

LABORATORY FATIGUE MODELS FOR RECYCLED MIXES WITH POZZOLANIC CEMENT AND BITUMEN EMULSION

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Abstract. In recent years, due to technical and economical advantages, the production of pozzolanic cements have considerably extended. In the case of asphalt recycling, using Pozzolanic cements has several advantages. Lower initial stiffness and less shrinkage microcracks than Ordinary Portland Cement (OPC) are some benefits of pozzolanic cements which may reduce the probability of occurring premature cracking in recycled layer. In this research in order to investigate the effects of I (PM) pozzolanic cement on the fatigue cracking of recycled mixes with bitumen emulsion and develop fatigue models for these mixes, extensive indirect tensile fatigue and resilient modulus tests were performed at different temperatures and curing times. Test results showed that at high strain levels I (PM) cement specimens have superior fatigue behavior than OPC specimens. Furthermore, the effects of I (PM) cement on fatigue life of recycled mixes related to the initial strain level. Therefore a boundary strain level was determined. Above the boundary strain level, adding cement caused a reduction in fatigue life, whereas below that level the reverse was true. Finally based on laboratory testing results distinct models were established for different boundary strain levels.

Keywords: cold recycling, bitumen emulsion, pozzolanic cement, fatigue model.

1. Introduction

Use of pozzolanic compounds as a partial replacement with cement can have several technical and economical advantages. These compounds can modify hydrated cement microstructure due to the formation of additional calcium silicate hydrate (C–S–H) phases through pozzolanic reaction between the pozzolan and calcium hydroxide formed during the hydration of cement (Stefanović *et al.* 2004). Based on extensive research works adding pozzolan to cement as a partial replacement, improves the mix durability by reducing the void content, increases the initial mix workability, increases the ultimate strength and reduces the heat of hydration and volumetric changes (Pekmezci and Akyüz 2004; Stefanović *et al.* 2004; Aydin and Gül 2006; Bai and Gailius 2009). These properties encourage the use of pozzolanic cements in cold recycling of asphalt with bitumen emulsion. Besides several aforementioned properties, the effect of pozzolanic cements on the fatigue properties of recycled mixes is not clearly understood.

Fatigue cracking is one of the major distresses that influences the service life and quality of pavement. Since fatigue cracking could be initiated at each point of bound layers where the tensile stress exceeds the tensile strength, the fatigue behavior of each bounded material should be well recognized (Tayebali *et al.* 1992; Ziari and Khabiri 2007).

Cold recycled mixes which usually use as applied thick bound bases act more like slabs (De Beer 1990).

Applying a vertical load on the surface of a slab usually generates horizontal compressive stresses in the upper half of the slab and horizontal tensile stresses in the lower half. The strain resulting from these stresses (particularly the tensile strain at the bottom of the layer) ultimately leads to fatigue failure after too many load repetitions.

The results of field studies on cold recycled mixes with bitumen emulsion (CRME) without cement, have shown that the fatigue life of these layers are equal or even more than conventional hot mixes (Miro *et al.* 2004; ARRA 2001). Furthermore based on laboratory research works, for an equal air void, cold mixes have lower stiffness and higher fatigue life than conventional hot mixes (Miro *et al.* 2004; Iowa Highway Research Board and Iowa Department of Transportation 2007; Karlsson and Isacsson 2006; Shu *et al.* 2008). Fatigue behavior of these mixes have been evaluated as open graded hot mixes in which fatigue life is usually greater than the case of dense graded mixes (Asphalt Institute 1986; Suleiman 2002; SHRP–A–404 1994). Proper viscosity of bitumen emulsion makes it possible to mix cold mixes at ambient temperatures. This will reduce the initial aging of bitumen which may affect the long term flexibility of the mix. Moreover the beneficial effects of some emulsifiers and additives, used in bitumen emulsion (e.g. polymeric additives) were considered to be the reasons for the higher fatigue life compared with hot mixes (Asphalt Institute 1986; Suleiman 2002). Because of the

crystalline nature of pozzolanic bonds, lightly cemented materials tend to be brittle, while strongly cemented materials may be prone to shrinkage, cracking and pumping (Suleiman 2002; Rita *et al.* 2001). In general, however, the addition of cement increases the resilient modulus, shear strength and permanent deformation resistance and decreases the moisture sensitivity but may reduce the ability of the material to sustain repetitive loading (Suleiman 2002; Salomon and Newcomb 2000).

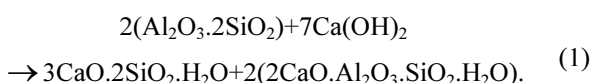
As previously mentioned, there is not enough information about the effect of cementitious additives on the fatigue characteristics of cold mixes. The results of a study on fatigue behavior of cement stabilized cold mixes showed that the relationship between stiffness and fatigue life is related to strain level (Brown and Needham 2000). As reported, at high strain levels, the fatigue life of mixes will be reduced by adding cement (Brown and Needham 2000). This was thought to be due to brittleness of these mixes and occurrence of micro shrinkage cracks. These will cause specimens to fail very quickly once crack initiation occurs (Brown and Needham 2000; Miller *et al.* 2006).

Due to lack of information regarding fatigue response of cold recycled mixes with bitumen emulsion and pozzolanic cement (CRMEP), in this research, the response of CRMEP was analyzed under different conditions (testing temperature and curing time). The main objectives of this research were:

1. Investigating the fatigue behavior of mixes containing different amounts of pozzolanic cement.
2. Comparison between the effect of pozzolanic cement and Ordinary Portland Cement (OPC).
3. Investigating the effects of curing time, cement content and temperature on resilient modulus and determining the relationship between resilient modulus and fatigue life.
4. Determining the boundary strain levels based on testing temperature.
5. Determining laboratory fatigue models for CRMEP.

2. Effects of pozzolanic compounds

As previously mentioned, there are some differences between the effects of pozzolanic cements and OPC. Pozzolanic cements are manufactured by incorporating different amounts (usually between 15% to 40%) of natural or artificial pozzolans to OPC clinker. Therefore, the properties of mixes containing pozzolanic cements are completely related to type and amounts of pozzolan. It is important to point out that the pozzolanic reactions, in respect to the OPC hydration reaction, are much slower reactions and occur in a noticeable extent after about two weeks (Stefanović *et al.* 2004). The mechanism of pozzolanic reaction in pozzolanic cements is not well recognized but the simple shaped of this reaction has been presented in Eq. (1):



According to Eq. (1), in order to form a solid product, pozzolanic material require the presence of calcium hydroxide which is a by-product of hydration reaction (Chengzhi *et al.* 1996; Leung and Balendran 2003). Based on several research results, calcium hydroxide concentration is the main reason of different properties of pozzolanic and OPC cements (Cojbašić *et al.* 2005). As shown in Fig. 1, in the case of OPC, the concentration of calcium hydroxide which has negligible contribution in strength is steadily increased by increasing the curing time. For pozzolanic cement, at the beginning of curing period, $\text{Ca}(\text{OH})_2$ has low concentration. Therefore, pozzolan particles have no considerable activity and exhibit the behavior of an inert material (Cojbašić *et al.* 2005). Due to the increase of (OH) concentration, which is a consequence of the creation of $\text{Ca}(\text{OH})_2$, the environment becomes more and more alkali. During this period (i.e. after about 7 to 14 days), the pozzolanic reaction starts to develop and as shown in Fig. 1, during the progression of this reaction the calcium hydroxide concentration will continuously reduce (Stefanović *et al.* 2004; Cojbašić *et al.* 2005). This reaction leads to production of the secondary cementitious compositions (e.g. C–S–H). Formation of C–S–H leads to decrease in air void and the final structure of pozzolanic cement mix will be more integrated than OPC. Production of the secondary cementitious compositions along with the reduction in air void have been known as the main reasons of considerable final strength increase for pozzolanic cements (Stefanović *et al.* 2004; Cojbašić *et al.* 2005).

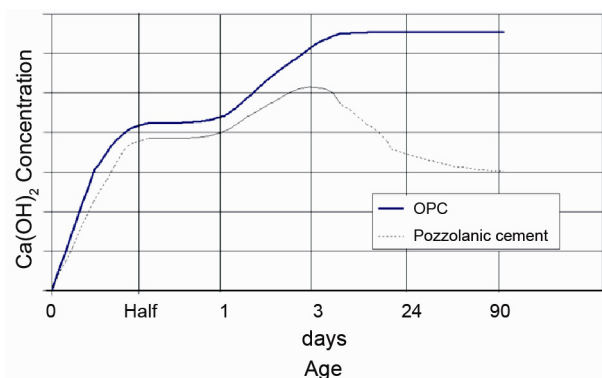


Fig. 1. Schematic diagrams of $\text{Ca}(\text{OH})_2$ Concentration during the Curing Period (Cojbašić *et al.* 2005)

3. Experimental design

3.1. Materials

3.1.1. Aggregates

Recycled Asphalt Pavement (RAP) materials were taken from Tehran West Depot. After crushing the lumps, the materials greater than 25 mm were removed and the gradation of the remaining materials was determined. Fig. 2 shows the gradation.

According to ASTM D2172 procedure the bitumen content of the RAP was determined. RAP materials contained 3.3% bitumen by total weight of the material.

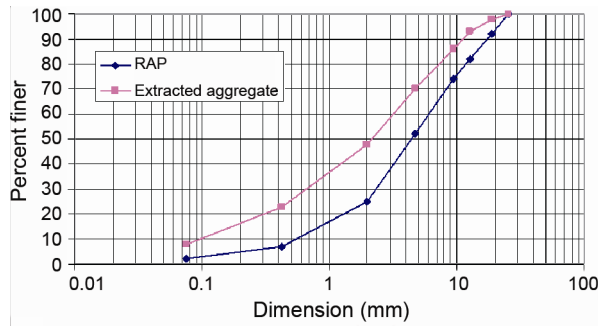


Fig. 2. The gradation of RAP materials and extracted aggregates

3.1.2. Bitumen emulsion

A cationic slow setting bitumen emulsion (CSS-1h) was used in the experimental program. The binder and water level of the bitumen emulsion was 60% and 35% of total weight of bitumen emulsion, respectively. Also the bitumen emulsion consisted of about 3.5% solvent and 0.5% HCL.

3.1.3. Cement

I(PM) pozzolanic cement was used in this research. This type of pozzolanic cement is a combination of 85% OPC clinker and 15% natural pozzolan (tuff) which consisted of the components reported in Table 1. In this table, the composition of OPC and the physical properties of both cements are also presented. The amounts of cement added to the mixes were 1, 2 and 3% by weight of the mix.

Table 1. Component materials of the I(PM) cement and OPC

A – Component materials			
Component	Average (by weight)%		
	I(PM)	Type I	
C ₃ S	40.0	49.0	
C ₂ S	23.0	25.0	
C ₃ A	9.0	12.0	
C ₄ AF	7.0	8.0	
CaSo ₄	2.5	2.9	
CaO	0.7	0.8	
MgO	0.8	1.2	
Loss of ignition	3.0	1.5	
Natural pozzolan (tuff)	15.0	0.0	
B-Physical properties			
Property	I(PM)	Type I	
Min. specific surface (cm ² /gr)	3000	2800	
Setting time	Initial (min)	60	
	Final (hr)	7	
Min. compressive strength (kg/cm ²)	7 days	175	
	28 days	315	

3.2. Mix design

Several procedures could be used to determine the optimum bitumen emulsion and water content in CRME design (AASHTO AGC-ARTBA Joint Committee 1998; Mućinis et al. 2009). In this research modified Marshall method was used. Following this method, first, mixtures

without cement were prepared containing 3% water (consisting of emulsion water, RAP water content (0.3%) and additional water added to the mixture).

Bitumen emulsion was added to the mix at different amounts (varying from 2.5% to 4.5%) at 0.5% increments. The mixes were then compacted by Marshall hammer. Then, samples were cured in oven for 6 hours at 60 °C. While in the molds, cured samples were then kept for 48 hours at room temperature. The specimens were then tested for maximum and bulk specific gravities and Marshall stability. The design criterion for determining the optimum bitumen emulsion specifies the void content within 9–14% (AASHTO AGC-ARTBA Joint Committee 1998). Using this criterion and determining the maximum specific gravity and stability, optimum bitumen emulsion content was assigned to be 4%. In the second step, the optimum moisture content (OMC) was determined for specimens containing different amounts of I (PM) cement at optimum bitumen emulsion content. The air voids of specimens at all cement contents were more than 14%. Therefore, with determining the maximum specific gravity, maximum stability and minimum air void, OMC values were determined. The details of testing results are presented in Table 2. As seen in this table, OMC values were increased as a result of increased I (PM) cement content.

3.3. Laboratory testing

The main laboratory program consisted of resilient modulus (M_r) and fatigue tests. Both of these tests were performed using cylindrical specimens applying Indirect Tensile method. The tests were performed at – 10, 5 and 25 °C by a UTM-14 testing apparatus. Specimens were tested after 7, 28 and 120 days curing at room temperature of about 25 °C. Before testing, specimens were placed in temperature control chamber of UTM at least for 5 hours until the target temperatures were attained.

The chamber was equipped with a reference specimen containing two thermometers to measure and record the core and skin temperatures. For each I (PM) cement contents (0 to 3% by weight of mixture) 53 specimens

Table 2. Optimum moisture content (OMC) determination

Cement (%)	Water (%)	Bulk SG	Max. SG	Stability (kg)	Air void (%)	OMC (%)
1.0	2.5	2.081	2.430	721	14.36	4.4
	3.5	2.088	2.427	779	13.97	
	4.5	2.091	2.419	822	13.56	
	5.5	2.078	2.426	845	14.34	
	6.5	2.075	2.424	725	14.40	
2.0	2.5	2.080	2.432	814	14.47	4.9
	3.5	2.083	2.437	831	14.53	
	4.5	2.090	2.431	865	14.03	
	5.5	2.090	2.429	877	13.96	
3.0	6.5	2.081	2.424	812	14.15	5.3
	3.0	2.062	2.423	893	14.90	
	4.0	2.071	2.429	912	14.74	
	5.0	2.077	2.431	973	14.56	
	6.0	2.076	2.427	991	14.46	
7.0	2.067	2.424	854	14.73		

were prepared. These specimens were divided into three groups. The first and second groups consisted of 18 cylindrical specimens for indirect tensile strength (ITS) and M_r tests. At each condition, ITS was repeated three times and the average of ITS values were reported.

M_r test was carried out based on ASTM D4123–04 (2004) standard testing method. Specimens were preconditioned by initial load cycles. The minimum preconditioning load cycles should be determined so that the resilient deformations become almost stable. Based on laboratory observations, 100 load repetitions were selected.

This test was performed at above-mentioned conditions at two different stress levels (15% and 30% of ITS) to determine M_r values at each condition (testing temperature and curing times).

Fatigue tests were conducted on the third group which for each condition consisted of 27 specimens according to EN–12697–24 standard testing method. Three different stress levels were assumed and similar aforementioned conditions of temperature and curing periods were applied.

The loading frequency was set equal to 0.66 Hz, consisting 0.25s loading and 1.25s recovery time. Loading continued until complete splitting of the specimen.

In ITFT the maximum tensile strain at the center of specimen is calculated using Eq. (2):

$$\epsilon_0 = \left(\frac{2\Delta H}{d} \right) \times \left(\frac{1+3\nu}{4+\pi\nu-\pi} \right). \quad (2)$$

If the Poisson’s ratio (ν) is assumed to be 0.35, Eq. (2) will be summarized as in Eq. (3):

$$\epsilon_0 = 2.1 \frac{\Delta H}{d}, \quad (3)$$

where d is diameter of specimen (mm); ϵ_0 is tensile strain ($\mu\epsilon$) at the center of specimen; ΔH is horizontal deformation, measured using two LVDTs (mm).

4. Results and discussion

4.1. Resilient Modulus (M_r)

The effects of temperature and curing time on M_r of CRMEP are shown in Figs 3A and 3B, respectively. With reference to Fig. 3A, the first and second numbers in the legend represent curing times and I (PM) cement contents, respectively. Similarly, in Fig. 3B the first and second values represent temperatures and cement contents. The results indicate that M_r values of CRMEP were increased steadily by decreased temperature and increased cement content and curing period. The relationships between testing temperature and curing time were correlated with exponential and logarithmic functions, respectively. These resulted in high R^2 values (at most cases greater than 0.9) which indicates that unlike the case of CRME the effects of temperature on M_r value is more sensible than curing time in CRMEP. Based on several research works, cement considerably accelerates the setting process of emulsified systems (Oruc *et al.* 2006).

Scanning analysis with electron microscope showed that the cementitious phase in cold mixes is dispersed within the bituminous binder which could have considerable effect on stiffening the organic binder at the initial days of curing (Brown and Needham 2000). Therefore, this phenomenon will reduce the slope of M_r increments during the curing period. Due to high temperature sensibility of bitumen emulsion mixes adding cement at high levels (greater than 3%) usually results in a significant decrease in temperature susceptibility of cold mixes (Oruc *et al.* 2006). With reference to Fig. 3A, the slopes of M_r diagrams have not appreciably changed upon adding I (PM) cement. This minor effect of cement on temperature sensibility of studied mixes might be as a result of low bitumen emulsion content compared with conventional cold mixes that usually contain more than 7% bitumen emulsion (Batista 2004; Brown and Needham 2000).

Fig. 3C shows the ratio of resilient modulus gained for I (PM) cement specimens to OPC. Based on the results, for 7-day specimens, at most cases the resilient modulus ratio is between 0.75 to 0.95. This ratio increased with increased curing time and after 120 days curing for all specimens the resilient modulus ratio is more than 1.0. According to Table 1, I (PM) cement contains about 15% natural pozzolan. As previously mentioned, at the beginning of curing period the pozzolan particles have no activity and act like filler (Stefanović *et al.* 2004; Čojbašić *et al.* 2005). By increasing the concentration of

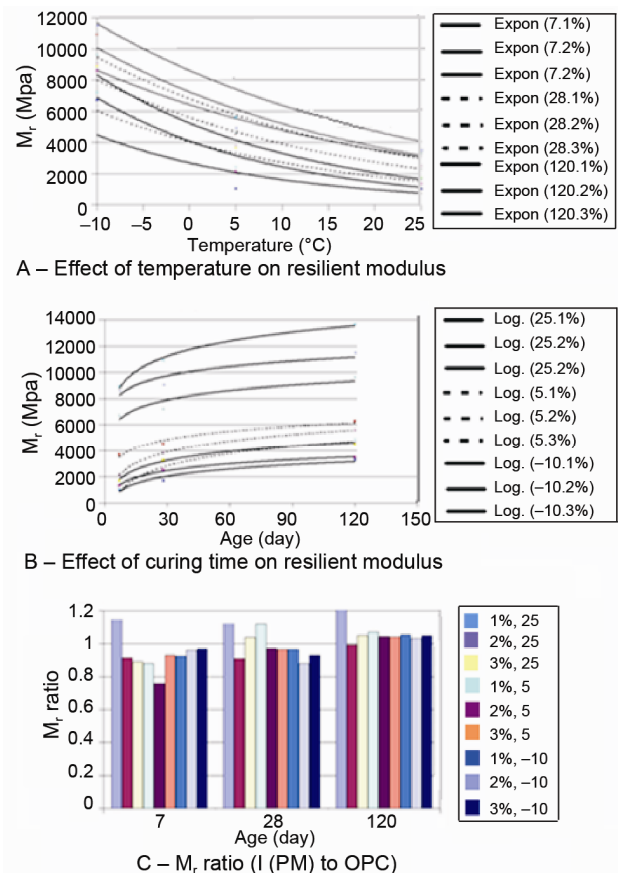


Fig. 3. CRMEP resilient modulus tests results

calcium hydroxide, the pozzolan reaction starts to develop and the secondary cementitious materials will be produced. These compositions which have higher volume than initial structures lead to reduced void content (Stefanović et al. 2004). This phenomenon named pore refinement effect. Production of secondary cementitious compositions and pore refinement after long curing time in pozzolanic cement structures are known as the main reasons for attaining higher stiffness than OPC structures (Chengzhi et al. 1996).

4.2. Fatigue

On performing fatigue tests, horizontal deformations were recorded and the deformation diagrams were plotted. A sample fatigue data is shown in Fig. 4. As it can be seen, the diagram of horizontal deformation can be divided into three periods. At the initial load cycles, due to occurrence of plastic deformations, the rate of deformation increment is relatively high. At the second period, the rate of deformation increment gets stabilized and the fatigue curve shows a straight trend. During this period, microcracks which formed during the first and second periods will progress (Thiago et al. 2008). The combination and progress of these cracks lead to complete fracture of specimen.

There are two definitions for fracture life in ITFT method. As it can be seen from Fig. 4A, in the first definition, failure is considered to occur when the constant rate of deformation increase, is replaced by a faster rate. After that point, the microcracks present in the specimen are combined into macrocracks and the specimen is broken into two pieces (Thiago et al. 2008).

According to EN-12697-24 standard, the fracture life shall be determined as the total number of load applications that causes a complete fracture of the specimen. This definition is shown in Fig. 4B. In this research due to less scattered data, the fracture life was considered as in the second definition.

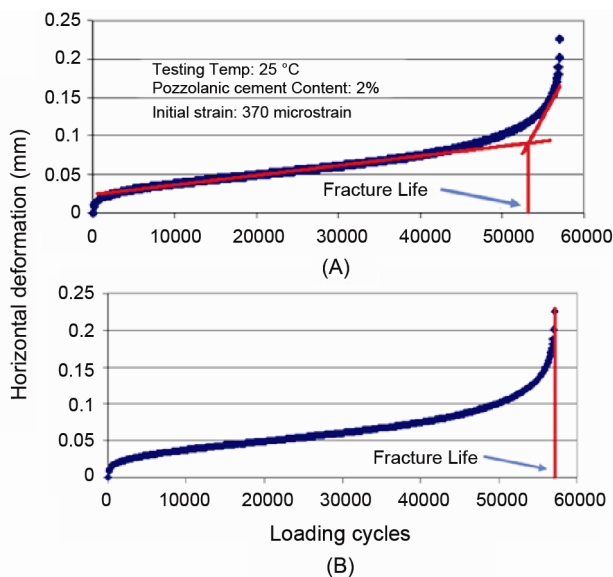


Fig. 4. Fracture life definitions for ITFT method

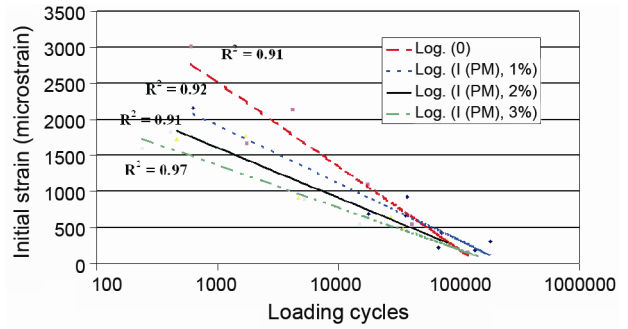


Fig. 5. Fatigue lines of CRMEP specimen at 25 °C

The results of fatigue tests, conducted at 25 °C are presented in Fig. 5. As it can be seen, the slopes of the fatigue lines have decreased by addition of I (PM) cement content. The reduction in slope of the fatigue line with increased cement content is the sign of changing the characteristics from a typical asphalt mixture to a cement treated material.

According to Fig. 5, except for specimens containing 1% I (PM) cement, approximately at above 200 microstrain level, the addition of cement caused a reduction in fatigue life, whereas at below 200 microstrain, the results were reversed.

In order to compare the effects of I (PM) cement and OPC, fatigue lines obtained for specimens tested at 5 °C and 25 °C are shown in Figs 6A and 6B, respectively. As seen in figure 6A, at above 380 microstrain, the fatigue life obtained from specimens with no cement is more than two other groups. Below 380 microstrain, the

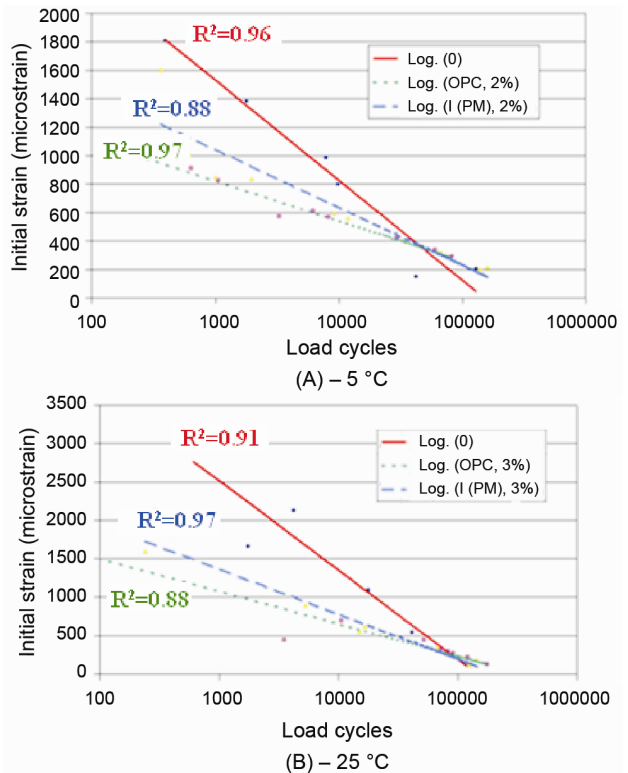


Fig. 6. Comparison between fatigue lines obtained for I (PM) cement and OPC

results were reversed and the OPC specimens have the most fatigue life. For both strain levels the I (PM) cement specimens have the mean results. According to Fig. 6B, similar results were achieved for specimens tested at 25 °C containing 0 and 3% of cements. At this condition the boundary strain level is about 240 microstrain.

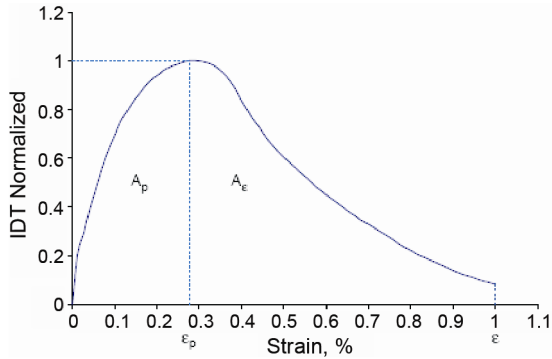


Fig. 7. Normalized IDT Curve for IT calculation (Huang *et al.* 2005)

The results indicated that in comparison with OPC and CRME (i.e. specimens with no cement), the I (PM) cement specimens have the mean fatigue behavior. At all strain levels the fatigue behavior of CRMEP was more similar to CRME. In order to better investigate the obtained results, the toughness index (TI) parameter was determined for compared specimens. This parameter is calculated from the stress-strain curve obtained from ITS test and is a good indication for determining the brittleness of a material. Fig. 7, presents a typical normalized indirect tensile stress-strain curve. The TI parameter is defined as Eq. (4):

$$TI_{it} = \frac{A_{\epsilon} - A_p}{\epsilon - \epsilon_p}, \quad (4)$$

where TI_{it} is toughness index; A_{ϵ} is area under the normalized stress-strain curve up to strain ϵ ; A_p is area under the normalized stress-strain curve up to strain ϵ_p ; ϵ is strain at the point of interest; ϵ_p is strain corresponding to the peak stress.

For an ideal brittle material with no post peak load carrying capacity, TI equals to zero (Huang *et al.* 2005). In Fig. 8A, the stress-strain curves obtained from ITS test for 28 days specimens at 25 °C is shown. According to this Figure, the CRME specimen has the highest fracture strain. Furthermore, the fracture strain of I (PM) cement specimen is slightly higher than that of OPC. Fig. 8B shows the TI parameter for specimens containing different amounts of cements tested at similar condition (i.e. after 28 days at 25 °C). Based on this figure, the maximum TI parameter was attained for CRME. By increasing the cement content TI parameter was steadily decreased. This indicates the more brittle behavior of specimens containing both cements than CRME specimens. As shown for all amounts of cements the CRMEP specimens have less TI than OPC. Results obtained at

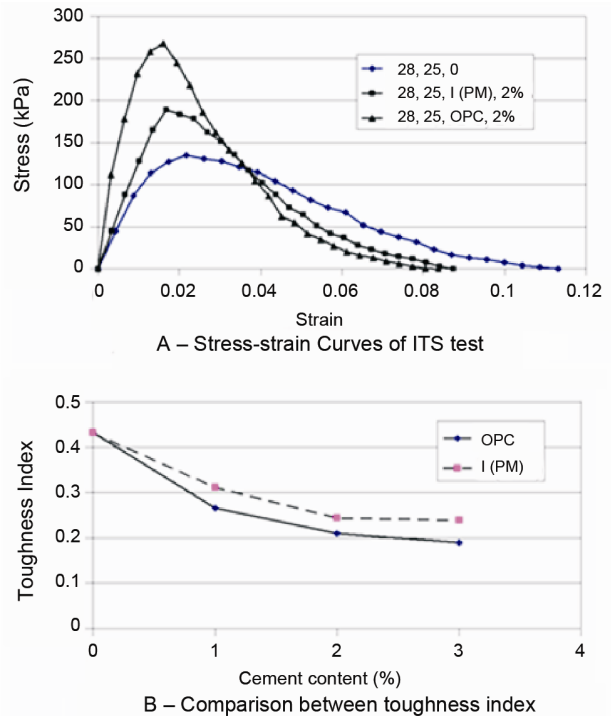


Fig. 8. Effect of cement type on toughness index

other testing conditions (i.e. other temperatures and curing times) were somehow similar to above mentioned condition. Therefore, it could be concluded that there is a meaningful relationship between the toughness index and fatigue behavior of studied specimens. Specimen with higher toughness index (e.g. CRMEP in comparison with OPC) is more flexible and has higher fatigue life at high strain levels.

The results indicated that at heavy loading conditions (high initial strains), I (PM) cement has preferable fatigue characteristics than OPC.

Fig. 9 shows the CRMEP fatigue lines at 5 and -10 °C. Based on the results the slopes decreased steadily by decreasing the test temperature. Laboratory observations confirmed that at low temperatures minor differences in specimens or testing conditions (e.g. strain level) leads to very different numbers of cycles to crack initiation. Therefore considerably more specimens were needed to achieve narrow distribution of the results.

The main factors that influenced the resilient modulus of CRMEP in this research were I (PM) cement content, curing time and testing temperature. Presented results show that, by adding cement, while the other parameters are fixed, the resilient modulus of the specimen will be increased and the fatigue line slope will be decreased. The same results were achieved upon the testing temperature was reduced. In contrast, as shown in Fig. 10, comparison of the fatigue life at a constant strain level showed no considerable changes in fatigue line slope with the addition of curing time. This is to some extent comparable to the results obtained from some other similar researches (Batista 2004). Therefore, the results achieved at different curing periods (7, 28 and 120 days) were grouped into one fatigue line in Figs 5, 6 and 9.

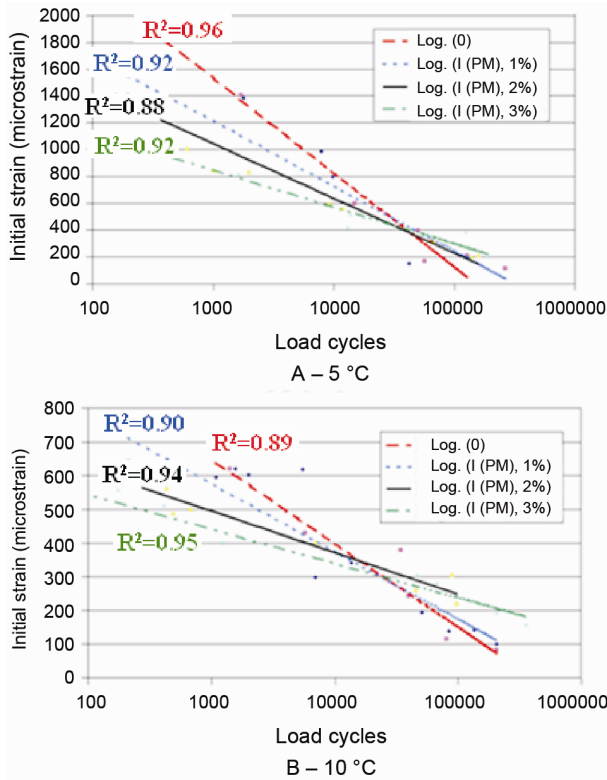


Fig. 9. Fatigue lines of CRMEP tested at 5 and $-10\text{ }^{\circ}\text{C}$

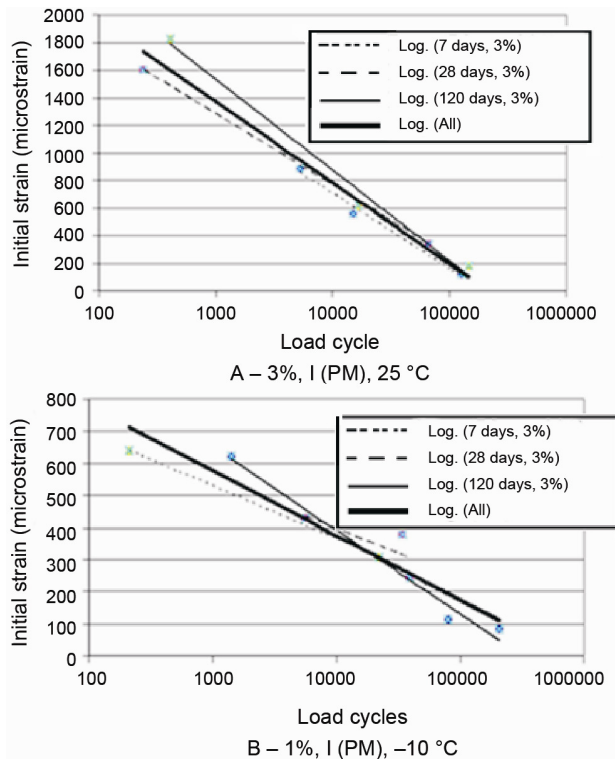


Fig. 10. Effect of curing time on fatigue life of specimens tested at 25 and $-10\text{ }^{\circ}\text{C}$

Similar to the results obtained at 25 $^{\circ}\text{C}$, the fatigue life of CRMEP specimens at 5 and $-10\text{ }^{\circ}\text{C}$ were also related to strain level. According to Fig. 9A, at 5 $^{\circ}\text{C}$, for an initial strain greater than about 400 microstrain, adding I (PM) cement content will result in a reduced fatigue

life. For strain levels below 400 microstrain, the reverse was resulted. With reference to Fig. 9B, at $-10\text{ }^{\circ}\text{C}$ similar results were achieved at about 300 microstrain.

5. Fatigue model

In order to present the fatigue models for CRMEP, first the general relationships between strain – fatigue life and M_r – fatigue life were determined. Then based on the obtained relationships, final fatigue models were developed so that the fatigue life of CRMEP could be estimated using the M_r and strain level values.

5.1. General strain-fatigue life relationship

According to Figs 5 and 9, the general relationship between initial strain and fatigue life will be as presented in Eq. (5):

$$N_f = ae^{b\varepsilon}, \quad (5)$$

where a and b are constant values; ε is initial strain (microstrain).

For example, as shown in Fig. 5, the above relationship for specimens containing 2% I (PM) cement at 25 $^{\circ}\text{C}$ could be presented by Eq. (6):

$$N_f = 208981e^{-0.0033\varepsilon}, R^2 = 0.91. \quad (6)$$

5.2. General M_r – fatigue life relationship

In order to determine the general relationship between M_r and fatigue life, at constant initial strains, the curves of fatigue life variation with M_r have been drawn. In Fig. 11, the curves obtained for 7 days cured specimens, tested at 5 $^{\circ}\text{C}$, are shown. Based on these results, at different strain levels, various relationships were obtained. According to Fig. 11A, for initial strains below 400 microstrain, there is a polynomial trend between fatigue life and M_r , whereas at above 400 microstrain the fatigue life is steadily reduced by M_r addition. Similarly for specimens tested at 25 and $-10\text{ }^{\circ}\text{C}$ the same trends were achieved at about 200 and 350 microstrain, respectively.

Considering the distribution of results at different conditions (testing temperature and curing times), the initial strain of 250 microstrain was chosen as boundary strain level. Based on Eqs (7) and (8), two different relationships were determined at each strain level.

$$N_f = aM_r^b + c, \varepsilon < 250\text{ }\mu\text{m/m}, \quad (7)$$

$$N_f = ae^{bM_r} + c, \varepsilon > 250\text{ }\mu\text{m/m}, \quad (8)$$

where a , b and c are constant values; M_r is resilient modulus (Mpa).

For example the above relationship for 7 days cured specimens tested at $-10\text{ }^{\circ}\text{C}$ and at initial strain of 600 microstrain, has been presented in Eq. (9):

$$N_f = 25911e^{-0.0003M_{r7}}, R^2 = 0.87, \quad (9)$$

where M_{r7} is resilient modulus value after 7 days curing (Mpa).

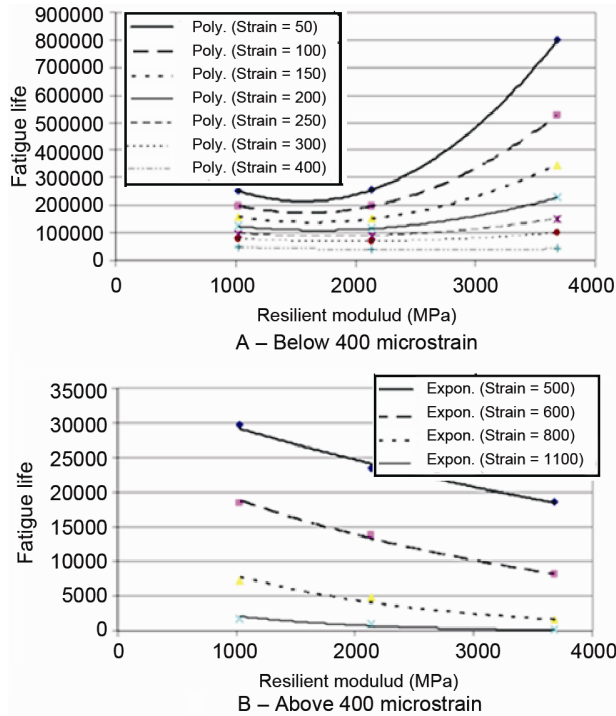


Fig. 11. Relationship between fatigue life and M_r (7 days curing, 5 °C)

5.3. Final fatigue models

In order to develop final fatigue models, the results of fatigue and M_r tests were analyzed by SPSS statistics software. Based on previously discussed analysis, several nonlinear regressions were evaluated. The controlling stages of each nonlinear model consisted of the followings:

1. As a prerequisite, the coefficient of determination (R^2) of the nonlinear model should be appropriate.
2. The linear relationship between the estimated values with nonlinear model and tested values should be fair enough. Other control criteria for this linear relationship were:
 - 2.1. The slope and constant values of the linear relationship should be close to 1 and 0, respectively.
 - 2.2. The results obtained from the nonlinear model should be equally distributed. The model which overestimated or underestimated the results, was declined notwithstanding the proper R^2 value.

The final fatigue model obtained at strain levels above 250 microstrain is presented in Eq. (10):

$$N_f = ae^{b\varepsilon + cM_r}, \varepsilon > 250 \mu\text{m/m} . \quad (10)$$

Similarly the final fatigue model for strain levels below 250 microstrain could be presented as in Eq. (11):

$$N_f = ae^{b\varepsilon} \times M_r^c, \varepsilon < 250 \mu\text{m/m} . \quad (11)$$

The final calibrated models, for initial strain levels of above 250 microstrain are presented in Eqs (12) to (14):

$$N_f = 421367e^{-0.0044\varepsilon - 0.000242M_{r7}}, \varepsilon > 250 \mu\text{m/m} , \quad (12)$$

$$N_f = 558034e^{-0.00416\varepsilon - 0.000242M_{r28}}, \varepsilon > 250 \mu\text{m/m} , \quad (13)$$

$$N_f = 648163e^{-0.00389\varepsilon - 0.0002M_{r120}}, \varepsilon > 250 \mu\text{m/m} , \quad (14)$$

where ε is initial strain (microstrain); M_{r7} , M_{r28} and M_{r120} are resilient modulus after 7, 28 and 120 days curing (MPa).

Fig. 12 illustrates the goodness of fit between predicted fatigue life by Eq. (12) and the measured values. As seen the coefficients of determination (R^2) for model is above 0.9 and the predicted values from models are equally distributed.

Similarly, the calibrated models for strain levels below 250 microstrain are presented by Eqs (15) to (17):

$$N_f = 0.212e^{-0.0208\varepsilon} \times M_{r7}^{1.99}, \varepsilon < 250 \mu\text{m/m} , \quad (15)$$

$$N_f = 0.138e^{-0.0207\varepsilon} \times M_{r28}^2, \varepsilon < 250 \mu\text{m/m} , \quad (16)$$

$$N_f = 0.889e^{-0.022\varepsilon} \times M_{r120}^{1.76}, \varepsilon < 250 \mu\text{m/m} . \quad (17)$$

Comparison between predicted fatigue life by Eqs (15) and measured values is shown in Fig. 13. Based on this figure the coefficient of determination indicates good prediction capability of model and the predicted values are equally distributed.

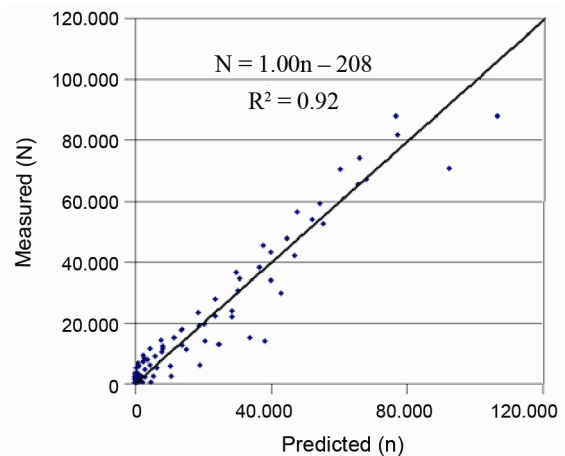


Fig. 12. Predicted versus measured fatigue life for Eq. (12)

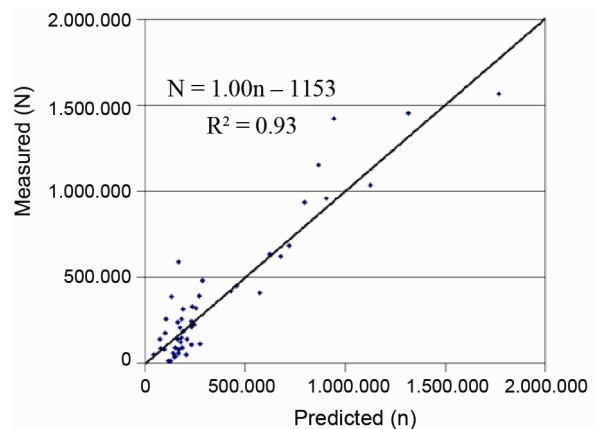


Fig. 13. Predicted versus measured fatigue life for Eq. (15)

In Table 3, the relationships between Measured (N) and predicted (n) values for all models are presented. As reported, all developed models resulted in high R^2 values. Furthermore, the slopes of the N-n relationships are close to 1 and the constant values are acceptable. High constant values for Eqs (16) and (17) are acceptable, because these models are presented for low strain levels which the fatigue life as high as 2×10^6 loading cycles were achieved for these levels.

Table 3. Measured (N) – predicted (n) relationships for Eqs (12) to (16)

Equation number	N-n relationship	R^2
(12)	$N = 1.00n - 208$	0.92
(13)	$N = 1.02n - 1099$	0.90
(14)	$N = 0.97n - 1669$	0.90
(15)	$N = 1.00n - 1153$	0.93
(16)	$N = 0.88n - 98306$	0.97
(17)	$N = 1.05n - 209877$	0.87

With reference to the presented models at low strain levels (Eqs (15) to (17)), the fatigue life of CMREP steadily increased by increasing M_r value. At high strain levels (Eqs (12) to (14)) the reverse was true and addition of M_r led to reduced fatigue life. Therefore, it could be concluded that for pavement with heavy loading conditions, the addition of I (PM) cement content has a detrimental effect on fatigue characteristics of CRMEP.

Various research results have indicated that for CRMEP layers which are mostly used as base layer in a pavement system, strain levels rarely exceed 200 microstrain (Brown and Needham 2000). Therefore it can be concluded that adding cement (up to 3%) to mixes clearly extends their fatigue life.

6. Conclusions

The main objective of this research was to develop fatigue models for cold recycled mixes with bitumen emulsion and pozzolanic cement (CRMEP). Laboratory experiments, consisted of resilient modulus and indirect tensile fatigue tests which have been carried out at three different temperatures (25, 5 and -10°C) and curing times (7, 28 and 120 days). Based on the results the following conclusions can be drawn:

1. Adding I (PM) cement content led to a reduction in fatigue line slopes. The least slopes were obtained for specimens tested at -10°C .
2. There was a meaningful relationship between toughness index and fatigue behavior of studied mixes. Specimen with higher toughness index is more flexible and has higher fatigue life at high strain levels.
3. Initial strain level was identified to be as an important parameter affecting CRMEP fatigue life. At 25°C and at above 200 microstrain, the addition of resilient modulus caused a reduction in fatigue life whereas for below 200 microstrain the results were reversed. The boundary strain levels for 5 and -10°C testing temperature were about 400 and 300 microstrain, respectively.

4. Unlike cement content and testing temperature, at a specified strain level, there was no particular relationship between curing time and fatigue curve slope. Therefore the fatigue models were presented separately for each of the curing times.

5. The results indicated that at heavy loading conditions (high initial strains), the fatigue behavior of I (PM) cement is better than OPC.

6. Based on fatigue life-initial strain and fatigue life-resilient modulus relationships the initial strain of 250 microstrain was selected as the final boundary strain level for all testing temperatures.

7. For strain levels of below and above 250 microstrain, distinct fatigue models were presented. According to these models, increased I (PM) cement content resulted in increased fatigue life at low strain levels (below 250 microstrain). Reversely, at high strain levels the addition of I (PM) cement had detrimental effect on fatigue characteristics of CRMEP.

References

- AASHTO AGC-ARTBA Joint Committee. 1998. Task Force No. 38. *Report of Cold Recycling of Asphalt Pavements*. Interim Report, AASHTO, Washington, D.C., USA.
- ARRA. 2001. Basic Asphalt Recycling Manual. *Publication No. NHI01-022, Asphalt Recycling and Reclaiming Association (ARRA)*. Annapolis, Maryland, USA.
- Asphalt Institute. 1986. *Asphalt Cold-Mix Recycling Manual Series No. 21 (MS-21)*. 2nd Ed. The Asphalt Institute, Lexington, Kentucky, USA.
- ASTM D4123-04. 2004. *Indirect Tension Test for Resilient Modulus of Bituminous Mixtures*. ASTM Book of Standards, Vol. 04.03, USA.
- Aydin, A. C.; Gül, R. 2006. Influence of Volcanic Originated Natural Materials as Additives on the Setting Time and Some Mechanical Properties of Concrete, *Construction and Building Materials* 21(6): 1277-1281. doi:10.1016/j.conbuildmat.2006.02.011
- Bai, J.; Gailius, A. 2009. Consistency of fly ash and metakaolin concrete, *Journal of Civil Engineering and Management* 15(2): 131-135. doi:10.3846/1392-3730.2009.15.131-135
- Batista, F. A. 2004. *Asphalt Cold Mixtures for Pavement Rehabilitation Overlays and Recycling*. National Laboratory of Civil Engineering, Lisbon, Portugal.
- Brown, S. F.; Needham, D. 2000. A Study of Cement Modified Bitumen Emulsion Mixtures, *Asphalt Paving Technology* 69: 92-121.
- Chengzhi, Z.; Ai Qin, W.; Mingshu, T.; Xiaoyu, L. 1996. The filling role of pozzolanic material, *Cement and Concrete Research* 26(6): 943-947. doi:10.1016/0008-8846(96)00064-6
- Ćojbašić, L.; Stefanović, G.; Sekulić, Ž.; Heckmann, S. 2005. Influence of the Fly Ash Chemical Composition on the Portland Cement and Fly Ash Mixture Hydration Mechanism, *Facta Universitatis – series Mechanical Engineering* 3(1): 117-125.
- De Beer, M. 1990. *Aspects of the Design and Behavior of Road Structures Incorporating Lightly Cementitious Layers*: PhD Thesis, University of Pretoria, Pretoria, South Africa.
- EN 12697-24. 2004. *Bituminous Mixtures – Test Methods for Hot Mix Asphalt-Part 24: Resistance to Fatigue*. European Committee for Standardization, July 2004.
- Huang, B.; Shu, X.; Li, G. 2005. Laboratory Investigation of Portland Cement Concrete Containing Recycled Asphalt

- Pavements, *Cement and Concrete Research* 35(10): 2008–2013. doi:10.1016/j.cemconres.2005.05.002
- Iowa Highway Research Board and Iowa Department of Transportation. 2007. *Evaluation of Long-Term Field Performance of Cold In-place Recycled Roads: Field and Laboratory Testing*. Final Report, Iowa, USA.
- Karlsson, R.; Isacson, U. 2006. Material-Related Aspects of Asphalt Recycling – State-of-the-Art, *Journal of Materials in Civil Engineering* ASCE 18(1): 81–92. doi:10.1061/(ASCE)0899-1561(2006)18:1(81)
- Leung, H. Y.; Balendran, R. V. 2003. Properties of fresh polypyrrolene fibre reinforced concrete under the influence of pozzolans, *Journal of Civil Engineering and Management* 9(4): 271–279.
- Miller, J.; Guthrie, S.; Crane, A. 2006. *Evaluation of Cement-Stabilized Full-Depth-Recycled Base Material for Frost and Early Traffic Conditions*. Federal Highway Administration, Recycled Materials Resource Center at the University of New Hampshire, Durham, New Hampshire, USA.
- Miro, R.; Martinez, A.; Cancela, A. 2004. *Case Study of In-Place Recycled Asphalt Pavement of the N-536 Highway in Spain*. TRB 83th Annual Meeting, Washington DC, USA.
- Mučinis, D.; Sivilevičius, H.; Oginskas, R. 2009. Factors determining the inhomogeneity of reclaimed asphalt pavement and estimation of its components content variation parameters, *The Baltic Journal of Road and Bridge Engineering* 4(2): 69–79. doi:10.3846/1822-427X.2009.4.69-79
- Oruc, S.; Celik, F.; Akpinar, M. V. 2006. Effect of Cement on Emulsified Asphalt Mixtures, *Journal of Materials Engineering and Performance* 16(5): 578–583. doi:10.1007/s11665-007-9095-2
- Pekmezci, B. Y.; Akyüz, S. 2004. Optimum Usage of A Natural Pozzolan for The Maximum Compressive Strength of Concrete, *Cement and Concrete Research* 34(12): 2175–2179. doi:10.1016/j.cemconres.2004.02.008
- Rita, I.; Zaman, M.; Miller, A. 2001. *Characteristics of Cold Processed Asphalt Millings and Cement –Emulsion Composite*. TRB 80th Annual Meeting, Washington, DC, USA, January 7–11.
- Salomon, A.; Newcomb, E. 2000. *Cold In-place Recycling Literature Review and Preliminary Mixture Design Procedure*. Minnesota Department of Transportation, Office of Research Services, Minnesota, USA.
- SHRP–A–404. 1994. *Fatigue Response of Asphalt-aggregate Mixes*. Strategic Highway Research Program, National Research Council, Washington, D.C., USA.
- Shu, X.; Huang, B.; Vukosavljevic, D. 2008. Laboratory evaluation of fatigue characteristics of recycled asphalt mixture, *Construction and Building Materials* 22(7): 1323–1330. doi:10.1016/j.conbuildmat.2007.04.019
- Stefanović, G.; Sekulić, Ž.; Čojbašić, Lj. 2004. The dependence of physical-mechanical characteristics of cement on fly ash origin, *Facta Universitatis – series Mechanical Engineering* 2(1): 141–153.
- Suleiman, N. 2002. *A State-of-the-Art Review of Cold in-Place Recycling of Asphalt Pavements in the Northern Plains Region*. North Dakota Department of Transportation, Bismark, North Dakota, USA.
- Tayebali, A. A.; Rowe, G. M.; Sousa, J. B. 1992. Fatigue Response of Asphalt Aggregate Mixtures, *Journal of the Association of Asphalt Paving Technologists* 61: 333–360.
- Thiago, F.; Kim, Y. R.; Lee, J. 2008. *Research on Fatigue of Asphalt Mixtures and Pavements in Nebraska*. Report NoP579, University of Nebraska, Nebraska Lincoln, USA, February 25.
- Ziari, H.; Khabiri, M. M. 2007. Interface condition influence on prediction of flexible pavement life, *Journal of Civil Engineering and Management* 13(1): 71–76.

PUCOLANINIO CEMENTO IR BITUMINIŲ EMULSIJŲ MIŠINIŲ PERDIRBIMO LABORATORINIAI NUOVARGIO MODELIAI

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Santrauka

Pastaraisiais metais dėl techninių ir ekonominių veiksnių pucolaninio cemento gamyba gerokai padidėjo. Pucolaninio cemento naudojimas perdirbant asfaltą turi keletą pranašumų. Mažesnis pradinis standumas ir mažiau mikrotrūkių nei įprastame portlandcemyje (*Ordinary Portland Cement, OPC*) – tai keletas pucolaninio cemento pranašumų, galinčių sumažinti priešlaikinio plyšių atsiradimo tikimybę perdirbamame sluoksnyje. Šiuo moksliniu tyrimu siekiama iširti *I (PM)* pucolaninio cemento poveikį nuovargio plyšių atsiradimui perdirbamuose bituminių emulsijų mišiniuose ir sukurti šių mišinių nuovargio modelius. Buvo atlikti išplėstiniai netiesioginio tempimo nuovargio ir elastingumo modulių bandymai, esant skirtingoms temperatūroms ir džiūvimo laikui. Bandymų rezultatai parodė, kad esant aukštam įtempimo lygiui *I (PM)* cemento bandiniai yra atsparesni nuovargiui nei portlandcemenčio (*Ordinary Portland Cement, OPC*) bandiniai. Be to, perdirbtų mišinių iš *I (PM)* cemento tvarumas yra artimas pirminiam įtempimo lygiui. Todėl buvo nustatytas ribinis įtempimo lygis. Viršijus ribinį įtempimo lygį ir įmaišius cemento tvarumas sumažėja, o esant žemesniam įtempimo lygiui buvo gautas priešingas rezultatas. Galiausiai remiantis tyrimais buvo nustatyti atskiri modeliai skirtingiems ribiniams įtempimo lygiams.

Reikšminiai žodžiai: šaltasis perdirbimas, bituminė emulsija, pucolaninis cementas, nuovargio modelis.

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