Effect of temperature on fatigue performances of asphalt mixes

D. Bodin, J.-P. Terrier, C. Perroteau, P. Hornych
** Laboratoire Central des Ponts et Chaussées (LCPC), Route de Bouaye - BP 4129, F-44341 Bouguenais cedex – France*

P. Marsac
*** CETE Ouest - LRPC Angers - Unité 45 matériaux et chaussées - ERA 36 Optimisation des Revêtements Routiers 23 avenue de l'Amiral Chauvin BP 20069, F-49136 Les Ponts de Cé Cedex, France*

ABSTRACT: For pavement design purpose, the fatigue performance of asphaltic materials is generally evaluated at a fixed temperature. However, previous results indicate a significant influence of the temperature on its value. A better applicability of design method to various climatic conditions needs a relevant law to take this effect into account.
In this paper, a study of temperature effects on fatigue characteristic of asphaltic mixes is reported. After a summary of previous results, a laboratory campaign on two base layer type materials is presented. For each of them, the fatigue performance ε_6 has been evaluated (EN 12697-24: 2007 – appendix A) for a wide range of loading levels, up to high loading levels for airfield pavement applications. Fatigue tests have been done at 4 temperatures, from 0°C to 30°C. The experimental results exhibit a minimum value of ε_6 function of the test temperature. The performance increases for both low and high temperatures. The resulting data base offer great prospect for advanced modeling of fatigue cracking of asphalt mixes.

KEY WORDS: asphalt mixes, fatigue tests, temperature, fatigue damage model.

1 INTRODUCTION

Fatigue cracking in asphalt concrete pavements is an important form of pavement distress. The accumulation of micro-cracks during the service life of the structure leads to macro cracks allowing the water to affect the unbound courses of the pavement and therefore the bearing capacity of the pavement. Understanding and modeling fatigue cracking of pavements over a wide range of environmental conditions without performing a large number of experiments, requires a mechanical approach of pavement behavior.
In research, advanced methods based on continuum damage modeling methods are used to model pavement structural damage under load repetition. Based on viscoelastic or elastic behavior, they integrate crack initiation, development and propagation under traffic loadings (Underwood et al., 2009 ; Sab and Zenzri, 1998). But for standard pavement engineering, mechanical approaches are classically limited to a fatigue life calculation assuming an empirical power law for fatigue life of the material, coupled with multilayer calculation of loading strains within the pavement materials. The classical power laws can be written as Eq. 1:

$$N_F(50\%,\theta,f) = 10^6 \left(\frac{\varepsilon}{\varepsilon_6(\theta,f)}\right)^{p(\theta,f)} \quad (1)$$

At a fixed frequency f and a given temperature θ, fatigue laws are fitted by linear regression in a log-log diagram on fatigue life versus the loading amplitude expressed as the maximal strain amplitude applied to the material. Several specimen sets are tested at different loading amplitude to cover the range of fatigue life between 10^4 and $>10^6$. Then the two parameters ε_6 and p are becomes material parameters which. They may be dependant on frequency and temperature. In fact, they are also dependent on test geometry and loading mode (Di Benedetto et al. 2004) and on specimen size (Bodin et al. 2006, Molenaar, 2007). However these aspects are out of the study. We focus here on there dependency with temperature θ.

Classically, material fatigue laws are evaluated at fixed conditions of temperature and frequency. However due to the strong dependence of asphaltic binders with temperature some previous results shows that $\varepsilon_6(\theta,f)$ may depend on the test temperature θ and the loading frequency f (Rao Tangella et al., 1990). In the present study, we focus on the influence of test temperature at a fixed frequency.

Such campaigns have been carried out in the past at LCPC (Laboratoire Central des Ponts et Chaussées – France) on trapezoidal cantilever beams in displacement controlled fatigue tests. Figure 1 shows the characteristic curves of $\varepsilon_6(\theta)$ which present a minimum value in a range between 10 and 20°C

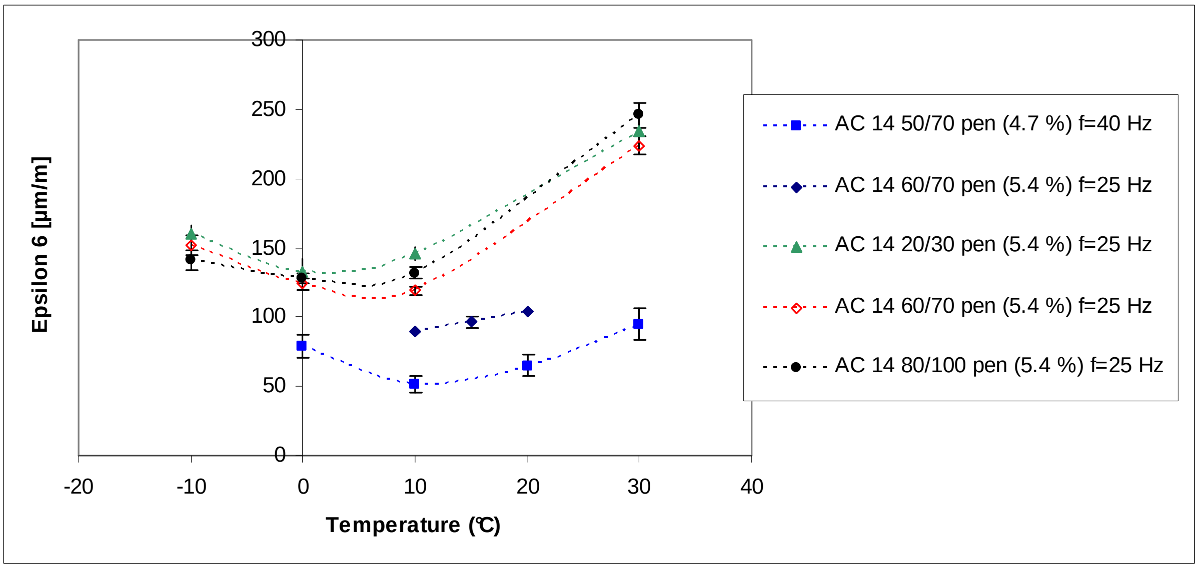


Fig 1: data available on displacement controlled fatigue tests at different temperatures (Domec, 2004 ; Moutier, 1991)

Fatigue data for temperatures around 0°C or below are lacking in the literature, however for temperatures above 10°C, results are consistent with those obtained on other fatigue test geometries reported bellow:

- Results in displacement controlled four point bending fatigue tests at 20 and 30°C for a loading frequency of 30 Hz are presented by Răcănel et al. (2009). Three specimens were tested at each temperature. For three materials, the interpolated strain level for a fatigue life of 10^6 increases between 20 and 30°C.
- Uniaxial strain controlled push-pull fatigue tests on three 0/10 asphalt concretes differing from the binder penetration grade (50/60, 70/100 and 160/220) for 0, 10 and 20°C show a rising trend of ε_6 with temperature. This trend is less significant for binder 50/60 but is more important with the increase of the penetrability of the binder used (Lunsdtrom et al. 2003). However these results have to be confirmed because of fatigue tests uncertainty (R^2 regression between 0.3 and 0.8).
- Uniaxial tests on cylindrical specimens (haversine tension tests) from 5 to nearly 30°C also show an increase of fatigue life with temperature (Underwood et al., 2009).
- Indirect tension fatigue tests could also point out the influence of temperature (Artamendi, 2009) on the cyclic response of asphalt concrete, however the loading signal is not tension compression and may also induce coupling phenomena of creep and cyclic damage.

These results show that temperature is a significant parameter when performing fatigue performance tests. However, just a few data are available in the literature. The goal of the presented paper is to extend the data base and to make a step forward in a better understanding of the mechanism affecting fatigue cracking.

2 EXPERIMENTAL APPROACH

2.1 Materials used

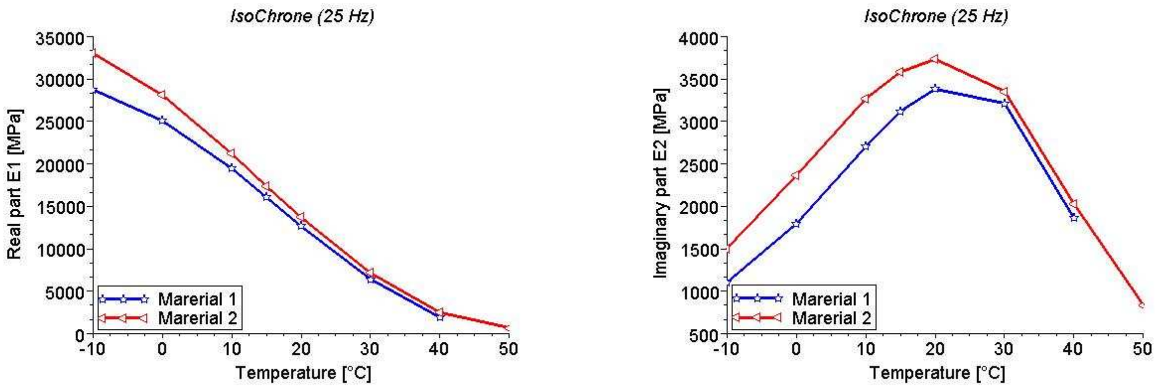
Two classical materials used for base layers are chosen to get meaningful result for pavement design and structural fatigue of asphaltic pavements. Composition is detailed in table 1. Material 1 and 2 were sampled from the field during rehabilitation works. Mix composition is reported in Table 1.

Table 1: composition of the tested asphalt mixes

Material	Grading curve - Sieve (mm)								Binder		
	0.063	0.25	1	2	6.3	10	14	16	type	content %	void %
1	na	7.9	23.4	32.3	56.4	75.6	97.1	99.8	35/50	4.70	5.6
2	6	11	25	38	45	68	98	100	20/30	5.30	3.4

Rheological behavior is evaluated using dynamic bending tests (EN 12697 – 26) for the two materials. The specimens are sawn from slabs (400x600x120 mm) manufactured with a rolling compactor according to standard (EN 12697-33). Then specimens are used to perform mechanical tests like complex modulus (EN 12697-26) and fatigue. For each temperature (from -10 to 40°C) complex modulus measurements are performed at five frequencies between 1 to 40 Hz. Fatigue tests presented further are performed at a fixed frequency of 25 Hz.

Figure 2 presents the variation of rheological properties, ie complex modulus, at the frequency of the fatigue tests (25 Hz), for temperatures between -10 and 40 °C.



(a) **(b)**

Figure 2: Complex modulus data for both materials at the frequency of 25 Hz (a) real part (b) imaginary part

The real part of the complex modulus decreases with temperature. The imaginary part exhibits a bell-shaped curve in the range of temperatures between -10 and 40°C. The maximum of the imaginary part of the complex modulus is reached at a temperature around 20°C. The variation of the modulus with temperature is also presented in Table 2 for each temperature.

Table 2: Complex modulus and phase angle function of temperature

	Temperature [°C]											
	-10		0		10		20		30		40	
Material	|E*| [Mpa]	Phi [deg]	|E*| [Mpa]	Phi [deg]	|E*| [Mpa]	Phi [deg]	|E*| [Mpa]	Phi [deg]	|E*| [Mpa]	Phi [deg]	|E*| [Mpa]	Phi [deg]
1	28771.7	2.2	25149.2	4.1	19661.2	7.9	13095.1	15.0	7190.0	26.6	2678.4	44.2
2	33073.4	2.6	28215.0	4.8	21452.7	8.8	14202.7	15.3	7895.3	25.2	3221.2	39.1

2.2 Fatigue tests

The scope of the study is to focus on the effect of test temperature (ie material temperature) on fatigue performance assessment. At each temperature, specimens are tested at different loading levels ε (ie the maximal value of the strain amplitude in the tested specimen). According to the beam theory this value is proportional to the displacement amplitude through a geometrical factor. For each tested specimen, the fatigue life is taken as the number of loading cycles needed to decrease the specimen's stiffness by 50%. The strain amplitude range has been fixed to reach fatigue life between 10^4 and 10^6 loading cycles (EN 12697-24).

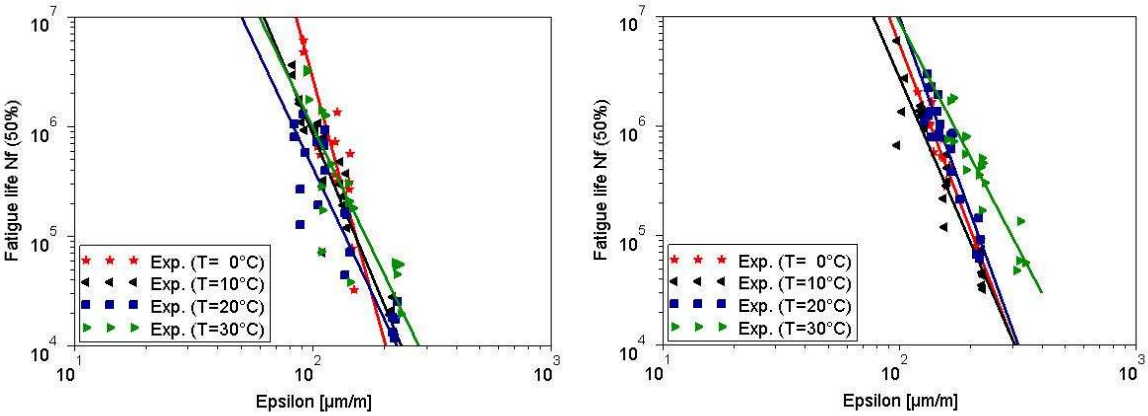
In a log-log diagram data of fatigue life vs strain amplitude are distributed along a linear like distribution, which allows to fit the fatigue line curve according (Eq. 1). Then the performance of the material is expressed by the parameter ε_6, which represents the loading level leading to a fatigue life of 10^6 cycles (statistically). The slope p also informs us on the sensitivity of the asphaltic mix to the increase of strain amplitude. The following section presents and analyzes the fatigue performance parameters obtained from fatigue tests, performed at four different temperatures, for the two previous mixes.

3 FATIGUE TESTS RESULTS

Fatigue tests at different temperature involve a large number of specimens. To be sure to be statistically representative of the material behavior, specimens sawn from different slabs are mixed up and divided into 4 sets, one for each temperature. For each temperature an average number of 20 specimens are tested at different loading amplitude. In total, for each material at least 80 specimen are submitted to fatigue tests.

3.1Fatigue lines

Figure 3 (a) and (b) present the fatigue lines obtained respectively for materials 1 and 2. Due to the scatter of the data the influence of temperature is not so clear on these plots. However, for material 1 the slope is qualitatively constant from 10 to 30°C but higher at 0°C.



(a) (b)

Fig 3: Fatigue lines at temperatures 0, 10, 20 and 30°C, (a) material 1 (b) material 2.

For material 2 the slope is qualitatively constant from 0 to 20°C and smaller at 30°C. These results are confirmed by Table 4. In this table, SN parameter is related to the standard deviation of $\log(N_F)$. It allows appreciating the scatter of fatigue failure. The standard deviation SN is less important for material 2. It could be due to the higher binder content and lower void content which offers less void cavities which could initiate the fatigue crack leading to failure.

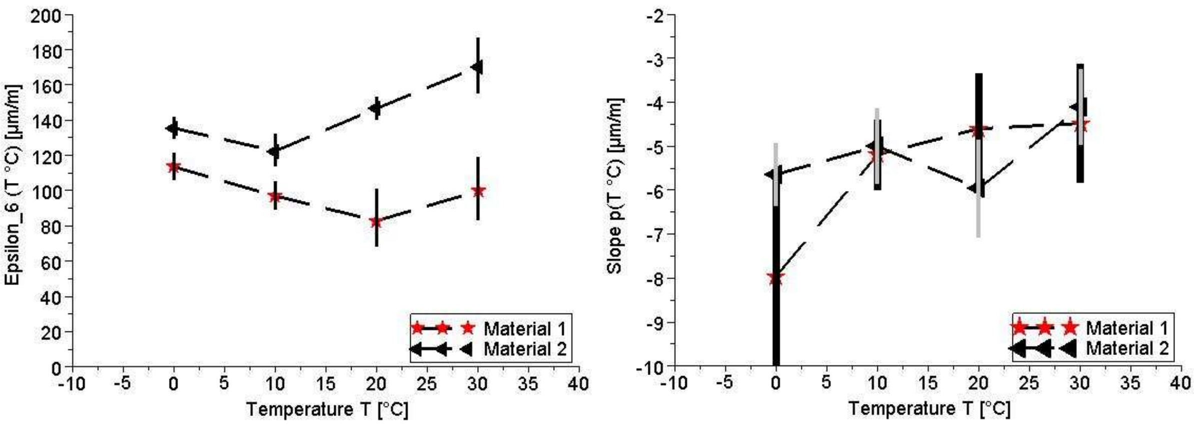
Table 4: Parameters of fatigue line of the two considered materials at the four temperatures

Material 1							
θ (°C)	Nb data	Slope p	$\varepsilon_6(\theta)$	$\Delta\varepsilon_6$ *	ε_6^{min}	ε_6^{max}	SN
0	16	-7.96 ± 2.2	113.5	5.8	105.8	121.8	0.35
10	26	-5.19 ± 0.8	97.0	6.1	89.4	105.2	0.28
20	24	-4.60 ± 1.3	82.9	12.5	68.0	100.9	0.44
30	22	-4.48 ± 1.4	99.7	13.6	83.4	119.4	0.43
Material 2							
θ (°C)	Nb data	Slope p	$\varepsilon_6(\theta)$	$\Delta\varepsilon_6$ *	ε_6^{min}	ε_6^{max}	SN
0	17	-5.64 ± 0.7	135.5	4.8	129	142	0.13
10	22	-5.00 ± 0.9	122.4	7.0	114	132	0.24
20	21	-5.96 ± 1.1	146.7	5.1	140	154	0.19
30	18	-4.11 ± 0.9	170.2	11.7	155	187	0.18

* EN 12697 - 24

3.2 Influence of temperature on fatigue lines parameters

On Figure 4, fatigue lines parameters ε_6 and the slope p, as well as their confidence interval are presented. For the two materials, ε_6 follows at typical curve with a minimum for intermediate temperature. Considering the calculated confidence interval we can conclude that temperature has a significant influence on fatigue behavior. In the explored range of temperature ε_6 exhibits a minimal value at intermediate temperature 20°C for material 1 and 10°C for material 2. This shape confirms the result obtained in the literature (Figure 1) which indicates a minimal value of ε_6 with temperature.



$\varepsilon_6(f = 25Hz, \theta)$ Slope p $(f = 25Hz, \theta)$

Fig 4: Fatigue line parameters function of temperature

For the slope parameter p the trend is less clear compared to the confidence interval. For material 1, the absolute value of the slope decrease with temperature. Its value is around -8±2 at 0°C and approximately constant around -5 from 10 to 30°C. For the material 2 the slope varies less and is around -5±1. The increasing trend between 0°C to 30°C is disturbed by the value obtained at 20°C which has a higher absolute value.

To analyze the shape of ε_6 vs temperature curve, we can consider qualitatively another well known effects occurring during cyclic tests on asphaltic materials. Their thermal dependency and dissipative behavior due to their intrinsic viscoelasticity may affect fatigue tests interpretation.

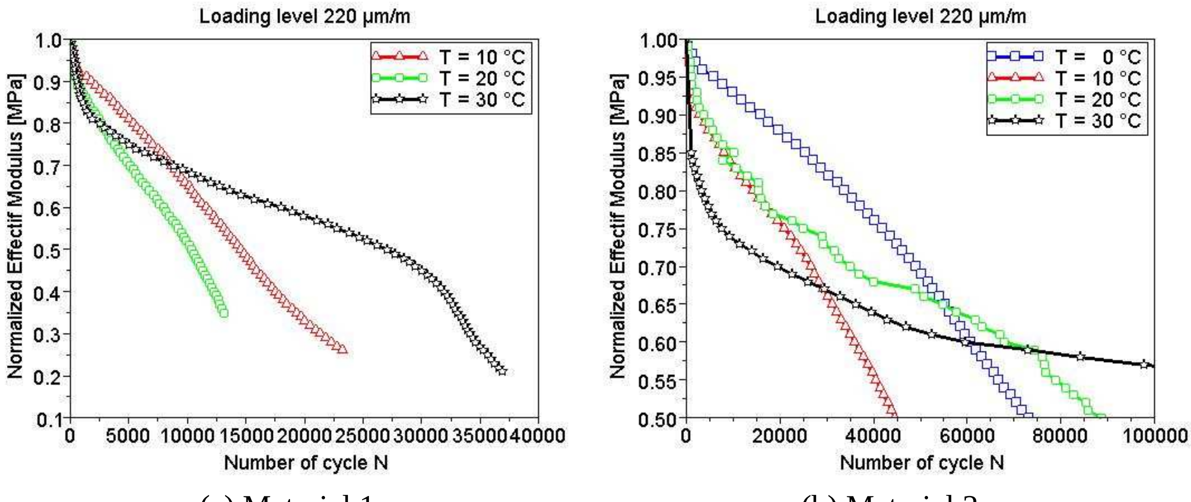
During fatigue test self-heating of the specimen occurs due to viscoelastic dissipation. For displacement controlled tests, the dissipation term is proportional to the square of the loading amplitude and to the imaginary part E_2 of the complex modulus E^* of the material (Bodin et al. 2004a). The dissipated energy contributes to material self-heating during cyclic tests which is coupled the mechanical response due to thermal softening of the material.

For asphalt mixes the imaginary part exhibits a maximum value, as shown Figure 2 for the tested materials. That means that the thermal softening which results of the viscoelastic dissipation is more important at this temperature. It could be the reason of the trend observed of ε_6 with temperature. For the two tested materials, imaginary part of the complex modulus reachs its maximum value around 20°C. This temperature is quite well correlated with the minimum of ε_6 for material 2. However, for material 1, the minimum of ε_6 is obtained at a lower temperature of 10°C. It indicates that the interpretation is not direct and simple and needs the help of modeling approaches to well capture the real phenomena. It has to be confirmed by further work and modeling results coupling damage analysis (Bodin et al. 2004b) and material self-heating (Bodin et al. 2004a) on trapezoidal specimen.

To highlight, the thermal effect and may be other nonlinear phenomena occurring during these cyclic tests, the following section focuses on the effect of test temperature on the shape of the stiffness decrease curve function of the number of loading cycles.

4 TEMPERATURE IMPACT ON STIFFNESS DECREASE

We have seen that the global performance parameter ε_6 is affected significantly by temperature. For a better understanding of what could play a role in that dependency. The analysis is now focused on the stiffness decrease rate of specimens tested at the same loading level but at different temperature. For each material Figure 5 presents the average (six specimens) stiffness decrease curve.
On each curve we can assume the classical shape of stiffness decrease in three stages: the second one is quasi-linear (Phase II), preceded by a curved stiffness drop (Phase I) and followed by the final drop leading to failure (Phase III).



(a) Material 1 (b) Material 2

Fig 5: Effect of temperature on the normalized stiffness decrease (a) material 1 (b) material 2.

On these plots (Figure 5), the first stage of the process (Phase I) seems to be affected by temperature. At low temperature ie 0°C the stiffness decrease observed at the beginning of the tests is nearly linear. For higher temperature ie 30°C, phase I is more important. For intermediate temperature, this first drop increase with the increase of temperature. Comparing the two materials, this phenomena seems to be affected by the binder content. Higher the binder content (material 2 compared to material 1), more pronounced is the phase I.

This phenomenon could be partially explained by the thermal dissipation as mentioned in the previous section. However, such non-linear phenomena as “thixotropy” could also occurs during these tests (Soltani and Anderson, 2005) and which could affects differently the stiffness decrease depending on the test temperature.

5 CONCLUSIONS

The goal of this paper is to present and discuss the influence of temperature on fatigue performance on asphaltic materials from experimental results. Two mixes (AC 14 for base layer) which have different binder type and binder content have been tested at four temperatures.

Classical analysis of the fatigue life versus the loading levels allows obtaining the performance parameter ε_6 (loading level leading statistically to a failure at 1 million cycles). In the range of temperature explored (0°C to 30°C) ε_6 is not constant and follows a parabolic like trend with a minimum at intermediate temperature. The temperature where this minimum is reach could be different from one material to the other.
Experimental results also show that the slope of the fatigue line is also affected by temperature. The trend observed (excepted for one value) is a decrease of the absolute value of the slope with the increase of temperature. To be generalized, these results should be confirmed by other extensive fatigue tests results.

The analysis done on the shape of stiffness decrease of specimen’s stiffness vs. the number of cycles, shows that the drop of stiffness at the beginning of the tests (phase I) is dependent on the test temperature. This could be due to the specimen self-heating which is intrinsically linked to temperature. However, we do not exclude that “thixotropy” or reversible destructuration of the binder could play a role in these (Gauthier et al., 2010).

Prospects of this experimental work will focus on modeling properly the coupled damage and heating phenomena. Additional work qualifying the phenomena (reversible non-linearity) occurring during cyclic tests on asphaltic materials will also enhance the analysis. Finally it will help for the evaluation of the real dependency of damage parameters with temperature for pavement engineering purposes.

6 ACKNOLEDGEMENTS

This work is part of a LCPC research project focus on improvements of the French pavement design method. Fatigue tests have been performed jointly at Laboratoire Central des Ponts et chaussées (Nantes centre) and at Laboratoire Régional des Ponts et Chaussées in Angers.

REFERENCES

Artamendi I., Allen B. and Phillips P. 2009, *Influence of temperature and aging on laboratory fatigue performance of asphalt mixtures,* Proceedings of the Advanced Testing and Characterization of Bituminous Materials, Rhodes, Greece, p. 185-194

Bodin D., Belzung F., de La Roche C., Chabot A. 2004a, *Thermomechanical Modelling for Fatigue Damage of Bituminous Materials*. 5th International RILEM Conference Cracking in Pavements, May 5-8 , Limoges, France, p. 675-682.

Bodin D., Pijaudier-Cabot G., de La Roche C., Piau J.-M and Chabot A., 2004b *A Continuum Damage Approach to Asphalt Concrete Fatigue Modelling*, Journal of Engineering Mechanics, ASCE, vol. 130 (6), pp. 700-708.

Bodin D., de La Roche C. and Pijaudier-Cabot G. 2006, *Size effect regarding fatigue evaluation of asphalt mixtures : Laboratory cantilever bending tests,* International Journal of Road Materials and Pavement Design, vol. 7, pp. 181-200.

CEN, EN 12697- 26. "*Bituminous mixtures - Test methods for hot mix asphalt – Part 26: Stiffness*." European standard, December 2004.

CEN, EN 12697-24:2007. "*Bituminous mixtures – Tests methods for hot mix asphalt – Part 24: Resistance to fatigue*". European standard, July 2007.

Di Benedetto H., de La Roche C., Baaj H., Pronk A. C., and Lundstrom R. 2004, *Fatigue of bituminous mixes: different approaches and RILEM group contribution*. Materials and structures, 37:202 – 216.

Gauthier G., Bodin D., Chailleux E., Gallet T. (2010) *Non linearity in bituminous materials during cyclic tests*, Road Materials and Pavement Design, special issue EATA 2010. In press.

Lundström R., Isacsson U. and Ekblad J. 2003, Investigations of stiffness and fatigue properties of asphalt mixtures. Journal of Materials. Vol. 38(24) p. 4941-4949

Molenaar A.A.A. 2007, *Prediction of asphalt fatigue cracking in asphalt pavements: Do we follow the right approach*. Transport Research record. n°2001 p. 155-162

Răcănel C. Romanescu, M. Dicu, A. Burlacu & C. Surlea. 2009. *Fatigue lines for asphalt mixtures used in wearing course*. Proceedings of the Advanced Testing and Characterization of Bituminous Materials, Rhodes, Greece. p. 795-805

Rao Tangella S.C.S., Craus J., Deacon J.A. and Monismith C.L. 1990, *Summary report on fatigue response of asphalt mixture TM-UCB-A-003A-89-3*. Strategic Highway Research Program – Project A-003. University of California - Berkeley, California. February.

Sab K. and Zenzri H. 1992, *Modélisation et simulation de l'endommagement par fatigue des chaussées bitumineuses*. Annales des Ponts et Chaussées, 63, p. 3–28. In French

SETRA and LCPC, 1997. *French design manual for pavement structures.* Technical Guide. ISBN 2-7208-7070-6.

Soltani A. and Anderson D. A. 2005, *New Test Protocol to Measure Fatigue Damage in Asphalt Mixtures*. Road Materials and Pavement Design. Vol. 6 No. 4, p. 485-514.

Underwood B.S. , Hou E.T. and Kim Y.R., 2009, *Application of simplified VECD modeling to the fatigue life prediction of asphalt concrete mixtures*. Proceedings of the Advanced Testing and Characterization of Bituminous Materials, Rhodes, Greece. p. 859-868