have positive rejuvenating effects and the mixture have linear viscoelastic properties suitable for use in paving.In line with these recommendations, a full-scale test carried out on the IFSTTAR Accelerated Pavement Testing facility (APT) to assess innovative asphalt materials with a high recycling rate is described in the present study. The main innovation in this project is the use of bio-materials obtained from renewable bio-mass and used as recycling agents to design durable asphalt mixtures with 50% RAP.

The aim of this study is to demonstrate the possibility to implementing the bio-recycled asphalt mixtures on actual road networks. Three proprietary bio-materials, were used to perform tailored laboratory mix design and the validation was undertaken in a real scale experiment conducted at the accelerated pavement testing facilities at IFSTTAR. The following sections will illustrate both the results of the laboratory tests as well as the outcomes of the monitoring of the four pavement sections, one was a control (EME2), subjected to a year-long testing aimed at accelerating rutting and fatigue cracking of the materials

2 Tested materials

The aim of the project was to maximise the use of RAP with bio-based materials. For this purpose, three mixes were designed incorporating 50% RAP content, in association with three different and complementary innovative bio-materials (Chailleux et al., 2018):

- Mix1: designed with a bio-based rejuvenator, SYLVAROAD™ RP1000, a performance Additive, from Kraton Chemical used to treat the RAP. It was specifically designed to increase RAP content up to 100 % or to reuse very hard, low quality RAP.

- Mix2: designed with a bio-binder, Biophalt® (Pouget, 2013), from Eiffage Infrastructure, for total replacement of bituminous binder in recycling techniques.
- Mix3: designed with a bio-based additive from Iowa State University, an Epoxidized Methyl Soyate (EMS) aimed to compatibilize virgin binder and aged binder from RAP.

Aggregate grading curve and binder content were chosen using aggregate packing optimisation concepts (GB5® type) in order to maximize mix density and particle interlock (Olard 2012, Olard 2015, Pouget 2016). The properties of these three GB5-type mixes (mix 1, mix2 and mix3) were compared with the reference high modulus asphalt mix (EME2), commonly used in France.

Figure 1 presents the final grading curves of the different materials manufactured in plant, in comparison with the target theoretical grading curve of the GB5® type material. The three innovative materials present very similar grading curves, close to the reference, whereas the EME2 presents a coarser gradation. These results show that it is possible to get this particular grading curve in real asphalt plant. Table 1 presents the final compositions of the mixes produced in plant for the full scale test.

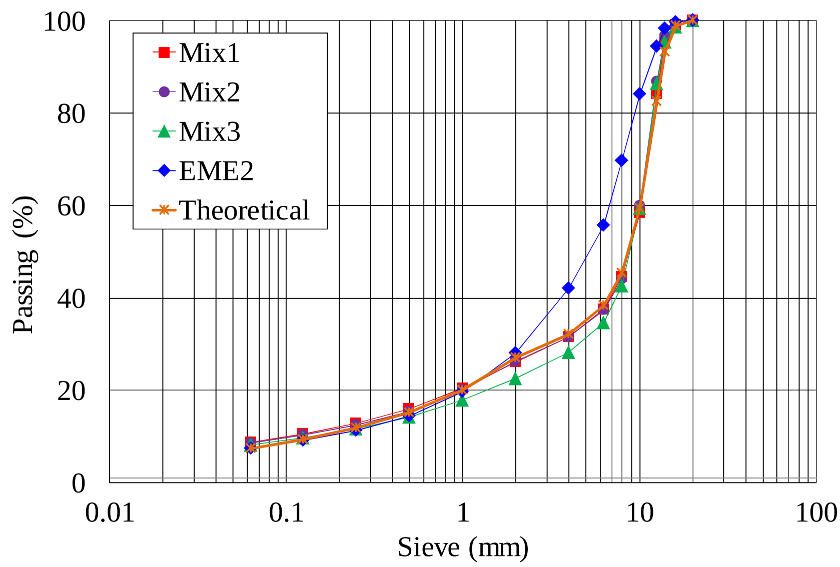


Figure 1. GB5® and EME 50% RA mixtures' gradation

Table 1. Binder content and size of aggregates of mixes produced in plant (control measurements)

Mixes	Mix1	Mix2	Mix3	Theoretical GB5 type	EME2
Binder content	4.49%	4.44%	4.36%	4.5%	5.26%
0/2 mm	17.3 %	17.7%	14.4%	19.6%	20.5%
2/4 mm	5.4%	5.3%	5.7%	5.1%	14.2%
4/6 mm	6.1%	5.9%	6.5%	6.2%	13.7%
6/10 mm	20.9%	22.6%	24.7%	21.0%	28.3%
10/14 mm	37.6%	36.8%	35.9%	33.9%	14.2%
14/20 mm	4.0%	3.2%	4.7%	6.8%	1.7%
Filler	8.8 %	8.6 %	8.1 %	7.4%	7.4 %

Three virgin aggregate fractions and two RAP fractions have been used for the mix design (see Table **2**). Binder has been extracted from both RAP fractions in order to be characterized independently. The binder content of the RAP 0/8 mm was 4.4%. The binder content of the RAP 8/12 mm was 2.9%.

The RAP 0/8 mm fraction brings 0.704% of aged binder to final mix and the RAP 8/12 mm brings 0.986% of aged binder to the final mix. The nominal binder content (4.5%) has been determined after preliminary gyratory compaction tests. By consequence, it is necessary to add 2.8% of fresh binder. The fresh binder is a conventional pure binder, 50/70 pen graded (Performance Graded PG 64-22).

It has to be noted that the RAP binder was very hard and could not be reused at a high content in a normal asphalt mix (penetration value at 25 °C < 10 x0.1mm and PG 94-4). This fact confirms the need for rejuvenation in order to be able to reuse this RA in high contents in new asphalt mixtures.

Table 2. Theoretical aggregates distribution of mixes (virgin aggregate and RAP)

		Theoretical GB5 type	Theoretical EME2
0/2 mm	Virgin aggregate	7.7%	17.5%
2/6 mm		-	17.7%
6/10 mm		-	22.1%
10/14 mm		37.2%	15.4%
Limestone Filler		2.3%	2.5%
RAP 0/8 mm	RAP	16.0%	10.0%
RAP 8/12 mm		34.0%	10.0%

All the materials produced in plant were characterised in lab using 2 point bending Fatigue tests (EN 12697-24) and 2 point bending complex modulus tests (EN 12967-26). The specimens were prepared from loose materials collected during the production. The materials were compacted using a slab compactor with rubber-tire wheels (EN 12697-33+A1, 2007), to produce 600 mm by 400 mm and 120 mm thick slabs. After compaction, the slabs were sawn to produce trapezoidal specimens, which were tested in two-point bending mode, to measure their complex modulus norm (|E*|) and phase angle (δ). Results at 15°C and 10Hz are presented in Table 3.

Two point bending test were also made to determine the fatigue parameters of these materials (EN 12697-24, 2012). The results of these tests are presented in Table 3. The fatigue parameters determined are the strain level to reach a 50 % reduction in modulus after 1 million cycles, ε_6, and the slope of the fatigue curve b.

All mixes exhibit high modulus, superior to the European criteria for this class of materials (>14 GPa for EME2 and >11 GPa for GB4). The fatigue performance of the tested mixes, measured in lab, is significantly below the reference mix.

Table 3. Mechanical properties of the plant manufactured mix (compacted in lab).

	Void (%)	Stiffness parameters (15°C and 10Hz) (EN 12967-22, 2012)		Fatigue parameters (EN 12697-24, 2012)	
		|E*| (MPa)	φ(°)	ε_6 (µstrain)	b
EME2	4.3	16770	10.3	126	-0.178
Mix1	2.9	14540	15.8	115	-0.190
Mix2	3.3	16200	16.7	100	-0.176
Mix3	4.5	16360	12.2	109	-0.156

Before construction, laboratory rutting tests were performed on mixes produced in lab following the European method (French wheel tracking test) and US method (flow number test). Results are presented in Table 4.

Table 4. Results of laboratory tests for resistance to rutting

Mixes	Rut depth EU method NF EN 12697-22+A1 30 000 cycles at 60°C	Rutting resistance (flow number) At 7% air void, T=54°C

	Rut Depth (%)	AASHTO TP-79 (cycles)
Requirements for a high modulus Mix	**< 7.5%**	Requirement medium traffic level **>190**
Requirements for a GB4 Asphalt base course material	**< 10%**	
Mix 1	5.6% (void content = 4.4%)	609
Mix 2	4.3% (void content = 3.5%)	578
Mix 3	3.7% (void content = 5.5%)	668
EME2	3.1 % (void content = 4.8%)	863

All three innovative materials met the European and US specifications for resistance to rutting. According to the European method, the results are similar for the 3 mixes, taking into account the repeatability of the tests.

3 Accelerated pavement test and monitoring procedures

3.1 Characteristics of the IFSTTAR accelerated pavement testing facility

The fatigue carrousel of IFSTTAR is an outdoor road traffic simulator designed to study the behaviour of real scale pavements under accelerated heavy traffic. The fatigue carrousel has a diameter of 40 meters and four loading arms, which can each carry loads up to thirteen tons, at a maximum loading speed of 100 km/h (Figure **2**). Two months of testing can represent up to 20 years of heavy traffic undergone by a moderate traffic pavement (150 heavy trucks/day). During loading, a lateral wandering of the loads can be applied to simulate the lateral distribution of loads of real traffic (Nguyen et al., 2013, Nguyen et al., 2017).

Figure 2. The IFSTTAR accelerated pavement testing facility

3.2 Characteristics of the experimental structures

Four different structures corresponding to materials described above, mix1, mix2, mix3 and EME2, were tested simultaneously. Due to construction constraints, it was decided to have the same mean thickness, equal to 9 cm, for all materials. The tested pavement structures are presented on **Error! Reference source not found.**. The actual thicknesses of the layers were measured at each phase of the pavement construction by means of topographical survey as well as measurements on core specimens (see Figure 4).

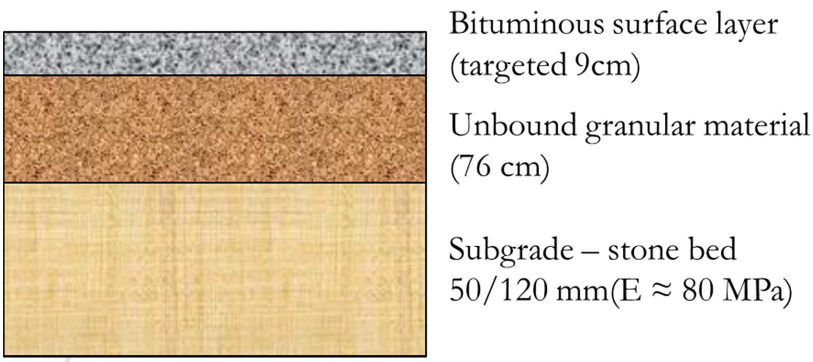
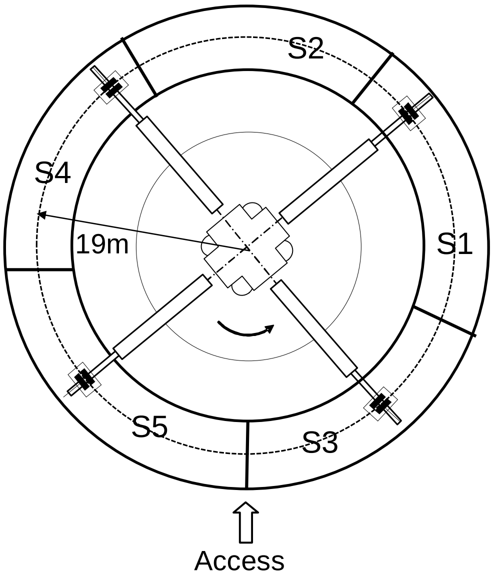


Figure 3. Tested pavement structures

The subgrade includes a stone bed (50/120 mm), and an unbound granular (UGM) subbase, consisting of 3 layers with a total thickness of 76 cm. The bearing capacity of the subgrade was measured at different positions on each structure by means of dynamic plate load test (NF P94-117-2), which gave values between 63 and 86 MPa for the stone bed, and between 103 and 111 MPa on top of the UGM layers (see Table 5).



Structures	Bituminous mixes	Averaged measured Thickness
S1	Mix1	86 mm
S2	Mix2	95 mm
S3	Mix3	91 mm
S4	Not included in this study	-
S5	Ref - EME2	102 mm

Subgrade :
Stone bed 50/120mm
+ 76 cm of UGM

Figure 4. Layout of the five tested structures on the APT facility

Table 5. Bearing capacity of soil and UGM

	Soil		Unbound Granular Material	
Pavement section	Mean Bearing capacity	Standard deviation	Mean Bearing capacity	Standard deviation
Mix1	63 MPa	6 MPa	107 MPa	5 MPa
Mix2	73 MPa	5 MPa	104 MPa	6 MPa
Mix3	86 MPa	8 MPa	111 MPa	5 MPa
EME2	69 MPa	8 MPa	103 MPa	5 MPa

The reference section was approximately 30 m long, and the other sections had a length of 22 m, and all the sections were 4.5 m wide. The inner part of the test track was used for the rutting evaluation and the outer part was used for the fatigue evaluation.

3.3 Test program

The bituminous mixes were produced and the pavements were built on the 30th and 31st of May 2017 by Eiffage.

The full scale experiment was realized in two phases:

- The first phase was performed between July and September 2017, to evaluate rutting resistance under a 65 kN dual wheel load.

- The second phase was performed between November 2017 and March 2018, to evaluate fatigue resistance of the pavements, also under 65 kN load. Right before the end of this phase, after the application of 1 million load cycles, no surface damage was observed on the four test sections. Therefore, it was decided to continue the experiment, and apply 400 000 additional load cycles, with a higher load (75 kN).

The loading conditions applied during the test are summarized in Table 6.

During the rutting test, loads were applied only when the pavement surface temperature exceeded 30°C. The pavement temperatures varied between 30 °C and 40 °C, with some short periods with higher temperatures, especially at the start of the test, where a maximum temperature of 53 °C was recorded on the surface of the pavement.

During the entire fatigue test, the pavement surface temperature varied between 36°C and -5.5°C, with most values comprised between 4 °C and 14°C. The mean surface temperature was 8.8°C, and the mean temperature in the middle of the bituminous layers was 9.0°C. These relatively low temperatures were adequate for testing fatigue resistance of the mixes.

Table 6. Loading conditions during each test phase

	Rutting test	**Fatigue test**	
Period	Summer 2017	November 2017 - February 2018	February – march 2018
Speed	43 km/h	76 km/h	43 km/h
Transverse wandering	+/- 26 cm	+/- 52 cm	+/- 52 cm
Surface Temperature: Min-Max	>30°C	-2.2°C ; 36.3°C	-5.5°C ; 26.7°C
Mean Temperature (middle of the layer)		9.2°C +/-4.7°C	8.2°C +/- 4.2°C
Load (dual wheels)	65 kN	65 kN	75 kN
Number of loads	200 000	1 million	400 000

3.4 Monitoring procedures

3.4.1 Profilometer for ruth depth measurements

Rut depth measurements were made with a profilometer equiped with a laser sensor. The transversal profile of the pavement was measured on a width of about 1.4m. The maximum rut depth value was then determined. The measurements (4 or 5 measurements per section) were performed on each section approximately every 40 000 loads.

3.4.2 Visual crack monitoring

During the experiment, pavement cracking was assessed by visual inspection on all the sections, and cracks were marked with paint, to facilitate their identification. On the APT, the extent of cracking is conventionally defined as the percentage of the length of the pavement affected by cracks. For longitudinal cracks, the “cracked length” corresponds to the measured length of the cracks. For transversal cracks, a length of 50 cm is conventionally attributed to each crack (seeFigure **5**).

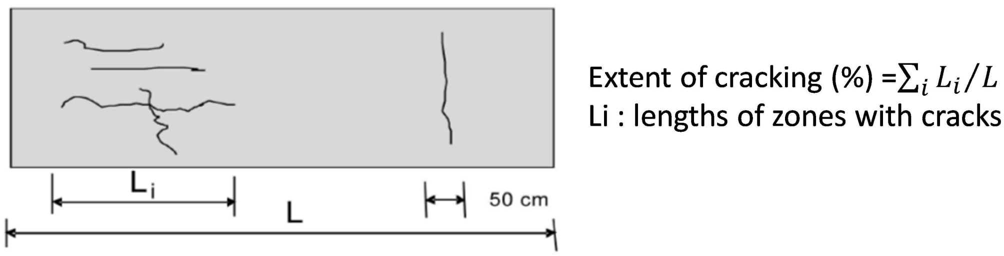


Figure 5. Evaluation of the extend of cracking

3.4.3 Pavement Instrumentation

Each test section was instrumented with the following sensors:

- 2 longitudinal asphalt strain gauges, placed during construction at the bottom of the bituminous layer, in the middle of the wheel path. The strain gauges used are KM-l00HAS (Tokyo Sokki Kenkyujo) (Duong et al., 2018). Mean signal between both gauges is considered. For each section, measurements were made approximately every 20 000 loads. Maximum strains under the passage of the 4 loading arms of the APT were determined, and the mean of these four values was calculated.
- Temperature sensors (8, placed at 4 different depths)

3.4.4 Falling weight deflectometer (FWD)

FWD Dynatest apparatus was used. Measurements were performed at 0, 500 000 loads, 1 million loads, and then every 100 000 loads, until the end of the experiment. The load was 65 kN. In this paper, the signal from the first geophone (the closest from the plate) is considered. Excepted at 0 and 500 000 loads, measurements were performed each meter.

3.4.5 Benkelman beam deflections

Deflections were measured between the two wheels, using a Benkelman beam, under the 65 kN load (or 75 kN load at the end of the test), at a speed of about 3 km/h. During each series of measurements, 4 measurements (at 4 m spacing) were made on each pavement section.

3.4.6 Non-destructive pavement micro-sampling and binder rheology

The microsampling concept presented in Figure **6** begins with sampling the pavement using a hammer drill equipped with a 12.7 mm drill bit and vacuum dust collector (FHWA HRT-15-051). For this project, about 10 holes were drilled 13 mm deep and the dust from each hole was collected to obtain about 100 g of material. This material constitutes the wearing course where most of the pavement oxidation occurs. The binder portion of the drilled dust was then extracted from the aggregate/fines portion by washing the samples in a mixture of 85:14.25:0.75, toluene:ethanol:water by volume. The samples were then centrifuged and filtered to remove the fines from the binder portion. The solvent was removed from the resulting solution using a rotary evaporator. Rheology of the recovered binders was then measured on a DSR with 4 mm parallel plate geometry (FHWA HRT-15-053).

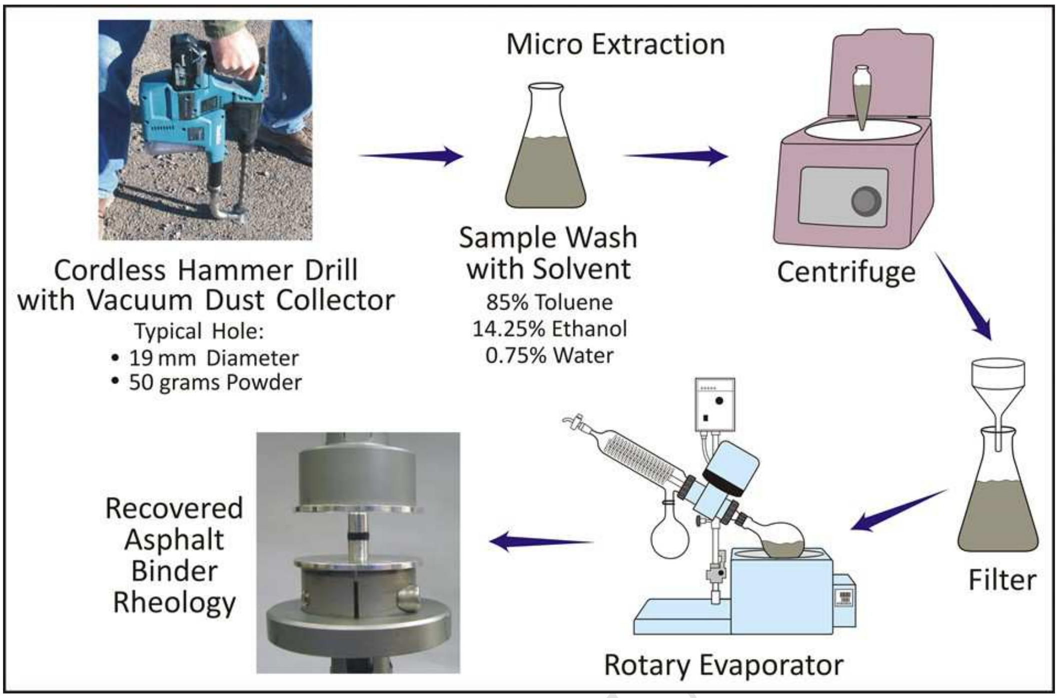


Figure 6. Micro-sampling concept

3.4.7 Monitoring timetable

The following measurements were performed on each section throughout the experiment:

- First phase (Rutting full scale test):
    - Temperature measurements, at the bottom, in the middle of the bituminous layer and at the surface, performed at time intervals of 10 minutes
    - Rut depth is measured at 0, 10 000, 20 000, 40 000 and then each 40 000, up to 200 000.
- Second phase (Fatigue full scale test) :
    - Strain gauge measurements performed approximately every 20 000 loads.
    - Temperature measurements, at the bottom, in the middle of the bituminous layer and at the surface, performed at time intervals of 10 minutes.
    - Falling Weight Deflectometer (FWD) tests: at 0, 500 000, and 1 million loads, and then at 1.1, 1.2, 1.3 and 1.4 million loads.
    - Visual crack monitoring before loading, and every 100 000 loads;

- Deflection measurements using a Benkelman beam (4 or 5 measurements per section) monitoring before loading, and every 100 000 loads;
- Microsampling and binder rheology were performed each month. Only comparison between the first month and the last is considered here.

4 Results

4.1 Performance of the tested structures during the rutting test

The evolution of mean rut depths (in %) during the test is presented on Figure 7.

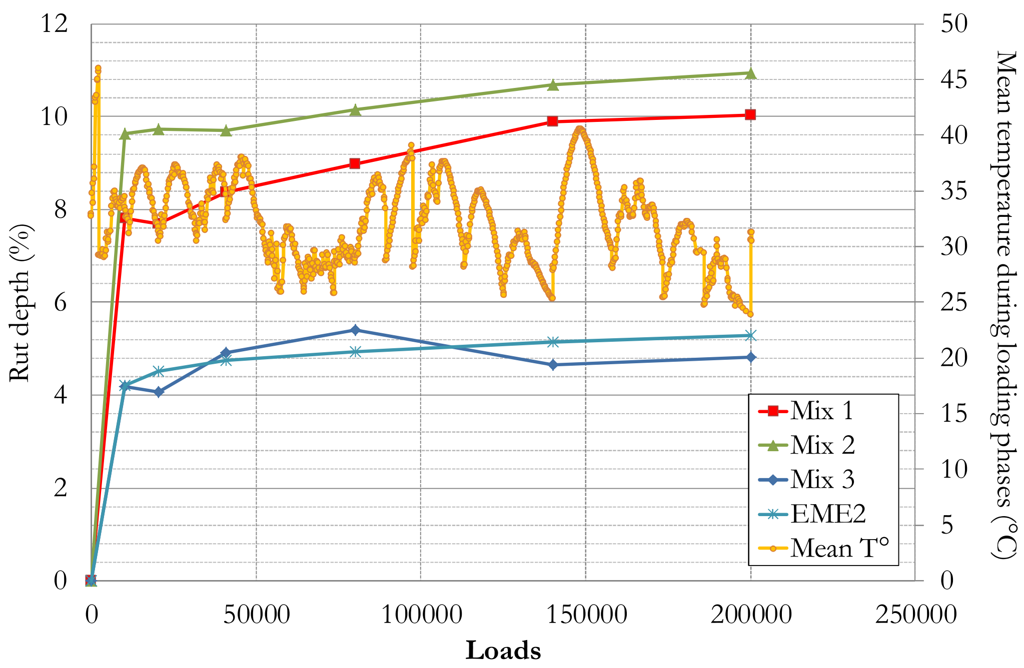


Figure 7. Evolution of rutting with the number of load cycles

Percentages of rutting are quite the same for the Mix 1 and EME2 sections (about 5%). Percentages of rutting of the Mix 2 (10.9%) and Mix 1 sections (10.0%) are more important.

Two hundred thousand cycles, with surface temperature upper than 30°C, with 65 kN dual wheel loads were applied during the rutting test, which represents about 20 years of trafic (150 trucks per day) with high temperature, in the West of

France. Severe test conditions were applied: high temperatures, low speed and narrow transversal wandering. At the end of the test, it could be concluded that:

- Rut depths increased rapidly on all sections during the first 10 000 cycles. This rapid increase could be due to post compaction, and also to the higher temperatures observed during these first 10 000 cycles.
- After 10 000 cycles, rutting continued to increase, but at a much lower rate, about 1 % of increase, until 200 000 loads for the EME2 and Mix 3 sections, and 2 % for the Mix 1 and Mix 2 sections. This indicates good performance of all the materials. It is important to note that, in France, for pavement with medium trafic (150 trucks per day), rutting is considered like a severe damage if the rut depth is upper than 25 or 30 mm (in our case, about 25% of rut depth).
- The results obtained on the test track are consistent with the laboratory rutting tests. The materials presenting the best performance on the test track (EME2 and Mix 3 sections) also presented the best performance in the laboratory .

Concerning the origin of the rutting, the pavement profiles at the end of the rutting test indicate only downward deformations for all the material, which suggests more a post compaction mechanism (of the bituminous or granular layers) than a shear flow mechanism. To illustrate this, Figure 8 compares a typical rut profile oberved in the experiment (Mix 1) and a picture of a trench cut in a pavement tested at WesTrack (FHWA, 1999), showing important rutting due to shear flow, limited to the top layer. Clearly, in this test, such lateral flow of the bituminous material was not observed.

However, it is not possible to conclude from the surface observations if the rutting affects only the bituminous layers, or also the granular base. It is only during the deconstruction of the pavement that it will be possible to cut trenches in the pavements, and to evaluate the deformations of the different pavement layers.

Figure 8. Typical rut profiles obtained in the experiment (left), and comparison with an example of rutting due to shear flow (Wes Track experiment) (right)

4.2 Performance of the tested structures during the fatigue test

4.2.1 Visual crack monitoring

Figure 9 presents the evolution of the extent of cracking, as a function of the level of traffic, on the four pavement sections. On the EME2 section, the first cracks were observed after 900 000 load cycles. Until 1.4 million loads, cracking increased regularly on this section, reaching 28% at the end of the test.

On the Mix3 section, the first cracks were observed after 1 000 000 load cycles. Until 1.4 million loads, the extent of cracking increased regularly, reaching 10% at the end of the test. On the Mix1 and Mix2 sections, no cracks were observed until the end of the test.

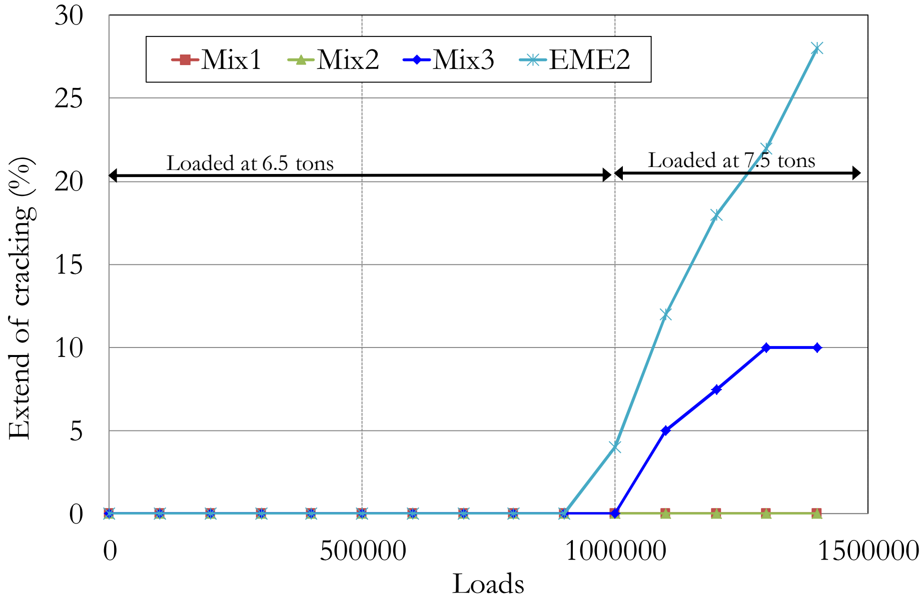


Figure 9. Extend of cracking, in percent, on the four sections

Pictures of the EME2 and Mix3 sections at the end of the test are presented on Figure 10. On these sections, the following crack patterns were observed: first, very fine isolated transversal cracks appeared (marked in white on Figure 10). Then, under traffic, these cracks started to open, and fines started to come out. Other thin transversal cracks developed nearby. The cracks marked in blue appeared at 1.1 million loads, in orange, at 1.2 million loads, in pink at 1.3 million loads and in yellow at 1.4 million loads.

The transversal orientation of the cracks is typical of fatigue cracking observed on the IFSTTAR APT, under dual wheels for pavements with thin bituminous layers (Hornych et al., 2008).

(a) EME2 section (b) Mix3 section

Figure 10. View of the cracks on the EME2 section (a) and Mix3 section (b) at the end of the test (1.4 million loads)

These results clearly indicate that the EME2 section started to deteriorate first and that the three sections with biomaterials presented a better behaviour, despite a slightly higher layer thickness for the EME2 (see Figure 4). These results differ from those of laboratory fatigue tests, which indicated the best fatigue resistance for the EME2 material (Table 3).

4.2.2 FWD Measurements

Figure 11 presents the evolution of maximum deflections, as a function of distance on the test track, for the successive FWD test campaigns at 0, 500 000 and 1 million loads and than every 100000 loads. The temperature corresponding to each series of measurements is indicated, but no temperature correction is applied to the measurements. Deflections measured with the FWD are approximately of the same level as deflections measured with the Benkelman beam (see Figure 12).

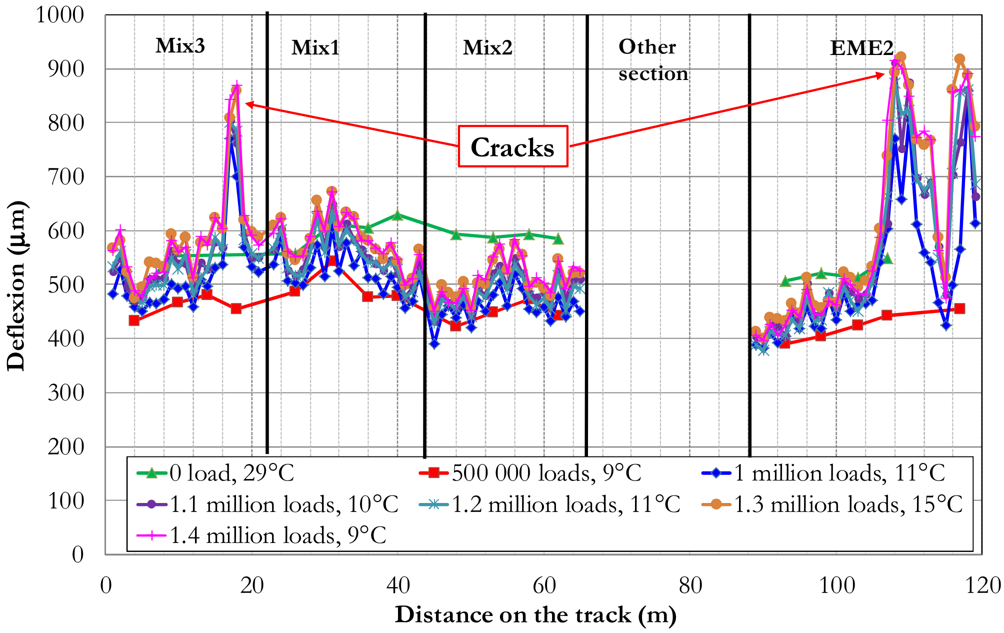


Figure 11. Evolution of deflections on the four pavement sections (measured with the FWD, geophone 1)

Figure 11 shows that :

- Between 0 and 500 000 loads, deflections decreased, probably due to post-compaction, and also to a decrease of temperature, and were quite similar on all pavement sections.
- At 1 million loads, deflections started to increase significantly at some points, on the EME2 and Mix3 sections. These deflection peaks indicate zones of damage on the pavement section. On the EME2 section, these high deflections were observed when cracks appeared at the same locations. But on the Mix3 section, high deflections were observed before the apparition of any cracks. The first cracks appeared only at 1.1 million loads. Thus, the FWD measurements were able to detect damage, before the apparition of surface cracking.
- During the next measurements, deflections continued to increase in the damaged zones, on the EME2 and Mix3 sections, and cracking also continued to develop.
- On the Mix1 and Mix2 sections, only a slight increase of deflections was observed until the end of the experiment, but no large deflection peaks, and no cracking was observed on these sections.

4.2.3 Benkelman beam deflections

Figure 12 shows mean Benkelman deflection values obtained on each section, at different numbers of load cycles, and also corresponding mean temperatures in the bituminous layers. The following trends are observed:

- During the first 100 000 loads, the deflection decreased, due to the decrease of temperature but probably also as a result of post-compaction.
- Then, up to 1 000 000 load cycles, the mean deflection levels remained close to 50 mm/100 on the four sections with some scatter probably due to temperature variations.
- At the end of the test, the deflections started to increase on all sections, but with some differences. At 1.4 million loads, the highest deflection was obtained on the Mix3 section (74 mm/100); slightly lower deflections were obtained on the EME2 and Mix1 sections (67 mm/100) and the lowest value was obtained on the Mix2 section (56 mm/100).

These differences are probably due to the initiation of damage:

- On the EME2 and Mix3 sections, cracking started at about 1 million loads, and this was consistent with the increase of deflections.
- On the Mix1 section, no cracks were observed at the end of the test, but the increase of deflection may be a sign that some internal damage occurred.
- The Mix2 section presented only a small increase of deflection, and no damage was observed on this section. It can be noted that this section also presented the lowest average FWD deflections at the end of the test.

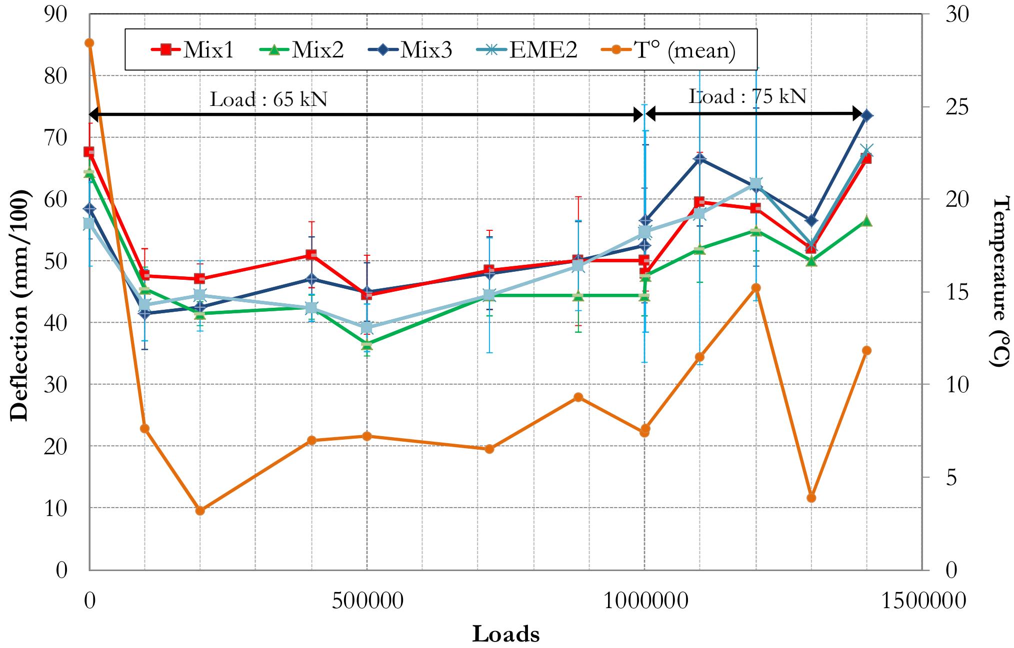


Figure 12. Evolution of deflections measured with the Benkelman beam

4.2.4 Strain gauge measurements

The evolution of the maximum longitudinal strains, for each section, is presented on Figure 13. At the beginning of the fatigue experiment, the longitudinal strains at the bottom of the asphalt layer were of the same level on the 4 pavement sections (between 100 and 130 µstrains, depending on temperature). On the Mix1 and Mix2 sections, the longitudinal strains presented no significant evolution until the end of the experiment. On the Mix3 and EME2 sections, however, longitudinal strains started to increase after about 250 000 loads, and then increased continuously until then end of the experiment, or until the gauges failed. The strain increase was more important on the EME2 section. This strain increase is certainly linked with damage of the asphalt material. It is interesting to notice that the strains started to increase well before cracks appeared on the surface of the pavements, which seems to indicate a bottom-up crack propagation.

The strain measurements at the bottom of the asphalt layers are consistent with the other measurements, indicating the development of damage first on the EME2 section, then on the Mix3 sections, and no significant evolution on the Mix1 and Mix2 sections.

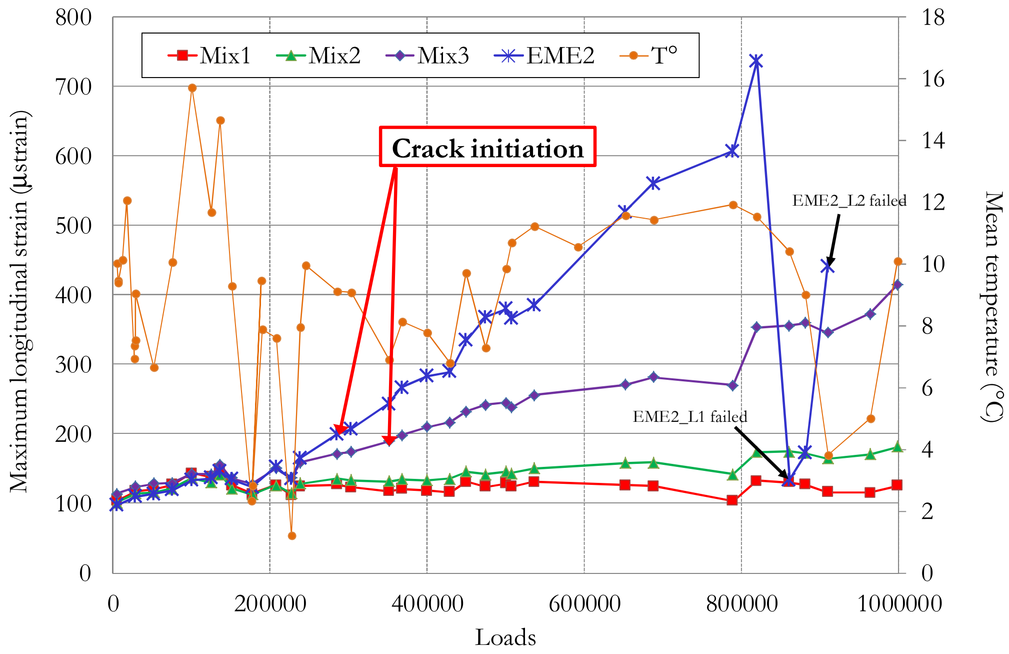


Figure 13. Evolution of maximum longitudinal strains during the fatigue test

All the measurements are very consistent, and lead to the conclusion that the 3 mixes with bio-materials present a better fatigue performance in situ in the test section pavement than the reference EME2 mix, which is considered in France as a high performance base layer material.

4.2.5 Material properties evolution assessed by non-destructive pavement micro-sampling and binder rheology

Figure **14** shows DSR results from the low temperature performance grading measurements for the extracted binders from the top 13mm of Mix1, Mix2, Mix3, and EME test section that were measured via 4 mm DSR after 5 months. These include two different components, Ts and Tm, respectively the critical temperatures for the stiffness S and the slope of the stiffness versus loading time m-value, considered as a parameter measuring relaxation ability of a binder. ΔT_c parameter is defined as Ts-Tm where a positive number indicates a stiffness controlled sample and a negative number indicates a relaxation controlled sample. Upon aging, bituminous binders are known to become more m-value controlled or more relaxation controlled as ΔT_c becomes more negative (Anderson, 2011). Discussions are ongoing in the US at the Federal Highway Administration Binder Expert Task Group on a limit for this parameter obtained from the low temperature Superpave grading using the Bending Beam Rheometer. Figure **14** shows that Mix1 and Mix2 have less negative ΔT_c values than the EME2 control and should, therefore, perform better with respect to cracking. Test track results in Figure 9 reveal this is exactly what occurred. Mix3 is a more complicated case as the volatility of the rejuvenator caused it to be removed from the binder portion of the mix during extraction and recovery meaning that it was not present in full in the binder during the rheology testing. Regardless, the corresponding extracted binder Tm, Ts, and ΔT_c are all slightly better than the EME2 control. This is also consistent with the cracking performance data in Figure 9.

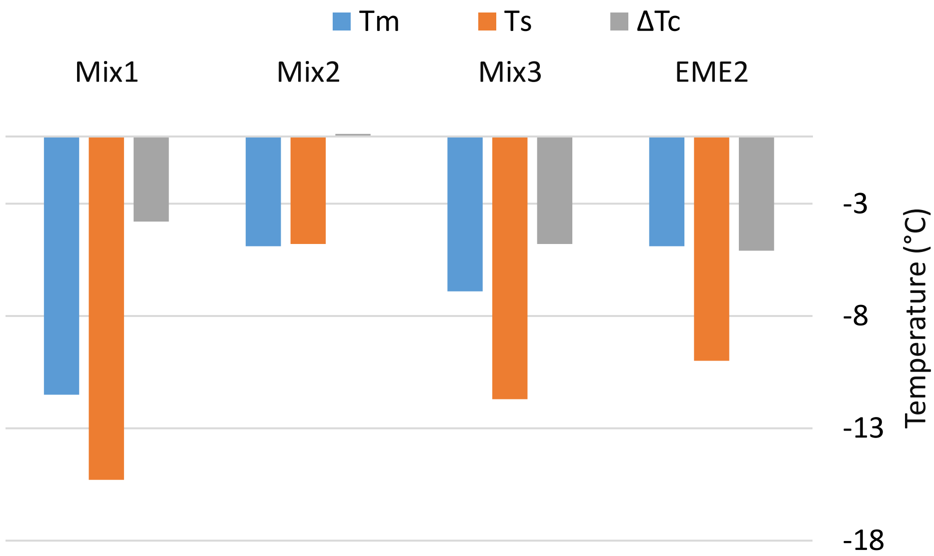


Figure 14. Low temperature DSR values of binders after 5 months on the test track

Figure 13. Low temperature DSR values of binders after 5 months on the test track.

5 Summary and findings

The results obtained on the test track are consistent with the laboratory rutting tests. In terms of rutting, all the bio-recycled asphalt mixtures performed satisfactory and the laboratory tests provided the same information obtained during the full-scale experiment: the best performance in terms of rutting are shown by EME2 and Mix 3. The results indicate good performance of all the materials. The pavement profiles at the end of the rutting test indicate only downward deformations for all the material, which suggests more a post compaction mechanism (of the bituminous or granular layers) than a shear flow mechanism.

All the measurements made in the full scale fatigue test were very consistent. On the EME2 section, the first cracks were observed after 900 000 load cycles. Until

1.4 million loads, the level of cracking increased regularly on this section, reaching 28% at the end of the test. On the Mix3 section, the first cracks were observed after 1 000 000 load cycles. Until 1.4 million loads, the level of cracking increased regularly on this section, reaching 10% at the end of the test. On the Mix1 and Mix2 sections, no cracking was observed until the end of the experiment. In addition, no rutting was observed, during the entire fatigue evaluation, on the four sections.

Finally all 4 sections were surveyed for binder evolution using a nondestructive evaluation method. The relaxation parameter ΔT_c measured on the 4 micro-sampled extracted binder blends tracked the extent of cracking in the sections. These results demonstrated the potential of the method, of the parameter and of the influence of the binder evolution in the pavement performance.

6 Conclusion

In order to understand whether bio-recycled asphalt mixtures can be implemented in road pavements, a full scale accelerated test was performed at IFSTTAR to validate the performance of the three mixes obtained by using proprietary bio-materials as recycling agents for asphalt mixture with high-content RAP. These mixes were tested simultaneously with a reference EME2 material, which is a high performance asphalt mix for base layers. From the findings summarized above is possible to conclude that

- Bio-materials of different nature could be successfully and efficiently used to partially or totally replace petroleum-based products as recycling agents to design durable recycled asphalt mixtures. This can help reducing the dependency of the asphalt industry from the oil industry
- In terms of rutting, all the bio-recycled asphalt mixtures performed satisfactory and the laboratory tests provided the same information obtained during the full-scale experiment: the best performance in terms of rutting are shown by EME2 and Mix 3. However the results indicate good performance of all the materials.

- Results of the full-scale experiment were substantially in agreement with laboratory results even if the lab fatigue results performed on mixes don't totally reflect the actual cracking.
- Cracking at full scale can be explain mainly by organic material evolution as it has been seen using the non-destructive pavement micro-sampling technic.

The three different bio-recycled asphalt mixtures present better fatigue performance in situ than the reference EME2 mix, which is considered in France as a high performance base layer material. This confirms that these mixes can be used successfully for road construction. Future work should include long term monitoring of pavements with similar bio-recycled asphalt mixtures to evaluate aging performance.

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