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MECHANISTIC-EMPIRICAL ASPHALT PAVEMENT DESIGN CONSIDERING THE EFFECT OF SEASONAL TEMPERATURE VARIATIONS

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The paper analyses the influence of seasonal temperature variations on fatigue strength of flexible and semi-rigid pavement structures chosen for KR4 traffic flow category. The durability of pavement determined assuming a yearly equivalent temperature of 10°C and assuming season-dependent equivalent temperatures was compared. Durability of pavement was determined with the use of Asphalt Institute Method and French Method. Finite Element Method was applied in order to obtain the strain and stress states by the means of ANSYS Mechanical software. Obtained results indicate a considerable drop in pavement durability if seasonal temperature variations are considered (up to 64% for flexible pavements and up to 80% for semi-rigid pavements). Durability obtained by the French Method presents lower dependence on the analysed aspect.

Keywords: asphalt pavement, durability, fatigue, FEM, mechanistic-empirical design, temperature variations.

1. INTRODUCTION

The basis for pavement design in Poland is the *Catalogue of typical semi-rigid and flexible pavement structures* in two versions: the *old catalogue*, issued by General Directorate for National Roads and Motorways (GDDKiA) in 1997 [19], and the *new catalogue*, an improved edition issued in 2014 [20]. Both provide lists of recommended pavement structures adequate for various traffic load categories. Both editions of the catalogue assume a constant equivalent temperature of 10°C

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for calculation purposes to simplify the design procedure. The aim of this paper is to examine how the resulting pavement durability obtained with the use of mechanistic-empirical approach is influenced by considering seasonal temperature variations in the design process [3, 4].

Mechanistic-empirical design method assumes partial application of mechanistic modelling and of empirical correlations in the design process [1, 5, 7, 9, 14]. The response of the pavement to the load – i.e. stresses, displacements, strains – is calculated with the use of proper software from an accurate Finite Element Method (FEM) model [2, 11, 12]. As the durability of the pavement is related to damage such as fatigue cracking or structural rutting, the relation between damage-inflicting stress and the damage itself is established empirically basing on past performance in the form of a transfer function.

When considering the effect of temperature, instead of assuming a constant equivalent temperature, the lifetime of a pavement is divided into three main seasons: spring/autumn[1] (6 months per year), summer (3 months per year) and winter (3 months per year) [19]. The equivalent temperature is different during each season. Its values are shown in the table below along with yearly distribution of traffic [10, 22].

Table 1. Seasonal temperature and traffic distribution in Poland

Season	Spring/autumn	Summer	Winter
Equivalent temperature	10 °C	23 °C	-2 °C
Traffic load[2]	50%	30%	20%

2. PAVEMENT MODELLING

2.1. CHOSEN PAVEMENT STRUCTURE TYPES

Seven pavement structures were chosen for the purpose of this paper from [19] and [20]. Only KR4 category was considered (the highest category that has a constant traffic load range in both catalogue editions[3]) . Chosen pavement structure types are listed in Table 2.

[1] It is assumed for simplicity that these seasons are similar regarding temperature conditions and can be grouped together.
[2] Expressed as percentage of total yearly traffic load.
[3] Traffic load range for category KR5 changed from 7.3-14.6 mln to 7.3-22 mln load axes during pavement lifetime. Traffic load range for category KR6 changed from >14.6 mln to 22-52 mln load axes during pavement lifetime [19, 20].

Table 2. Pavement structure types

	old catalogue	*new catalogue*
Flexible pavement with aggregate subbase	A	A1
Flexible pavement with asphalt concrete subbase	C	B
Semi-rigid pavement with HBM subbase	E	C
Semi-rigid pavement with lean concrete subbase	F	- [1]

Figure 1 presents the differences in layers' properties (incl. thickness) for analyzed pavement types.

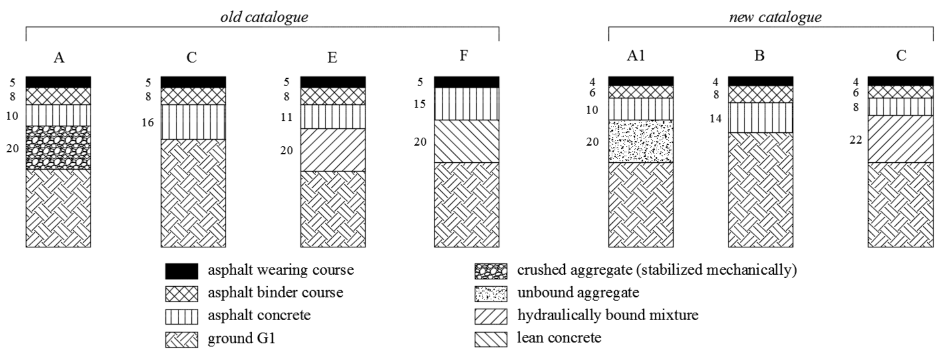

Figure 1. Pavement structure types

Each pavement structure is uniquely defined by the properties of each consecutive layer: its thickness and the values of Young's modulus E and Poisson's ratio ν during each of the seasons. An example of pavement characteristics is shown in Table 3.

Table 3. Sample pavement characteristics – type "A"

Layer no.	Layer	Thickness [cm]	Spring/autumn		Summer		Winter	
			E [MPa]	ν	E [MPa]	ν	E [MPa]	ν
1	Asphalt wearing course	5	10300	0.30	2800	0.40	19300	0.25
2	Asphalt binder course	8	10100	0.30	3000	0.40	18800	0.25
3	Asphalt concrete subbase	10	9600	0.30	3000	0.40	18100	0.25
4	Crushed aggregate subbase	20	400	0.30	400	0.30	400	0.30
5	Ground G1 subgrade	250	100	0.30	100	0.30	100	0.30

[1] As no typical structure of semi-rigid pavement with lean concrete subbase is proposed in the *new catalogue*, only one model – according to the *old catalogue* guidelines – was created.

2.2 MODEL DESIGN

Pavement models used for the purpose of this paper were created using ANSYS Mechanical software (release 16.2). This mechanistic approach to pavement design uses finite element analysis as the leading method of performing calculations.

Durability of pavement structure is associated with fatigue – the number of normative load axes of magnitude 100 kN or 115 kN that can be applied to the structure without its failure [13, 15]. The aim of each model is to obtain the response of the structure to one load axis. Then, the durability of the pavement can be found according to the Asphalt Institute Method or the French Method. Three models have been created for each pavement structure (one per season) to account for changing values of the Young's modulus and Poisson's ratio.

Main assumptions are as follows:

- The response of the pavement is fully elastic; all plastic behavior is neglected.
- All pavements modelled in ANSYS are flexible. Semi-rigid pavements are created as two separate models, one with parameters as in phase I (non-cracked pavement) and one with parameters as in phase II (cracked pavement).
- Pavement is modelled as a layered structure with unrestricted number of layers. Each layer is defined by three parameters – thickness, Young's modulus and Poisson's ratio – and is assumed to be homogenous and isotropic.
- The structure itself is weightless.
- Quadrilateral low-order 4-node axisymmetric elements of type SOLID272 are used. They are 2D elements and represent a section of an axisymmetric finite element model.
- The pavement is modelled as an axisymmetric cylinder of radius r = 2m.
- Boundary conditions: movement along X-axis is constrained for the side surface of the cylinder. Movement along Y-axis is constrained for the bottom surface of the cylinder.
- Ground depth must be high enough to ensure deflection is equal to zero at the layer bottom.
- A uniform mesh was chosen. Dimensions of the elements were limited to 2 cm.
- Normative 100 kN load axis load was divided into two 50 kN loads (one per wheel). These in turn were recalculated to pressure p = 650 kPa applied to a circular area of radius r = 15.65 cm per wheel. This pressure was applied to the center of the top surface of the axisymmetric model.
- G1 ground category was chosen. It was assumed that each ground can be improved to fulfill the G1 type requirements, hence this type is most representative.

3. ANALYSIS PROCEDURE

3.1. MECHANISTIC ANALYSIS

Once a model is created in ANSYS Mechanical software for any of the pavement structure types, the solution is found using FEM.

First, stress distribution in X-direction for grid nodes is graphed. For the purpose of semi-rigid pavement analysis (phase I only), the value of X-component of stress σ_x is read at the bottom of the layer bound with hydraulic binders. Then, elastic strain component in X- and in Y-direction is graphed for grid nodes and for subgrid points. For both flexible and semi-rigid pavement analysis the value of X-component of elastic strain ε_h is read at the bottom of asphalt layers; the value of Y-component of elastic strain ε_v is read at the top of the ground layer. Finally, the results are processed according to the Asphalt Institute Method and to the French Method in order to obtain the durability of the pavement.

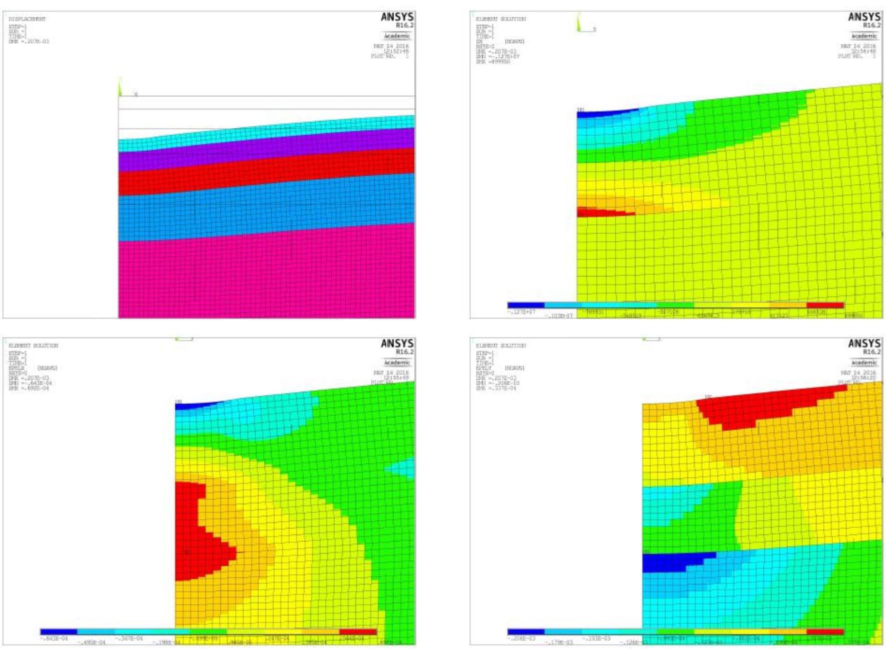

Figure 2. Sample graphs: (a) Deformed shape, (b) X-component of stress, (c) X-component of strain, (d) Y-component of strain

Table 4. Results from ANSYS Mechanical – flexible pavements

Pavement type	ε_h (microstrains)			ε_v (microstrains)		
	spring/autumn	summer	winter	spring/autumn	summer	winter
A	64.6	130	41.1	96.4	172	66.9
C	51.3	127	30.1	42.2	163	19.2
A1	79	152	50.9	121	203	84.7
B	62.2	150	36.9	50.8	192	23.4

Table 5. Results from ANSYS Mechanical – semi-rigid pavements

Pavement type	phase	ε_h (microstrains)			ε_v (microstrains)			σ_x (MPa)		
		spring/autumn	summer	winter	spring/autumn	summer	winter	spring/autumn	summer	winter
F	I	4.34	29.1	7.15	12	16.2	10.1	0.388	0.532	0.323
	II	73.9	153	51.1	120	205	85.1	-	-	-
E	I	20.2	32.6	17.8	24.8	35.4	19.7	0.212	0.306	0.166
	II	63.5	135	38.5	101	186	69.8	-	-	-
C	I	12.6	24.2	14.2	22.8	30.2	19.3	0.326	0.437	0.274
	II	84.3	151	54.5	122	194	84.4	-	-	-

3.2. Transfer functions

3.2.1. Asphalt Institute method

The first durability criterion is fatigue cracking of 20% of the pavement surface. The transfer function is as follows:

$$N_c = 18.4 \cdot 10^M \cdot 6.167 \cdot 10^{-5} \cdot \varepsilon_h^{-3.291} \cdot E^{-0.854} \tag{3.1}$$

where:

$$M = 4.84 \cdot \left(\frac{V_a}{V_a + V_v} - 0.69 \right)$$

E - Young's modulus of the bottom asphalt layer [MPa];

ε_h - horizontal tensile strain at the bottom of asphalt layers [-];

V_a - asphalt in the asphalt mix [%]. For the purpose of this paper, a value of $V_a = 10\%$ was adopted[1];

V_v - air voids in the asphalt mix [%]. For the purpose of this paper, a value of $V_v = 8\%$ was adopted[1].

[1] See [19], appendix B, table B – value chosen for asphalt concrete used for the subbase layer.

The second criterion (of permanent vertical deformations) assumes a deformation of 12 mm to be the durability limit. It uses the following transfer function:

(3.2)
$$N_p = \left(\frac{k}{\varepsilon_v}\right)^{1/m}$$

where:

$k = 0.0105$, $m = 0.223$,

ε_v - vertical compressive strain in the subgrade.

Taking into account semi-rigid pavement instead of flexible pavement, one must consider two distinct phases in the lifetime of the former. During phase I, tensile stresses in concrete layer do not reach its flexural strength, causing no cracks in the layer. Hence, the limiting value of stress is the flexural strength of concrete. According to Dempsey's criterion,

(3.3)
$$\log N_f = 11.784 - 12.121 \cdot \left(\frac{\sigma}{R_b}\right)$$

where:

N_f - number of load axes in phase I;

σ - tensile stress in the layer with hydraulic binder [MPa];

R_b - flexural strength of this layer [MPa][1].

Pavement response in phase II (after concrete cracking) is analogical to the response of a flexible structure. Additionally, the overall durability of pavement is lowered by the damage caused in phase I. Generally,

(3.4)
$$N = N_f + N_{II} \cdot \left(1 - \frac{N_f}{N_{cp}}\right)$$

where:

N - overall durability of the semi-rigid pavement;

N_f - number of load axes in phase I;

N_{II} - durability of the pavement calculated as for a flexible pavement, with Poisson's ratio and Young's modulus of cracked concrete;

N_{cp} - durability of the pavement calculated as for a flexible pavement, with Poisson's ratio and Young's modulus of non-cracked concrete.

[1] The flexural strength is estimated to be equal to 10% of the compressive strength. For lean concrete, minimum admissible value of compressive strength is 6 MPa ([19], Appendix B, point 2.3.4.). For hydraulically bound mixtures, minimum admissible value is equal to 6 MPa ([21], table 1.2.). Hence, for both types, the value of R_b = 0.6 MPa.

3.2.2. FRENCH METHOD

Authors of the *new catalogue* have chosen not to disclose their choice of coefficients used in the transfer functions for this method [3, 16]. The choice of proper parameters in order to simulate their findings is obstructed by the differences in French and Polish regulations regarding laboratory testing, material classification and admissible traffic loading. Justification for the choice of parameters for the purpose of this paper is presented below.

The fatigue cracking criterion uses the following transfer function:

$$N_c = \left[\frac{\varepsilon_h}{\varepsilon_6(10^{\circ}\mathrm{C}, 25\,\mathrm{Hz}) \sqrt{E(10^{\circ}\mathrm{C}) / E(\theta_{eq})} \cdot k_r \cdot k_c \cdot k_s} \right]^{1/b} \cdot 10^6 \tag{3.5}$$

where:

ε_h - horizontal tensile strain at the bottom of asphalt layers;

$\varepsilon_6(10^{\circ}\mathrm{C}, 25\,\mathrm{Hz})$ - horizontal tensile strain that causes destruction of a sample by bending with 50% probability, after N_c loading cycles, in equivalent temperature $10^{\circ}\mathrm{C}$, with frequency of loading application characteristic for stresses in that layer. The value taken as $\varepsilon_6(10^{\circ}\mathrm{C}, 25\,\mathrm{Hz}) = 115 \cdot 10^{-6}$ for subbase layer[1];

$E(10^{\circ}\mathrm{C})$ - stiffness of the SMA under the equivalent temperature of $10^{\circ}\mathrm{C}$ [MPa]. For all investigated pavement structures, $E(10^{\circ}\mathrm{C}) = 9600$ MPa [2];

$E(\theta_{eq})$ - stiffness of the SMA under the equivalent temperature in the investigated season [MPa];

b - slope of the fatigue curve. The generally accepted value is $b = -0.2$ for all typical SMA mixes[3];

k_r - risk coefficient. For the purpose of this paper, value taken as $k_r = 0.75$, which corresponds to 2% risk that the pavement will reach its durability before the end of the period for which it was designed (in Poland - 20 years), assuming no strengthening measures have been undertaken in the meantime[4];

k_c - material coefficient. It is used to adjust the mechanistically obtained results to the actual response of the mixtures. The value taken as $k_c = 1.3$ for subbase layer [7];

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[1] See [6], Table 9.
[2] Parameter taken from [19], Appendix B, Table C.
[3] See [6].
[4] It is assumed is the French Method that minimum risk (expressed as percentage) is equal to 2% and can be higher for lower traffic load categories [6].

k_s - ground coefficient. The coefficient accounts for local worsening of ground conditions below the pavement layers. For simplicity, this value is taken as $k_s = 1$ (assuming no reduction in durability of the pavement).

The permanent deformation criterion uses the following transfer function:

(3.6) $$N_p = \left(\frac{\varepsilon_v}{k}\right)^{-1/m}$$

where:

$m = 0.223$;

ε_v - vertical compressive strain in the subgrade;

k - empirical coefficient. It is assumed to be equal to 0.016 for KR1 and KR2 traffic classes and 0.012 for the remaining classes by analogy of the Polish and French traffic classifications.

The response of semi-rigid pavement is considered in two separate phases, analogically as in the Asphalt Institute Method. Instead of Dempsey's criterion Eq. (3.7) is used:

(3.7) $$N_f = \left(\frac{\sigma_{t,ad}}{\sigma_6 \cdot k_r \cdot k_c \cdot k_s \cdot k_d}\right)^{1/b} \cdot 10^6$$

where:

N_f - number of load axes in phase I;

$\sigma_{t,ad}$ - tensile stress in the layer with hydraulic binder [MPa];

σ_6 - flexural strength of concrete determined after 360 days (after 106 load cycles) [MPa][1];

k_d - discontinuity coefficient for the hydraulic binder layer (for hydraulic binders with low Young's modulus value is taken as $k_d = 1$);

b, k_r, k_c, k_s - coefficients (analogically as for the formula 3.5).

3.3. EFFECT OF SEASONAL TEMPERATURE VARIATIONS

In pavement design, cyclic loading is considered to have a constant, normative amplitude. Hence, each singular pavement structure has a durability equal to n load cycles. The approach gets more complex when the pavement response changes in time. This response depends on two parameters:

[1] The flexural strength is estimated to be equal to 10% of the compressive strength for lean concrete; as minimum admissible value of compressive strength is 6 MPa [19], σ_6 = 0.6 MPa. For hydraulically bound mixtures of lowest admissible class (i.e. C5/6), the value is taken as σ_6 = 0.57 MPa [5].

Young's modulus E and Poisson's ratio ν of each layer of the structure. These two parameters are strongly dependent on temperature for all layers that contain asphalt mixtures (the subbase and layers bound with hydraulic binders do not change their properties significantly in the typical yearly temperature range in Central Europe). Hence, the durability of the pavement is different in each season.

Miner's hypothesis should be applied to establish cumulative damage done by traffic load throughout the year. The hypothesis states that if n load cycles of constant amplitude cause structure failure, then one load of such magnitude causes partial damage equal to $1/n$ of failure. When accumulated damage reaches 1[1], failure occurs.

(3.8) $$D=\sum_{k=1}^{k=m}\frac{n_k\left(\Delta\sigma_k\right)}{N_k\left(\Delta\sigma_k\right)}$$

where:

D - accumulated damage. When this value reaches 1, failure occurs;

$n_k\left(\Delta\sigma_k\right)$ - number of load cycles applied (of associated load amplitude $\Delta\sigma_k$);

$N_k\left(\Delta\sigma_k\right)$ - maximum number of load cycles from S-N curve (for associated load amplitude $\Delta\sigma_k$).

In the case of changing seasonal parameters and constant load magnitude, the accumulated damage D inflicted on the pavement can be calculated according to Eq. (3.8) as:

(3.9) $$D=\frac{n_{sa}}{N_{sa}}+\frac{n_s}{N_s}+\frac{n_w}{N_w}$$

where:

$n_{sa}=0.5\,n$ - number of load axes applied in spring/autumn during the lifetime of the pavement[2];

$n_s=0.3\,n$ - number of load axes applied in summer during the lifetime of the pavement;

$n_w=0.2\,n$ - number of load axes applied in winter during the lifetime of the pavement;

n - total number of load axes applied during the lifetime of the pavement;

N_{sa}, N_s, N_w - durability of the pavement in spring/autumn, summer or winter respectively, with Young's modulus E and Poisson's ratio ν taken for seasonal equivalent temperature (see table 1), calculated according to the Asphalt Institute Method or the French Method.

Eq. (3.9) can be transformed, assuming $D = 1$:

(3.10) $$n=\left(\frac{0.5}{N_{sa}}+\frac{0.3}{N_s}+\frac{0.2}{N_w}\right)^{-1}$$

[1] Empirical research gives various values here (see e.g. [8]), but for design purposes it is most common to use 1.

[2] The number of load axes that causes failure is equal to this value by catalogue design approach (when the influence of seasonal temperature variations is neglected).

The durability of the pavement is calculated to be equal to n normative load axes.

3.4. RESULTS

Tabularized results are shown in table 6. The symbols are as follows:

N_{sa} - maximum admissible number of load axes assuming no influence of seasonal temperature variations on durability (i.e. with constant equivalent temperature for the whole year).

N_{year} - maximum admissible number of load axes considering the influence of seasonal temperature variations on durability (i.e. with seasonal equivalent temperatures).

$\Delta N = \dfrac{N_{year} - N_{sa}}{N_{year}}$ - relative drop in durability.

Table 6. Final results

Model description			Asphalt Institute Method			French Method		
flexible/semi-rigid	catalogue	type	N_{sa} [mln]	N_{year} [mln]	ΔN	N_{sa} [mln]	N_{year} [mln]	ΔN
flexible	old	A	6.19	3.67	41%	15.75	13.79	12%
	old	C	13.22	4.80	64%	49.88	23.86	52%
	new	A1	3.19	2.08	35%	5.76	5.54	4%
	new	B	7.01	2.72	61%	19.04	9.98	48%
semi-rigid	old	F	56.20	11.19	80%	15.88	10.76	32%
	old	E	1792.47	695.15	61%	141.98	66.28	53%
	new	C	184.02	55.50	70%	18.61	13.91	25%

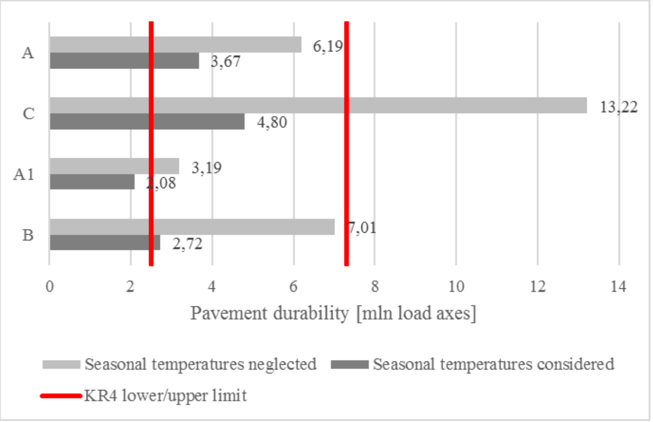

Figure 3. Flexible pavements – Asphalt Institute Method

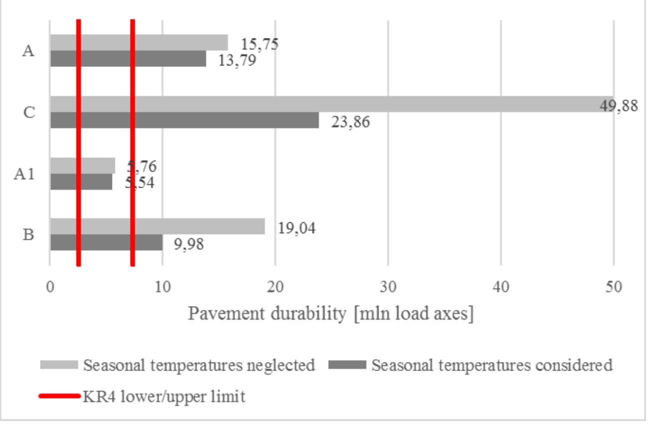

Figure 4. Flexible pavements – French Method

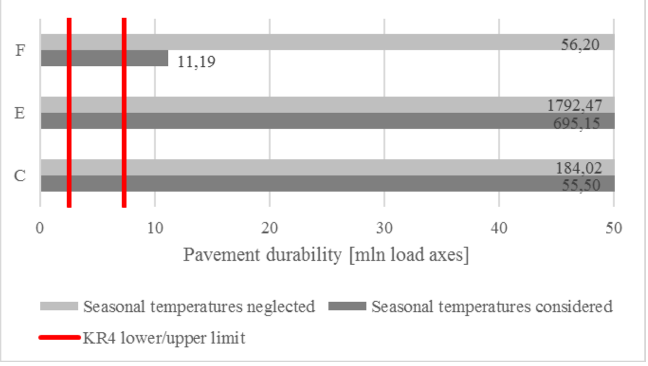

Figure 5. Semi-rigid pavements – Asphalt Institute Method

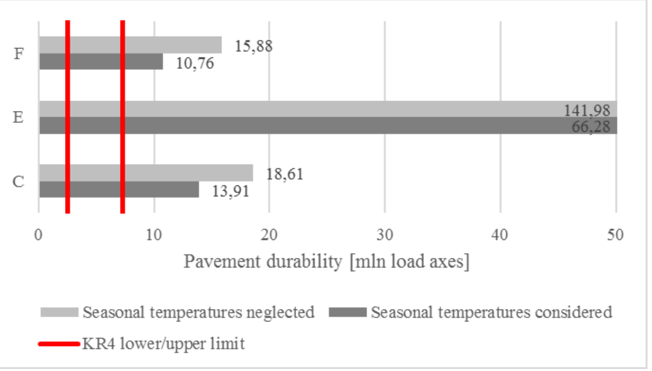

Figure 6. Semi-rigid pavements – French Method

4. CONCLUSIONS

Durability of pavement obtained when considering the effect of seasonal temperature variations is always lower than the one obtained with a constant annual equivalent temperature parameter.

Relative drop in durability is lower in the case of flexible pavement – it ranges from 4% (which can be considered negligible) to 64%. In the case of semi-rigid pavement type, the differences are striking – the drop ranges from 25% to as high as 80%.

All investigated cases have been evaluated for application for KR4 traffic category. For 12 out of 14 cases, the applicability did not change due to calculated significant drop in durability. This fact suggests big safety margins in both applied methods and implies overdesign.

Generally, the French Method displays higher accuracy than the Asphalt Institute Method. Additionally, the results obtained for semi-rigid pavements are much more realistic.[1] Results obtained by the Asphalt Institute Method suggest that the pavement types E and C are either extremely overdesigned (the latter would be suitable for KR6 traffic load category, even after the drop in durability of 60-70% is taken into account), or that the design procedure is flawed. The relative error is much lower when using the French Method, and the results seem much more reasonable, placing the C pavement slightly above the KR4 category.

Lowest drop in durability was noted for flexible pavements type “A” and type “A1”. It can be observed that the presence of an additional non-asphalt subbase lowers the influence of seasonal temperature variations on pavement durability.

[1] Analysis by the Asphalt Institute Method estimates the durability of a semi-rigid E pavement to be about 1.8 billion load axes. The empirical research shows that in reality this number is manifold lower.

The mechanical model of pavement structure to be analyzed in this paper is purely elastic and characterized by Young's modulus and Poisson's ratio. More sophisticated material model of asphaltic layers should take into account viscoelastic properties as it was presented in [17, 18]. Such an approach would make possible the analysis of both temperature influence as well as various velocities of moving loads.

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MECHANISTYCZNO-EMPIRYCZNE PROJEKTOWANIE ASFALTOWYCH NAWIERZCHNI DROGOWYCH Z UWZGLĘDNIENIEM SEZONOWYCH ZMIAN TEMPERATURY

Keywords: MES, nawierzchnie asfaltowe, projektowanie mechanistyczno-empiryczne, trwałość zmęczeniowa, zmienność temperatury.

STRESZCZENIE:

Celem pracy jest analiza wpływu sezonowych zmian temperatury na trwałość podatnych i półsztywnych nawierzchni drogowych. W artykule porównano trwałość nawierzchni wyznaczoną przy założeniu całorocznej temperatury ekwiwalentnej 10°C oraz przy założeniu różnych temperatur ekwiwalentnych odniesionych do trzech pór roku: wiosny/jesieni, lata oraz zimy.

Do analizy wybrano siedem typów nawierzchni odpowiednich do obciążenia ruchem kategorii KR4 według dwóch edycji „Katalogu typowych konstrukcji nawierzchni podatnych i półsztywnych" (z 1997 roku i z 2014 roku).

Wszystkie nawierzchnie zostały zamodelowane jako konstrukcje składające się z warstw o określonych parametrach. Każdą z warstw zdefiniowano poprzez jej grubość, moduł Younga oraz liczbę Poissona υ. Przygotowano trzy modele w odniesieniu do każdego z typów nawierzchni (jeden dla każdej pory roku), by uwzględnić zależność parametrów E oraz υ od temperatury. Stan naprężeń i odkształceń poszczególnych nawierzchni uzyskano w programie ANSYS Mechanical poprzez analizę wykorzystującą metodę elementów skończonych (MES).

Trwałość zmęczeniową każdej z siedmiu analizowanych nawierzchni wyznaczono dwukrotnie, stosując kryteria Instytutu Asfaltowego oraz kryteria metody francuskiej.

Przedstawione w pracy wyniki wykazują, że trwałość nawierzchni otrzymana przy uwzględnieniu sezonowych zmian temperatury jest zawsze niższa niż ta uzyskana przy założeniu stałej temperatury ekwiwalentnej. Spadek trwałości jest niższy w przypadku nawierzchni podatnych – od 4% do 64%. W przypadku nawierzchni półsztywnych spadek trwałości osiąga od 25% do 80%.

Wszystkie wyniki zostały ocenione pod względem możliwości zastosowania analizowanej nawierzchni do obciążenia ruchem kategorii KR4. Tylko w jednym z czternastu przypadków trwałość nawierzchni spadła poniżej dolnej granicy dopuszczającej nawierzchnię do tej kategorii. Ten rezultat sugeruje, że w obu analizowanych metodach obliczeniowych wykorzystuje się wysokie marginesy bezpieczeństwa.

Metoda francuska jest bardziej dokładna niż metoda Instytutu Asfaltowego – spadek trwałości waha się od 4% do 53% dla metody francuskiej i od 35% do 80% dla metody Instytutu Asfaltowego. Wyniki otrzymane w przypadku nawierzchni półsztywnych, metodą Instytutu Asfaltowego, są niewiarygodne – nawet po uwzględnieniu spadku trwałości o 60-70% nawierzchnie te są obliczeniowo adekwatne dla kategorii KR6.

Najniższy spadek w trwałości zanotowano przy podatnych nawierzchniach typu A i A1, w których stosuje się dodatkową podbudowę z materiałów niewrażliwych na zmianę temperatury (kruszywo). Dzięki temu wpływ sezonowych zmian temperatury na ich trwałość jest niższy – 4% do 12% według metody francuskiej i 35% do 41% według metody Instytutu Asfaltowego.