

Structural strength assessment of the reconstructed road structure in terms of the loading time and yield criterion

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Abstract. This article reports the results of numerical simulations of the stress-strain states in the rebuilt road structure compared to the solutions contained in the Polish Catalogue, with the true characteristics of the layer materials taken into account. In the case analysed, a cold-recycled base layer with foamed bitumen as a recycling agent was used. The presented analysis is complementary to the mandatory in Poland procedure of mechanistic pavement design based on a linear elastic model. The temperature distribution in the road structure was analysed at the reference temperature of 40°C on the asphalt layer surface. The loading time was included in the computer simulations through the use of the classic generalized Maxwell model and thus the stiffness-time history of the layers had to be determined. For this purpose, the dynamic modulus E^* tests of the loading time frequency from 0.1 Hz to 20 Hz were carried out, and the yield point was modelled using the Coulomb-Mohr failure criterion calculated on the basis of triaxial compression tests. The analytical solution to the problem was found with ABAQUS. The results demonstrate that the high temperature of asphalt layers and long loading time noticeably reduces the stiffness modulus in those layers. That reduction changes the principal stress levels, which significantly influences the shear stress both in the recycled base layer and in the subgrade soil. Should the yield point be exceeded rapidly in the recycled layer, the horizontal stresses in the asphalt layers will increase and adversely affect the durability of the reconstructed road pavement structure, especially in the zones of slow heavy vehicle traffic.

1. Introduction

The individual design of the pavement layer system in Poland is based on the assumption that a single load from the equivalent standard axle (ESAL) causes very small deformations. Therefore, to analyse the stress-strain states in the flexible pavement the elastic model is used at a fixed loading time of 0.02s (approximately 60 km/h) and equivalent temperature of + 13°C [1]. This model allows correct design of the pavement structure for the reference conditions assuming proper drainage of the road frame, but it does not consider the loading time. The provisions of Polish legislation provide for the use of viscoelastic models that allow the implementation of loading time in the design [2]. There are numerous road sections with a speed limit where the loading time is much greater than the design value given in the Catalogue. In this case, the phenomenon of pavement layer stiffness relaxation should be introduced in the pavement rehabilitation or reconstruction design method.

Another extremely important issue, usually overlooked in structural analysis, is the assessment of the limit state of stresses in the subgrade and in recycled bases, especially in typical pavement layer systems operating under KR1-KR2 traffic where, unlike in much thicker road structures operating under the K3-KR7 traffic, exceeding the ultimate limit state at the same load level is highly probable



due to the thin package of structural layers. With these factors taken into account, the diagnosis of the KR1-2 surface will indicate exceeding the ultimate limit state due to among other factors extended vehicle loading time in synergy with high temperature as the probable cause of excessive deformations. In this situation, the elastic-plastic model will complement the stress-strain state considerations [3]. The design of flexible pavements working within elastic stress has to include a very important issue of meeting several conditions according to which the limit state cannot be exceeded in any of the layers, the structure should not deform when passing the permissible elastic stresses and tensile stresses should not be greater than the permissible stress [4, 5]. Nevertheless, structural damage may occur in the case of a thin compressible layer present in the subgrade or due to the moistening of the subgrade layers.

This article focuses on the stress-strain state in the KR1-2 traffic pavement manufactured with a foamed bitumen stabilized cold recycled base course and the subgrade with the specified parameters of the Coulomb-Mohr plastic model. It should be noted that due to the properties of the foamed bitumen, the mineral mix can contain a lower quality mineral material and even quite a large amount of mineral dust up to 15% relative to other types of recycled base courses [6]. Unfortunately, engineering practice hardly ever considers the subgrade ultimate limit state and relaxation in the pavement asphalt layers and recycled base courses, assuming the new pavement layer system is based on the optimization resulting from the solutions adopted in the Catalogue [1].

2. Materials and methods

2.1. Standard based parameters of pavement structural layers

The upper layers of the pavement structure were produced from AC11S asphalt concrete (wearing course) and AC16W (binder course). The gradation curves and the physical and mechanical parameters complied with the requirements set forth in PN-EN 13108 [7] and in WT2/2010 [8] applicable in Poland for traffic load category KR1-2. The mix standard parameters and frame compositions are summarized in Table 1.

Table 1. Frame composition and standard parameters of bituminous and recycled mixtures.

Property	Symbol	u.m.	AC11S	AC16W	RAP50/VA50
Bitumen content	Bitumen/ cement	[%]	5.7/0	4.6/0	2.0/2.0
Sensitivity to moisture damage to PN-EN 12697-12,	ITSR	[%]	96.2	91.0	-
Resistance to moisture damage [12]	TSR		-	-	81.9
Void space to PN-EN 12697-8	V _m	[%]	2.5	4.6	12.6
Voids filled with bitumen in the mineral mix, % (v/v),	VFB	[%]	83.9	69.0	-
Voids in mineral aggregate, % (v/v)	VMA	[%]	15.3	15.0	-
Aggregate grain size above 2 mm,	-	% (m/m)	60.0	68.0	60.4
Aggregate grain size of 0.063/2 mm,	-	% (m/m)	29.5	26.5	34.2
Aggregate grain size below 0.063 mm	-	% (m/m)	10.5	5.5	5.4

The recycled base layer used for the pavement reconstruction was designed as a cold deep-recycled mix with foamed bitumen (MCAS) [9]. The denotation 50RAP/50VA means 50% RAP content and 50% virgin aggregate content. The primary coarse aggregate (VA) had a continuous gradation curve with the maximum grain size "D" equal to 31.5 mm and met the Polish requirements of WT - 2010 [8] based on EN 13242 [10]. In order to ensure the required size of aggregate in the Marshall sample, the components of the recycled waste material were screened through a 22.4 mm sieve. The mineral mixture was designed following the Wirtgen guidelines [11]. The results of grouping with the given recycled mixture in relation to other waste materials are discussed in [13]. The final MCAS mixture was conditioned suggesting results in [14].

The first testing stage was devoted to all layers of the structure and required the determination of the generalized Maxwell (GM) model parameters in the linear viscoelasticity (LVE) range. The tension-compression test was carried out under controlled strain of a low amplitude of $25\mu\epsilon$ [12, 15] in accordance with PN-EN 12697-26 Annex D [16]. The test temperatures were: -7°C , 5°C , 13°C , 25°C and 40°C . The complex modulus E^* and the phase angle δ were determined.

The fracture condition for the MCAS layer containing 50RAP/50VA aggregate was determined according to the Coulomb-Mohr (CM) failure criterion using elements of ASTM D2664 [17] and information provided in [18]. The 80 mm (diameter) x 160 mm (height) specimens were core drilled from a set of six compacted MCAS samples 150 mm (diameter) x 200 mm (height). The angle of internal friction " ϕ " was 41.5° , while the cohesion quantity c was assumed to be 473.2 kPa at the recommended load speeds given by Jenkins [19].

2.2. Mechanical parameters of pavement layers structure

The first stage of testing consisted in determining the rheological parameters of the materials from which the structural layers were made. Since the analysis took into account the phenomenon of relaxation, a model accounting for the changes in stiffness over time had to be used. Hence the behaviour of pavement structural layers was described using the generalized Maxwell (Weichert) model in the linear viscoelasticity range (LVE) [20, 21]. The accuracy of asphalt concrete rheological parameters determined using the generalized Maxwell model greatly influences the accuracy of viscoelastic strains determined in asphalt concrete. The viscoelastic parameters were based on the results of dynamic tests with the oscillating load applied in the controlled strain conditions according to PN-EN 12697-26 Annex D [16]. This type of stiffness modulus test is much more convenient in the situation when the parameter searched for is the instantaneous stiffness modulus E_o .

Viscoelastic parameters of the GM model were obtained by determining the imaginary part (E'') and the real part (E') of the complex modulus $E^* = E' + iE''$ of the frequency function [21]. The form of the complex modulus components are represented using Fourier transforms in formulas (1)(2)

$$E' = E_o - \sum_{i=1}^N E_i + \sum_{i=1}^N \frac{E_i \tau_i^2 \omega^2}{1 + \tau_i^2 \omega^2} \quad (1)$$

$$E'' = \sum_{i=1}^N \frac{E_i \tau_i \omega}{1 + \tau_i^2 \omega^2} \quad (2)$$

where: E_i —the i -th modulus of elasticity in the GM module, E_o — the modulus of instantaneous elasticity, τ_i —the i -th relaxation time in the GM model, ω —reduced frequency.

The temperature shift factor was determined from equation (3):

$$a_T = \omega \cdot e^{(A_1 + T \cdot A_2)} \quad (3)$$

where: T — the test temperature, ω —the frequency at the test temperature, A_1 , A_2 —model parameters.

Goodness-of-fit measures such as coefficient of determination R^2 and normalized mean error MNE were used to determine the model fit to the experimental data [23]. The further wear diagnosis of the pavement structure under the KR1-KR2 traffic category accounted for the limit state with a certain assumption of the ground condition. The load carrying capacity parameters of the subgrade in relation to the Coulomb-Mohr model were derived from the literature data. Finally, the subgrade parameters were assumed to be G1 category, i.e., a layer of sand and gravel mix with the angle of friction " ϕ " of 40° and the cohesion value c of zero [24].

The viscoelastic character of the pavement layer system was described by means of a maximum of five Maxwell elements. Knowing that asphalt concrete in a given range of small strains is a thermorheologically simple material [25], the load time-temperature relationship was established using a horizontal temperature shift factor [23]. The final form of the master curve included the optimization of the real and imaginary parts of the stiffness modulus, linking in the analysis the relationship

between the complex stiffness modulus and the phase angle. Table 2 summarizes the parameters fitted in the GM model in the form of a master curve and the Coulomb-Mohr boundary condition.

Table 2. Master curve parameters of the pavement structural layers on the basis of the Generalized Maxwell (GM) Model.

Asphalt concrete AC11S - Wearing course						
	Parameters of the generalized Maxwell model		α_T (exponential function)		Criterion CM	
	g_i [-]	τ_i [s]	A_1	A_2	ϕ [°]	c [kPa]
the generalized Maxwell (GM) model	$g_1=0.29731$	$\tau_1=0.00027$	8.31	-0.274	-	-
	$g_2=0.29731$	$\tau_2=0.00457$				
	$g_3=0.24517$	$\tau_3=0.09663$				
	$g_4=0.16021$	$\tau_4=5.01952$				
	$E_o = 23930\text{MPa}$					
	$R^2 = 0.99$; RMSE = 12.45%					
Asphalt concrete AC16W –Binder course						
	Parametersof the generalized Maxwell model		α_T (exponential function)		Criterion CM	
	g_i [-]	τ_i [s]	A_1	A_2	ϕ [°]	c [kPa]
the generalized Maxwell (GM) model	$g_1=0.29808$	$\tau_1=0,00001$	12.45	-0.264	-	-
	$g_2=0.23997$	$\tau_2=0.00013$				
	$g_3=0.20785$	$\tau_3=0.00157$				
	$g_4=0.17046$	$\tau_4=0.02198$				
	$g_5=0.08365$	$\tau_5=1.42083$				
	$E_o = 20690.39\text{MPa}$					
$R^2 = 0.99$; RMSE = 7.1%						
Recycled base with foamed bitumen (50%RAP/50%VA)						
	Parameters of the generalized Maxwell model		α_T (exponential function)		Criterion CM	
	g_i [-]	τ_i [s]	A_1	A_2	ϕ [°]	c [kPa]
the generalized Maxwell (GM) model	$g_1=0.21416$	$\tau_1=0.00001$	11.32	-0.232	41.5	473.2
	$g_2=0.21416$	$\tau_2=0.00063$				
	$g_3=0.20886$	$\tau_3=0.02993$				
	$g_4=0.20886$	$\tau_4=5.72972$				
	$g_5=0.115396$	$\tau_5=5.72972$				
	$E_o = 10784.41 \text{ MPa}$					
$R^2 = 0.96$; RMSE =5.1%						

The results of the analysis compiled in Table 2 indicate that the model fit to the experimental data was done with a moderate mean square error of less than 13%. The correlation coefficient was greater than 0.96. The largest dispersion of results, based on the root-mean-square error RMSE, was observed for AC11S. Probably an increased amount of bitumen in the wearing course caused slight disturbances in master curve fitting at high temperature. Nevertheless, all the results of the qualitative evaluation of the master curve model fit were considered satisfactory. The set of master curves at 25°C is shown in Figure 1.

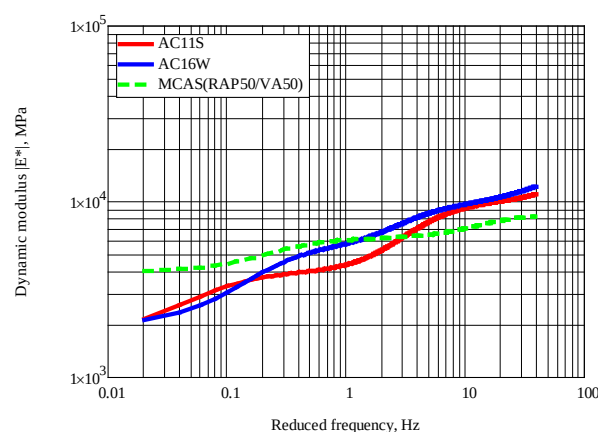


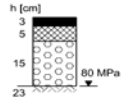
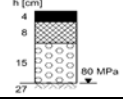
Figure 1. Dynamic modulus (master curves) at 40°C of the pavement structure.

Analysis of the fitting results for the Maxwell model parameters (Table 2) and of the master curve plots indicates that the AC11S layer containing the largest amount of bitumen attained the highest level of the instantaneous modulus. The MCAS recycled layer had the lowest relaxation rate, being the least sensitive to the vehicle loading time. The cement content in the MCAS mixture (2.0%) was responsible for this result. The parameters of the physical Maxwell model from Table 2 were directly implemented to the ABAQUS numerical model.

3. Numerical model

The numerical model assumed considering the stress-strain state for two types of structures under light traffic KR1 and KR2. It should be remembered that the standard load of a vehicle is the same as for structures operating under higher traffic categories. The numerical model included the pavement surface temperature of 40°C, whereas the temperature distribution was based on the findings and equations contained in [26]. The temperature distributions measured at the mid-thickness of analysed layers are shown in Table 3 [1, 26].

Table 3. Pavement layer system and temperature distribution in the pavement.

Traffic category	Type	thickness	depth	Mid-thickness	Temperature of the layer [23]	Graphical representation
-	-	[m]	[m]	[m]	[°C]	[-]
KR1	AC11S	0.03	0.03	0.015	37,5	
	AC16W	0.05	0.08	0.06	33	
	MCAS	0.15	0.23	0.16	30	
	subgrade	-	below 0.23	-	25	
KR2	AC11S	0.04	0.04	0.020	37	
	AC16W	0.08	0.12	0.08	30	
	MCAS	0.15	0.27	0.20	31	
	subgrade	-	below 0.27	-	25	

The ABAQUS modelling software was used in the numerical analysis. A finite element rotationally symmetric model was chosen for the surface of the structure. The subgrade was determined in such a way that the dimensions of the model did not affect the state of stresses and strains from external loads. The cylindrical area was adopted, with dimensions 7 m in diameter and 7 m in thickness. The model required that the boundary conditions be defined. The adopted variant made the displacement of points on the side walls in directions perpendicular to them and the displacement in the base of the lower cylindrical area impossible. The finite element mesh together with the finite element dimensions was based on the results of simulations [27, 28]. The height of the finite element in the layer the base layer was set at a maximum level of 5 cm, while the asphalt layers were divided according to the principle of 4 elements for the thickness of the layer. As a result, the maximum height of the asphalt layer element was less than 5 cm. The computational model consisted of two-dimensional elements with non-linear parabolic shape functions of the CAX8R type [29]. The analysis assumed full interlayer bonding and full isotropy of the pavement structure materials.

The cyclic load was applied on the pavement structure, with two variants: 1 Hz and 10 Hz, i.e. an approximate vehicle speed of 6 km/h and 60 km/h respectively. The cyclic load corresponded to the form of the haversine function determined in ABQUS by fitting it to the recorded modulated wave function (4):

$$a = A_0 + A \cdot \sin \omega_1 \cdot (t - t_0) \cdot \sin \omega_2 \cdot (t - t_0) \quad \text{for } t > t_0 \quad (4)$$

where: A_0 , A , ω_1 , ω_2 – parameters of the function fitted to haversine form

After fitting the parameters of the modulated function, the representation of the haversine cycle of the vertical stress on the contact surface between the wheel and the pavement is shown in Figure 2.

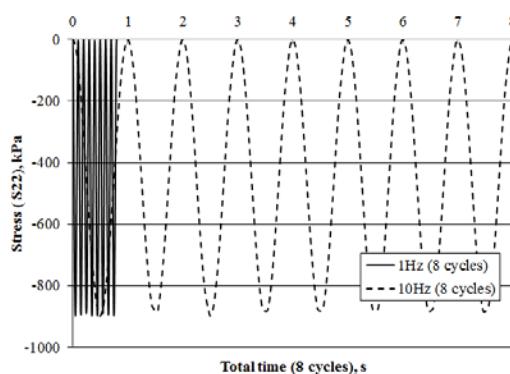


Figure 2. Load cycle haversine for 1Hz and 10 Hz..

Eight load cycles were used in the analysis to simulate the maximum number of standard vehicles that can pass within 24 hours on the KR1 traffic category road. For the KR2 traffic category, the load pattern also included 8 load cycles.

4. Analysis of road structure load bearing capacity

4.1. The influence of loading time on the stress and strain distribution

The analysis started with the simulation of the stress-strain state in the cross-section of the pavement structure, taking into account the influence of the loading time (1Hz, 10Hz) and pavement layer system (KR1, KR2). The simulation of the stress-strain state in the structure at the unusually low frequency of 1 Hz (8 cycles) was aimed at drawing attention to the fact that the level of damage to the pavement structure, for example in the intersection areas, is of great importance in terms of the durability of roads under light traffic. Three significant quantities were chosen from the large set of numerical model results. The first quantity was the total horizontal strain (E11). In the classic elastic model, the strain value at the underside of asphalt layers is used in estimating fatigue life. It should be noted that taking into account the parameters of the plastic model changes the stress field distribution in the structure. The accuracy of determining this parameter has a considerable impact on the quality of the pavement fatigue life estimation [27]. Figure 3 shows the change in the total horizontal strain (E11) and plastic vertical strain (PE22) in the cross-section of the pavement structure.

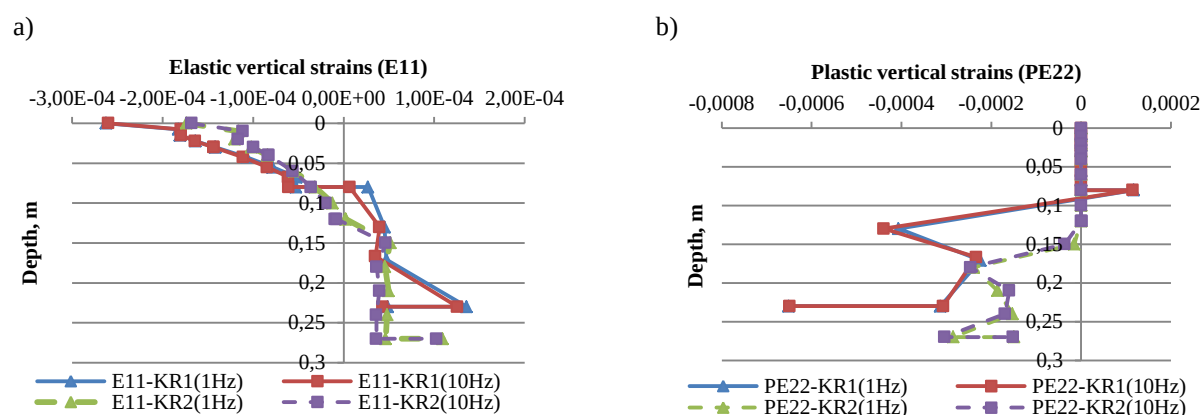


Figure 3. Strain distribution after 8 loading cycles: a) total horizontal strains (E11); b) plastic vertical strains (PE11)

The increased loading time (lower frequency) caused an increase in the maximum horizontal strain (Fig 3a) due to the stiffness modulus relaxation in the structural layers at 10 Hz being much smaller than that for 1 Hz. Therefore, in the structure under KR1 traffic ($h = 0.08\text{m}$), the loading time change from 10Hz to 1Hz causes a 14% increase in horizontal strain, while in the structure under KR2 traffic ($h = 0.12\text{m}$) the horizontal strain increase can be up to 76%. This suggests that ignoring the phenomenon of relaxation, especially in the area of intersections during the summer, is a gross oversimplification. The reduced asphalt layer stiffness due to relaxation also contributes to increased horizontal strains in the recycled layer. The increase in horizontal strains and the initiation of the cracking process in the base layer will make it act not as a cracked layer of large blocks but as a conventional unbound mixture. Then, the design assumption set forth in the Catalogue that the stiffness modulus $E = 1500\text{ MPa}$ further in the in-service period will become significantly overstated. The reduced stiffness of the asphalt layers will also lead to an increase in vertical strains in the base and subgrade. Vertical plastic strains determine the scale of potential structural deformations and ultimately create a need for the reconstruction of the entire pavement structure. Analysis of the results (Fig. 3b) shows that the decrease in the stiffness of asphalt layers (low frequency) caused an increase in horizontal plastic strains in the KR1 subgrade (about 3.5%), with only 1.5% in the KR2 subgrade. This increase is slight but considering the thickness of the structure, the level of vertical plastic strains in the KR2 subgrade and base layers is noticeably lower than in the KR1 structure. This indicates high sensitivity of the KR1 road pavement structure to overloading or to an excessive number of vehicles with a standard wheel load $> 50\text{ kN}$. Exceeding the permissible number of vehicles may lead to the rapid destruction of the road surface. The effect of stiffness change in the structure due to the loading time was also reflected by the deflection level (vertical deformation U22 in the wheel axis of the vehicle). The changes in vertical deflection are shown in Figure 4 in the cross-section of the road structure.

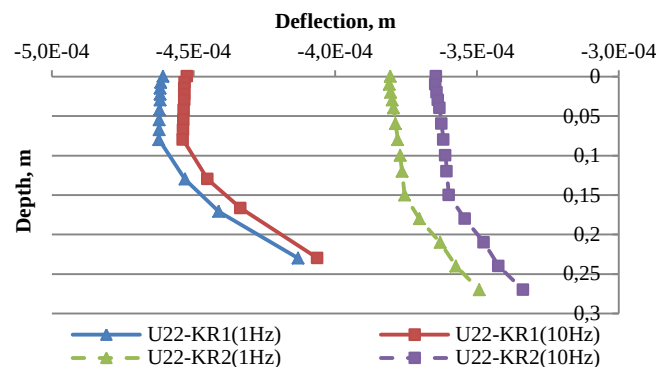


Figure 4. Deflection of the road structure after 8 loading cycles.

As expected, the decreased stiffness in asphalt layers due to relaxation increased the deflection at the contact area between the wheel and the pavement surface by about 2% in KR1 and 4.5% in KR2. However, the deflection difference between KR1 and KR2 for the frequency of 1Hz was 21%. The elastic-plastic model adopted for the analysis was a major factor in the change of the stress field distribution (plane state of strain). Hence the friction angle will be of crucial importance for the redistribution of vertical and horizontal stresses in the subgrade.

4.2. The influence of internal friction angle in the subgrade

Limit state parameters in geomaterials depend on a number of factors, especially on the compaction level and moisture content [5, 28]. The cohesion of the recycled base layer with foamed bitumen can be reduced, as in other bituminous mixtures, through the increase in bitumen temperature. According to Tan [30], the friction angle is independent of temperature and, often, a change in the moisture content (improper drainage of the road frame) reduces its value. Figure 5 shows the distribution

simulations for the vertical plastic strains (PE22) and horizontal strains (E11) against the internal friction angle (40° , 30° and 20°) in the structure designed for the KR1 traffic category.

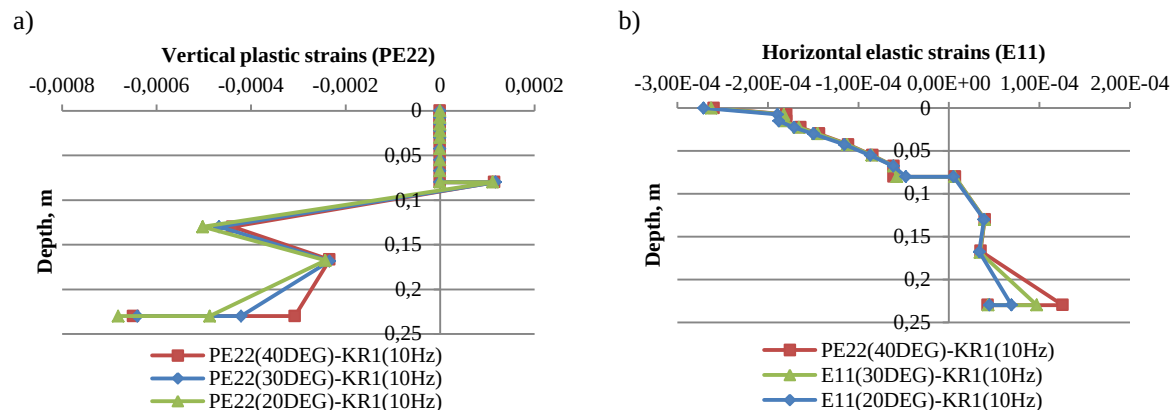


Figure 5. Strain distribution vs. friction angle: a) plastic vertical strains (PE11); b) total horizontal strains (E11).

Simulations were performed for friction angles of 40° , 30° and 20° at the load frequency of 10Hz after 8 loading cycles. The results in Fig. 5b show that the reduction in the friction angle in the subgrade has a destructive effect on the pavement strength due to the progressive accumulation of vertical plastic strains. Increased strain values were observed both at the top and bottom surface of the recycled base. The difference of the vertical permanent deformation (PE22) at the underside of the base layer increased by 37% in relation to the friction angle reduction in the subgrade from 40° to 20° . This is a high value considering the small number of cycles. This plastic deformation gain does not include the "shakedown" effect discussed in detail in [31].

As for the horizontal deformations (E11) in Figure 5a, the reduction in the friction angle in the subgrade also increased their values in the recycled base layer. After the 8th loading cycle, the recycled base was entirely within the large tensile strain region, which may lead to the premature loss of load carrying capacity in the structure as a whole. A large tensile strain could contribute to high cracking propagation in the base coarse. Taking into consideration many imperfections during a recycled base course manufacturing it is highly possible to observe many transverse cracks in pavement in a short time. In that respect, it should be pointed out that the effect of longer loading time other than that specified in the catalogue [1], considering high temperatures in the summer, may cause permanent deformations regarded as structural. Therefore, paradoxically, the layers designed for the KR1 and KR2 traffic need a well compacted subgrade with a high friction angle. Also, the load cycles increased above 115 kN/axle will cause permanent deformations to appear in the early service period of those pavement structures.

5. Conclusions and observations

As a result of the rheological tests of the materials and numerical analyses of pavement structures made with those materials, the following conclusions were formulated:

- pavements designed for light traffic (KR1, KR2) are sensitive to exceeding the ultimate limit state in the subgrade and in the recycled base layer in the summer;
- the time of loading is an important parameter to be considered at intersections during high temperature periods;
- the use of 8 loading cycles (one day for the KR1 category) in the summer causes structural deformations to appear in the structure;

- longer time of loading leads to increased horizontal strains, which in the cases analyzed was up to 76%, in the asphalt layers of the pavement, affecting its fatigue life;
- a reduction in the friction angle in the subgrade increases plastic deformations in the recycled base layer (MACAS);
- the major friction angle reduction caused the recycled base layer to work within the large tensile strain region after the 8th loading cycle, which suggests that fast exhaustion of the load carrying capacity of the structure as a whole should be expected;
- when constructing pavements designed for the KR1-2 traffic, attention should be given to the quality of the subgrade. Considering the small overall thickness of structural layers, insufficient quality of the subgrade may lead to premature failure of the pavement.

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