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INFLUENCE OF ASPHALT VISCO-ELASTIC PROPERTIES ON FLEXIBLE PAVEMENT PERFORMANCE

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Abstract. Even though every layer of pavement structure is important and affects pavement performance, the asphalt layers visco-elasticity plays significant role. Bitumen properties, as well as asphalt mixture properties, vary depending on temperature and loading conditions. These variations influence entire pavement bearing capacity and has to be evaluated in pavement design. The main challenge is material behaviour description through simple models to incorporate them to pavement design. Generally, pavements are designed using Multilayer Elastic Theory assuming that all materials are elastic, isotropic, and homogenous. This paper presents analysis of two pavement structures response calculated according to three pavement design approaches. The dynamic modulus and phase angle of asphalt mixtures was estimated using Hirsch model after binder complex shear modulus tests. The visco-elastic behaviour was described with rheological Huet-Sayegh model and pavement responses estimation was done using MnLayer and ViscoRoute2 software. The analysis reviled static and dynamic load influence on pavement structure based on elastic and visco-elastic properties of asphalt layers. This allowed optimisation of layer thicknesses and determination of more cost beneficial pavement structure with appropriate performance.

Keywords: asphalt dynamic modulus, pavement design, pavement performance, predictive model, master curve, multilayer-elastic theory.

1. Introduction

Safe, functional, comfortable, qualitative, and economical road network infrastructure is a priority for every developed country. The condition of road infrastructure reflects the economic level of the state. Flexible asphalt pavement is most common in Lithuania due to well-known technology, quick construction, and easy maintenance. Asphalt pavement in Lithuania is designed referring to a standardized pavement structure catalogue (*KPT SDK 07 Design Rules of Road Standardized Pavement Structures*) that is based on experience and historical-empirical observation.

In some cases, the standardized catalogue becomes ineffective, such as when new or reconstructed pavement deteriorates more quickly than it was designed to. This is particularly relevant in road sections where the design load was underestimated (with overloaded vehicle traffic), as well as areas where acceleration, deceleration, and static loads can appear (Vaitkus *et al.* 2014a, 2014b). Likewise, climate change, unusual high and low temperature extremes, as well as increment of their frequency and duration affect pavement and lead to different performance and deterioration than was empirically observed in the past (Wistuba, Walther 2013). The road surface and pavement structure are operated under aggressive and complicated to forecast conditions of traffic loads and temperature. Therefore, the pavement design methods and empirical models must be periodically updated and revised.

According to *Richtlinien für die rechnerische Dimensionierung des Oberbaus von Verkehrsflächen mit Asphaltdeckschicht RDO Asphalt 09* and *Guide for the Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures MEPDG* pavement design procedures the material tests and analysis methods enables to evaluate the mechanical behaviour of each pavement layer, estimate the pavement response to traffic loads under certain temperatures, and forecast the pavement performance. These methods lead to design unique (non-standard) pavement structures and analytically evaluate the application of new materials and mixtures through their mechanical properties. This leads to optimisation of pavement layer thickness and ensures more efficient use of road infrastructure development and maintenance funds.

The research focuses on the application of dynamic modulus of asphalt concrete (AC) mixture predicted with

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Hirsch model corresponding to the visco-elastic properties of bitumen binder, to the flexible pavement design and performance forecast.

2. Analysis of the asphalt pavement design, response, and performance methods

Generally, the flexible (asphalt) pavement is designed by selecting the layer thickness and material from standardized catalogue of pavement structures according to the total number of equivalent axles (ESAL). This approach is based on empirical data underestimating the pavement responses, damage and performance. The advanced pavement design methods based on mechanical properties of materials allow prediction of pavement performance and evaluation of the structure resistance to fatigue and permanent deformation.

Pavement performance models presented in separate procedures Shell, Asphalt Institute (AI), RDO Asphalt, Transport and Road Research (TRR), Belgian Road Research Centre (BRRC), and MEPDG are different, as well as their coefficients for limit load (or strain) estimations (Huang 2004). These performance models can be based either on fatigue and permanent deformation laboratory tests or experimental road sections (Priest, Timm 2005) and accelerated pavement testing (Al-Khateeb *et al.* 2007) observations. Pavement performance models have to be calibrated according to local climate and traffic conditions (Li *et al.* 2011). Typically, the limit (critical) number of loads are found after performing the fatigue tests and calibrated according to field observations.

During pavement design the structure response under load are computed in critical locations of stress and strain. Stress and strain applied in pavement performance models allows estimating limit loads of pavement's resistance to (Di Benedetto *et al.* 1996; Hogentogler, Terzaghi 1929; Lundstrom *et al.* 2007; Rowe, Bouldin 2000):

– fatigue cracking – strain in the bottom of asphalt base layer (Eq (1));
– longitudinal cracking – strain in the of asphalt surface;
– hydraulically bound base fatigue cracking – strain in the surface of base layer;
– permanent deformation – stress in the top of the unbound base, subbase layers and subgrade (Eq (2)).

In this research the RDO Asphalt pavement performance models (Eqs 1 and 2) were used for limit load estimation.

$$N_{i-j}^{rib}(AC) = \frac{SF}{F} a \varepsilon_{y,i-j}^{k}, \tag{1}$$

$$N_{i-j}^{rib}\left(AggB;SG\right) = 10^{\frac{1}{0.7}\left(\frac{0.00875 E_{V_2}}{\sigma_{z,i-j}^{pr} \cdot \gamma}\right)}, \tag{2}$$

where SF – coefficient for indirect tension fatigue tests; F – safety coefficient; a, k – coefficient detriment from fatigue test; $\varepsilon_{y,i-j}$ – horizontal strain in the bottom of asphalt base layer under load i and temperature j, ‰; E_{V_2} – deformation modulus of subgrade or aggregate base, MPa; $\sigma_{z,i-j}^{pr}$ – vertical stress in the top of aggregate base layer of subgrade under load i and temperature j, MPa; γ – safety coefficient.

In most cases the multilayer-elastic theory is used for flexible (asphalt) pavement design and response estimation, where materials are described over Yung modulus and Poisson ratio. The modulus determined at specific load, frequency and temperature conditions allows estimating the pavement response at those specific conditions and can be directly added to performance models (Park 2004). It is accepted that pavement will perform well if the design loads (or stresses) is less than limit load (or stresses) in all expected conditions during pavement service. Therefore, the Minner hypothesis (Eq (3)) should be confirm: the design loads should be less than limit loads over all pavement design life. In other words, the total sum of design and limit load ratio should less than one (Hopman *et al.* 1989).

$$\sum\nolimits_{Minner} = \sum_{i=1}^{m}\sum_{j=1}^{k}\frac{N_{i-j}^{pr}}{N_{i-j}^{rib}} \leq 1, \tag{3}$$

where N_{i-j}^{pr} – the design load number of axle load class i under temperature interval j; N_{i-j}^{rib} – the limit (critical) load number of axle load class i under temperature interval j; m – axle load class (m = 2 t, 4 t, 6 t, ... i); k – pavement surface temperature interval (k = −17.5 °C, −12.5 °C, −7.5 °C ... j).

The advanced pavement design concept based on performance is fulfilled by following:

1) the characterisation of design loads, climate and geological conditions;

2) the estimation of minimum thickness of resistant to frost;

3) the assumption of thickness of pavement layers under expected subgrade condition and the selection (or test) of material types according to their mechanical properties;

4) the calculation of pavement response (stress and strains) under loads at specific temperature and moisture conditions;

5) the estimation the limit loads (or stresses) corresponding to pavement performance criteria;

6) the conformation of the Minner hypothesis: if Minner rule is true, the design pavement structure should perform well at specified performance criteria, if no, the changes in the layer thicknesses and martials have to be provided (staring from third position) and the additional calculations should be done.

3. Asphalt mixture elastic and visco-elastic behaviour

Because of bituminous binder nature the asphalt mixture behaves as visco-elastic material at most loading and temperatures and only in low temperatures the behaviour become elastic and brittle. This visco-elastic behaviour of asphalt layers should be evaluated at specific traffic loading and climate conditions, consequently the pavement responses and performances estimated. To do so, the complex modulus and phase angle of asphalt mixtures have to be determined at various temperature and dynamic loading frequency conditions. At linear visco-elastic range the asphalt mixture and binder behaviour are assumed to be thermo-rheologically simple, which means that time-temperature superposition principle can be used for complex modulus master curve construction (Chehab *et al.* 2002; Dickinson 1974; Yusoff *et al.* 2011)and master curves relating the absolute value of the modulus to the reduced frequency were constructed. These were found to fit closely to an equation which is one arm of an hyperbola whose asymptotes represent the purely viscous and purely elastic behavior expected at infinitely low and infinitely high frequencies, respectively. The rapidity with which an asphalt changes from a viscous to an elastic response as the frequency of loading increases (shear susceptibility parameter. The complex modulus and phase angle master curve fully describes the behaviour of asphalt mixture and could be applied for analytical methods and pavement design procedures.

The shifted master curve of asphalt mixtures and binders has a sigmoidal function shape (Pellinen *et al.* 2003). Rowe *et al.* (2009) used a Generalized Logistic Sigmoidal Model (or Richards Model) for asphalt mixture dynamic modulus (Eq (4)) and phase angle (Eq (5)) characterisation, which can be used for binder dynamic shear modulus. The shift factor estimated for each isoline can be described as a polynomial function (Eq (7)) regarding to temperature.

$$\log\left|G(\omega)^*\right| = \delta + \frac{\alpha}{\left[1+\lambda e^{\beta+\gamma(\log(\omega_R))}\right]^{1/\lambda}}, \quad (4)$$

$$\varphi(\omega) = -90\alpha\gamma\frac{e^{\beta+\gamma(\log(\omega_R))}}{\left[1+\lambda e^{\beta+\gamma(\log(\omega_R))}\right]^{(1+1/\lambda)}}, \quad (5)$$

$$\log(\omega_R) = \log(\omega a_T), \quad (6)$$

$$\log(a_T) = a_2T^2 + a_1T + a_0, \quad (7)$$

where $G^*(\omega)$ and j(ω) – the complex shear modulus and phase angle at an angular frequency ω; δ, α, β, γ, λ – parameters, which defines the shape of the master curve; ω_R – reduced frequency (rad/s) at reference temperature; a_T – the shift factor; T – isoline temperature; a_0, a_1, a_2 – polynomial function fitting parameters.

The characteristics for definition of visco-elastic behaviour of asphalt mixtures can be experimentally tested or predicted (estimated) using Olard-DiBenedeto (Olard, Di Benedetto 2003), Andrei-Witczak (Andrei *et al.* 1999), Hirsch (Christensen *et al.* 2003)rational and effective model for estimating the modulus of asphalt concrete using binder modulus and volumetric composition. The model is based upon an existing version of the law of mixtures, called the Hirsch model, which combines series and parallel elements of phases. In applying the Hirsch model to asphalt concrete, the relative proportion of material in parallel arrangement, called the contact volume, is not constant but varies with time and temperature. Several versions of the Hirsch model were evaluated, included ones using mastic as the binder, and one in which the effect of film thickness on asphalt binder modulus was incorporated into the equation. The most effective model was the simplest, in which the modulus of the asphalt concrete is directly estimated from binder modulus, VMA, and VFA. Models are presented for both dynamic complex shear modulus (|G*|, improved Hirsch (Christensen, Bonaquist 2015) or similar models (Dongre *et al.* 2005; Francken, Verstraeten 1974; Huet 1963; Sakhaeifar 2011; Sayegh 1965). These models combine bitumen viscosity or dynamic shear modulus and the volumetric composition mixture aggregates. Pellinen *et al.* (2007) investigated the application of different prediction models and the calculation errors. Researchers emphasized that these models should be used with caution in assessing the characteristics of the asphalt mixture at high temperatures, where the average prediction errors approached up to 40%, while the experimental results variation is normally less than 20% (Pellinen *et al.* 2007).

The Hirsch model (Eqs (8), (9) and (10)) adaptation is analysed in this research work to predict the complex modulus of asphalt mixture. The Hirsch model selected because it is unrelated to specific gradation and sieve sizes or glass modulus assumptions.

$$\left|E^*\right|_{mix} = Pc\left(E_a\left(1-\frac{VMA}{100}\right)+3\left|G_b^*\right|\left(\frac{VFA\cdot VMA}{10000}\right)\right)+ (1-Pc)\left(\frac{1-\frac{VMA}{100}}{E_a}+\frac{VMA}{3VFA\left|G_b^*\right|}\right)^{-1}, \quad (8)$$

$$Pc = \frac{\left(P_0+\frac{VFA\cdot 3\left|G_b^*\right|}{VMA}\right)^{P_1}}{P_2+\left(\frac{VFA\cdot 3\left|G_b^*\right|}{VMA}\right)^{P_1}}, \quad (9)$$

$$\varphi = -9.5(\log Pc)^2 - 39\log Pc + 9.6, \quad (10)$$

where E_a – mineral aggregate modulus, psi^2; Pc – aggregate contact volume; VMA – voids in the mineral aggregate, %; VFA – voids filled with the binder, %; $\left|G_b^*\right|$ – shear modulus of the binder, psi; P_0, P_1, P_2 – model fitting parameters; φ – phase angle, °.

In this research the Hirsch model is used for the dynamic modulus of asphalt mixture estimation based on mixture volumetric content and binder mechanical properties. Multilayer theory is applied for pavement structure response calculations. The visco-elastic behaviour of asphalt layers under time and temperature is based on the Huet-Sayegh model.

The visco-elastic behaviour of asphalt mixtures was determined by the rheological Huet-Sayegh model (Huet 1999, 1963; Sayegh 1965). Huet-Sayegh model (Eq (9)) is based on master curve of dynamic modulus of asphalt mixture. This model precisely predicts both the bituminous binder and asphalt mixes visco-elastic behaviour at any temperature and frequency (Di Benedetto *et al.* 2004; Chabot *et al.* 2010; Chupin *et al.* 2012; Duhamel *et al.* 2005).

$$E^* = E_0 + \frac{E_\infty - E_0}{1+\delta\left(i\omega\tau(\theta)\right)^{-k} + \left(i\omega\tau(\theta)\right)^{-h}}, \quad (11)$$

$$\tau(\theta) = e^{A_0 + A_1\theta + A_2\theta^2}, \quad (12)$$

were E_0 – *static* modulus, when $\omega \to 0$; E_∞ – *glassy* modulus, when $\omega \to \infty$; i – complex number, $i^2 = -1$; ω – angular frequency, $\omega = 2\pi f$; f – frequency, Hz; h, k – exponents of the parabolic dampers, when $1 > h > k > 0$; δ – positive non-dimensional constant; t – time parameter; A_0, A_1, A_2 – constants of function.

4. Experimental tests and methodology

4.1. Materials

Experimental tests were conducted with three wearing layer, two binder layer, and two base layer asphalt mixtures types where two binder types were used. Four of them where produced in the batch plant in 2015, and three cored from the Section No. 12 of the Road of Experimental Pavement Structures, which was constructed in 2007 (Čygas *et al.* 2008; Vaitkus *et al.* 2012). Table 1 presents the specification of asphalt mixtures and bitumen binders and Fig. 1 shows grading curves of tested mixtures.

Flexible pavement modelling and response estimations were done for the Section No. 12 of the Road of Experimental Pavement Structures (Fig. 2, DK II). The unbound aggregate base and subbase layer and subgrade elastic modulus were determined according to material types, measured deformation modulus during construction, and analysed deflection basin measured from Falling Weight Deflectometer (FWD). The elastic modulus of base layer from crushed dolomite is 400 MPa, of subbase layer from sand – 120 MPa, and of subgrade from stabilised soil – 120 MPa. The Poisson

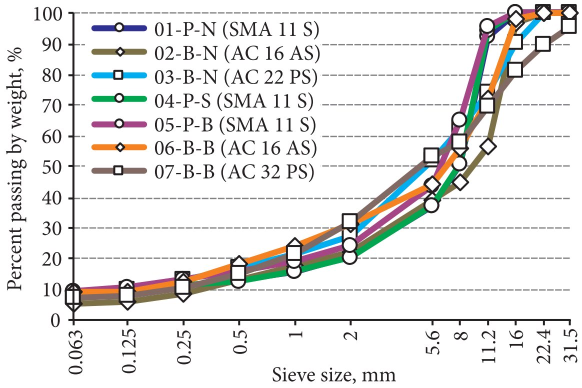

Fig. 1. Grading curve of tested mixtures

Table 1. The specification of asphalt mixtures and bitumen binders

Characteristic	Labels and specification						
Asphalt mixture type	SMA	AC	AC	SMA	SMA	AC	AC
Nominal maximum aggregate size NMAS, mm	11	16	22	11	11	16	32
Layer of asphalt mixture[1)]	V	A	P	V	V	A	P
Mineral type of aggregate[5)]	Gn	Gn	Gv	Gn	Gn	Gn	D
Code of asphalt mixture	01-P-N	02-B-N	03-B-N	04-P-S	05-P-B	06-B-B	07-B-B
Code of binder	01-P-A	02-B-A	03-B-A	04-P-R	05-P-R	06-B-R	07-B-R
Air voids content, %	1.50	6.30	7.20	3.00	0.50	3.70	7.40
Production years	2015	2015	2015	2015	2007	2007	2007
Binder type	PMB45/80-55	50/70	50/70	PMB45/80-55	PMB45/80-55	50/70	50/70
Binder content, %	6.53	3.70	3.88	6.51	6.30	4.21	3.89
Asphalt mixture ageing	N[2)]	N[2)]	N[2)]	S[3)]	R[4)]	R[4)]	R[4)]
Binder ageing	RTFOT	RTFOT	RTFOT	Recovered	Recovered	Recovered	Recovered
Needle penetration at 25 °C, 0.1 mm	46.6	35.5	40.0	32.8	39.3	48.3	27.0
Softening point, °C	69.6	57.0	56.6	68.3	69.6	58.8	64.9
Voids in mineral aggregate VMA, %	16.84	14.99	16.01	18.26	15.1	13.67	16.24
Voids filled with binder VFA, %	91.34	58.02	55.01	83.58	96.79	72.89	54.13

Notes: [1)] mixture of wearing layer – V, mixture of binder layer – A, mixture of base layer – P; [2)] new asphalt mixture after mixing in the batch plant; [3)] long-term asphalt mixture ageing under laboratory conditions (loose mixture aged for 48 h at 100 °C); [4)] mixture aged under natural conditions, cored from Section No. 12 of the Road of Experimental Pavement Structures; [5)] crushed granite – Gn, crushed gravel – Gv, crushed dolomite – D.

coefficient accepted to be constant 0.35 for asphalt mixtures and 0.45 for unbound layers and subgrade.

4.2. Tests methods

The first three binders (noted as 01-P-A, 02-B-A, and 03-B-A) which were taken form batch plant, were short-term aged according to *LST EN 12607-1:2007 Bitumen and Bituminous Binders. Determination of the Resistance to Hardening under the Influence of Heat and Air. Part 1: RTFOT Method.* Recovered binders (noted as 04-P-R, 05-P-R, 06-B-R, and 07-B-R) from long-term aged mixture SMA 11 S (04-P-S) and mixtures cored from pavement, were kept for 8–9 h in toluene then aggregates were separated from binder solution with centrifuge, and finally the binder were separated from solvent using rotary evaporator according to *LST EN 12697-3:2013 Bituminous Mixtures. Test Methods for Hot Mix Asphalt. Part 3: Bitumen Recovery: Rotary Evaporator*. Experimental methods covers the physical (needle penetration, softening point, volumetric and granular content, as well as binder content tests) and (linear visco-elastic range and dynamic shear modulus tests) mechanical properties determination of asphalt binders. All mixtures investigated at the same experimental conditions.

The dynamic mechanical behaviour of binder tested with dynamic shear rheometer (DSR) under oscillation test mode by applying a sinusoidal shear strain to the sample at set angular frequency. The strain sweep (from 0.05% to 32%) test under maximum frequency (15.6 Hz) at lower (10 °C and 46 °C) and higher (46 °C and 82 °C) temperatures (for each plates geometry) were performed to determine the safe strain at linear visco-elastic (LVE) range. 5% decrease of the initial value of the shear modulus is assumed to be the threshold of LVE range (Airey *et al.* 2003; Anderson *et al.* 1994; Marasteanu, Anderson 2000). The dynamic shear modulus and phase angle was determined from the frequency sweep (from 0.156 Hz to 15.6 Hz) tests, which were performed at constant LVE strain and various temperatures (from 46 °C to 10 °C with 8 mm geometry and from 46 °C to 82 °C with 25 mm, with increase every 6 °C). Unshifted isolines of dynamic shear modulus of bitumen were applied to Hirsch model.

4.3. Results analysis and definition of visco-elastic behaviour of asphalt mixtures

With calibrated Hirsch modulus further predictions of asphalt complex modulus were conducted with other frequencies and temperatures. Complex modulus isolines were determined at temperature ranges from 10 °C to 82 °C under frequency range from 0.156 Hz to 15.6 Hz. The master curve of dynamic modulus of asphalt mixture is constructed according to the Gordon-Shaw method (Gordon, Shaw 1994) assuming the validation of time-temperature superposition principle. The pairs of isolines were shifted to fluent master curve by optimising vertical displacement (Gordon, Shaw 1994). The Richards Generalized sigmoidal model was fitted to get master curve function. And the predicted dynamic modulus at mixture test conditions were estimated by using determined master curve functions and shift factors.

The coefficients of master curve function of predicted dynamic modulus according Hirsh model presented in Table 2. The master curve of dynamic modulus, phase angle and shift factors for each asphalt mixture were transformed to Huet-Sayegh model coefficients that are presented in Table 3.

In summary of chapter must be noted, that the linear visco-elastic ranges for dynamic modulus tests should be defined for each different mixtures and binder types. The predicted or observed dynamic modulus of asphalt mixture under pavement service conditions (with determined temperature and load frequency); can be used for pavement performance evaluation. The constructed master curve of asphalt dynamic modulus led to adapt the rheological Huet-Sayegh model for analysis of visco-elastic behaviour.

5. Flexible pavement response modelling and performance prediction

5.1. Pavement structure and load conditions

Flexible pavement response estimated using the analytical methods based on multilayer elastic theory with elastic and visco-elastic material properties. Complex modulus of asphalt mixture implementation into flexible pavement design was executed in this order:

1) define the design parameters;
2) estimate the stress and strains (responses) under different conditions;
3) predict the pavement performance;
4) evaluate cost benefit.

In this analysis part structure response was estimated for two the new (DK I) and aged (DK II) pavement structures (Fig. 2) and analysed with three pavement design approaches. Pavement response (stress and strain) was estimated under 10 t weight static and moving axial load at variable RDO Asphalt 09 and constant temperatures. Temperature variation in asphalt layer was calculated every 10 mm according to Speth (1985) and Hess (1998) method.

Considering the traffic flow, content and speed data from the Road of Experimental Pavement Structures (Vaitkus *et al.* 2012) and other assumptions, the pavement responses estimated according to:

– average speed of 58 km/h, which according to Gillespie *et al.* (1992) cause about 5 Hz loading frequency (≈ 0.035 s) from five axle truck;
– single axle load of 10 t weight (wheel load ≈ 49.82 kN);
– circular pavement and wheel contact area with radius of 150 mm (contact area ≈ 70 686 mm^2);
– tyre pressure of 850 kPa (Kleizienė *et al.* 2015).

The layers of analysed pavement structures had the same thickness. The mechanical properties of unbound base layer and subgrade accepted to be elastic and the same, but the properties of the asphalt layers were different.

Table 2. The coefficients of master curve function of predicted dynamic modulus according Hirsh model

Asphalt mixtures		SMA 11 S (N)	AC 16 AS (N)	AC 22 PS (N)	SMA 11 S (S)	SMA 11 S (R)	AC 16 AS (R)	AC 32 PS (R)
Binder type		PMB 45/80-55	50/70	50/70	PMB 45/80-55	PMB 45/80-55	50/70	50/70
Binder ageing		RTFOT	RTFOT	RTFOT	Recovery	Recovery	Recovery	Recovery
Dynamic modulus $\|E^*\|$ master curve function's Richards coef. (Eq (2))	α	1.985	1.143	2.067	2.367	2.217	1.424	15.180
	β	−0.682	0.363	−0.646	−0.318	−0.411	−1.048	−2.245
	γ	−0.741	−0.495	−0.568	−0.396	−0.411	−0.605	−0.178
	δ	1.785	3.107	2.376	1.919	1.680	2.378	−10.306
	λ	0.361	0.0001	0.0001	0.0001	0.0001	0.0001	1.636
	R^2	*1.000*	*0.999*	*0.999*	*0.997*	*0.999*	*1.000*	*1.000*
	Se/*Sy*	*0.011*	*0.028*	*0.025*	*0.054*	*0.032*	*0.022*	*0.013*
Shift factor (a_T at T = 22 °C) polynomial function's coef. (Eq (5))	a_2	0.001	0.001	0.001	0.001	0.001	0.001	0.001
	a^1	−0.146	−0.143	−0.151	−0.154	−0.155	−0.157	−0.160
	a0	2.926	2.907	3.023	3.052	3.072	3.094	3.176
	R^2	*0.999*	*0.999*	*0.999*	*0.999*	*0.999*	*0.999*	*0.999*
	Se/*Sy*	*0.031*	*0.027*	*0.025*	*0.027*	*0.030*	*0.026*	*0.032*
Phase angle φ function's Richards coef. (Eq (3))	α	5.498	7.122	5.554	7.009	6.994	11.202	237.973
	β	−0.703	−0.476	−2.122	−1.112	−0.770	−1.677	−4.227
	γ	−0.276	−0.136	−0.518	−0.223	−0.388	−0.193	−0.072
	λ	0.714	0.020	1.613	0.0001	3.133	0.0001	0.010
	R^2	*0.968*	*0.990*	*0.999*	*0.998*	*0.989*	*0.977*	*0.996*
	Se/*Sy*	*0.181*	*0.099*	*0.032*	*0.043*	*0.106*	*0.152*	*0.067*

Table 3. The coefficients for Huet-Sayegh model at 20 °C reference temperature

Type of asphalt mixture (ageing)	E_0	E_∞	δ	k	h	A_0	A_1	A_2
SMA 11 S (N)	57	5627	3.53	0.42	0.77	7.08	−0.370	0.00215
AC 16 AS (N)	1101	13961	9.34	0.40	0.40	6.80	−0.328	0.00074
AC 22 PS (N)	202	25483	4.77	0.36	0.82	7.11	−0.364	0.00186
SMA 11 S (S)	63	13233	11.69	0.37	0.37	6.90	−0.343	0.00130
SMA 11 S (B)	43	5874	8.21	0.36	0.36	7.09	−0.361	0.00177
AC 16 AS (B)	82	6038	1.00	0.39	0.39	7.03	−0.342	0.00091
AC 32 PS (B)	4	21747	7.10	0.31	0.31	7.09	−0.355	0.00148

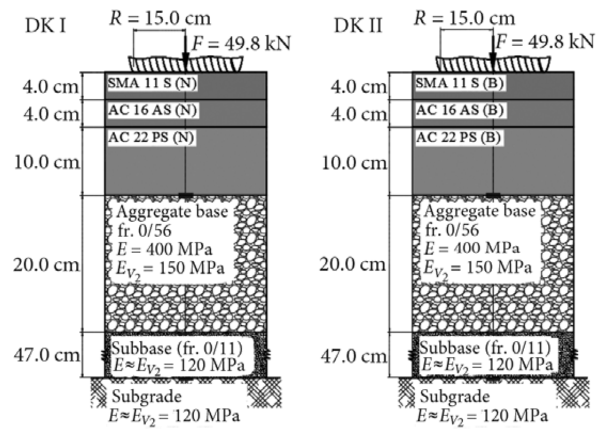

Fig. 2. The pavement structures and analysis points: DKI – with new asphalt mixtures; DKII – with mixtures from eight years old pavement

The new pavement (DKI) composed of asphalt wearing (SMA 11 S (N)), binder (AC 16 AS (N)), and base (AC 22 PS (N)) layers. The aged pavement (DKII) composed of asphalt wearing (SMA 11 S (B)), binder (AC 16 AS (B)), and base (AC 22 PS (B)) layers, which were cored from the Section No. 12 of Road of Experimental Pavement Structures. The aged pavement was operated for eight years and received about 0.6 million of equivalent standard axle loads.

5.2. Pavement response and performance

To evaluate elastic and visco-elastic behaviour of asphalt layers the pavement response was calculated by applying the multi-layered elastic theory based MNLAYER (Khazanovich, Wang 2008) and VISCOROUTE2 (Chabot *et al.* 2010) software. The pavement responses were computed with three pavement design approaches: SK I – flexible pavement response under static load at constant 20 °C temperature for all asphalt layers (applying elastic properties of all layers); SK II – responses under static load and evaluating the temperature variation within asphalt layers (applying elastic properties for all layers); SK III – pavement responses under moving load and evaluating the temperature variation within asphalt layers (applying visco-elastic properties for asphalt layers and elastic properties for other pavement layers).

Figures 3 and 4 present the vertical stress and horizontal strain analysis from static load and constant temperature. The pavement responses variation analysed according to distance from load centre. Calculations have reviled that the highest stresses and strains are located at the loading centre.

The pavement response analysis (Fig. 4) revealed that the moving load (SK III) caused 20% (for DK I) and 19% (for DK II) lower horizontal strain compare to static load (SK II). The decrease of horizontal strain percentages in different pavement structures indicates that the visco-elastic properties influence on pavement structure response decreases with increasing of pavement age.

The pavement performance analysis was focused on the estimation of limit loads number, considering the asphalt base layer resistance to fatigue (Fig. 6), unbound base layer (Fig. 7) and subgrade (Fig. 8) resistance to permanent deformation. The analysis of structural resistance to fatigue showed 51% (SK II) and 55.5% (SK III) lower limit load to asphalt fatigue of aged (DK II) pavement structure

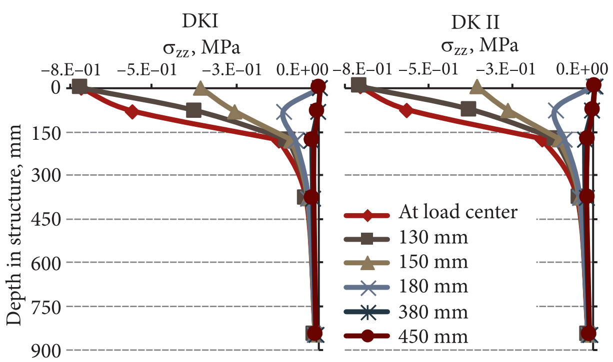

Fig. 3. The vertical stress change in pavement structures depending on distance for load centre

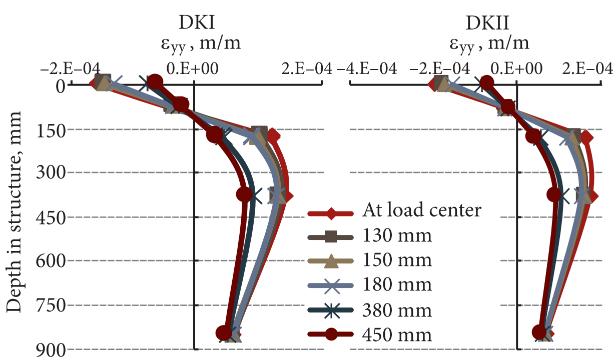

Fig. 4. The horizontal strain change in pavement structure depending on distance for load centre

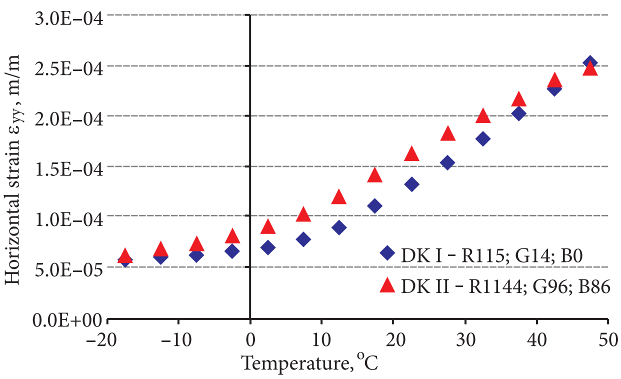

Fig. 5. The horizontal strain depending on pavement surface temperature

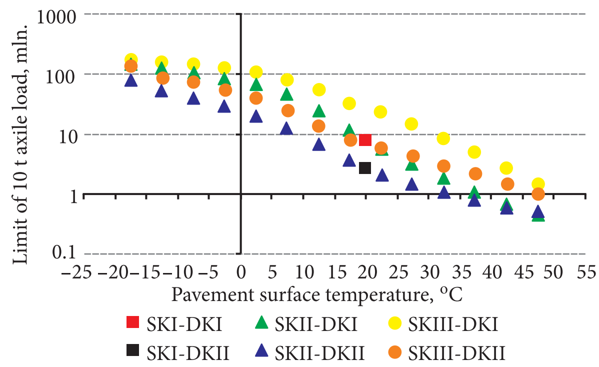

Fig. 6. The limit load according to asphalt base layer resistance to fatigue

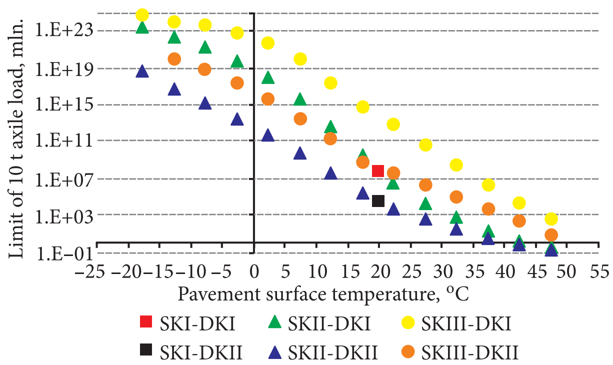

Fig. 7. The limit load according to aggregate base layer resistance to permanent deformation

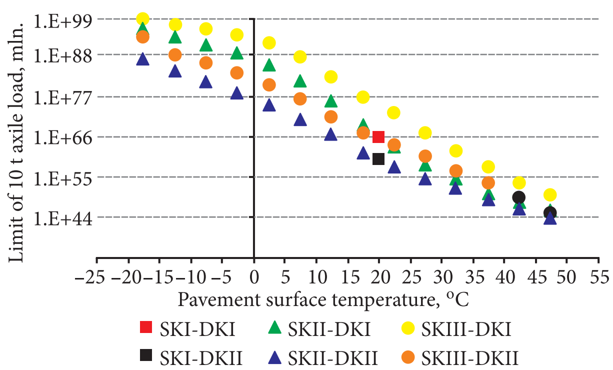

Fig. 8. The limit according to subgrade layer resistance to permanent deformation

in comparison to similar new alternative (DK I) pavement. The analysis of structural resistance to permanent deformation showed that at surface temperature higher than 35 °C the load influence on unbound base layer will be equally damaging irrespectively to pavement age or loading type.

The flexible pavement design based on experiment test data of visco-elastic behaviour of asphalt in aged pavement indicates the resistance to fatigue (limit load) decline comparing to the alternative new pavement structure. Also, the analysis of pavement response and performance showed that visco-elastic properties influence on responses decrease with operation time. The aging of bitumen and fatigue of asphalt cause higher horizontal strains into aged pavement structure.

The pavement design analysis showed that new and aged pavement structure can be optimized considering three main performance criteria by evaluating the responses under conditions of pavement in service.

5.3. Pavement structure optimisation based on performance criteria and economical evaluation

According to previous observations, the pavement structure (class III) layer thickness was optimised considering material mechanical properties and pavement

Table 4. The comparison criteria of pavement structures and design for economical evaluation

Phase	Criteria	Analysis (comparison) of variants				
		1.	2.	3.1	3.2	4.
Pavement design expenditure comparison	Pavement layers material test needs	Theoretical	AC modulus (at 20 °C) AC fatigue	AC modulus (4–34 or °C) AC fatigue	AC volumetric Binder DSR AC fatigue	AC volumetric Binder DSR AC fatigue
	Time for analysis and design	3 h	5 h	24 h	14 h	40 h
	Need of special calculation software	None	MnLayer or similar	MnLayer or similar	MnLayer or similar	ViscoRoute or similar
	Qualification level/ Experience of pavement design	Bachelor	Master/ 2 and more	Master/ 3 and more	PhD/ 5 and more	PhD/ 5 and more
	Pavement design expenses, EUR/h	70.08	122.64	140.16	262.79	262.79
Layer thicknesses and materials of pavement structure comparison	Pavement structures class III:					
	AC wearing	4 cm	3 cm	3 cm	3 cm	3 cm
	AC binder	4 cm	4 cm	4 cm	4 cm	4 cm
	AC base	10 cm	8 cm	7 cm	7 cm	6 cm
	Aggregate base	20 cm	20 cm	20 cm	20 cm	20 cm
	Frost resistant layer	47 cm	47 cm	47 cm	47 cm	47 cm
	Subgrade	–	Improved	Improved	Improved	Improved
	Preparation of subgrade soil	E_{V_2} = 45 MPa	Improved with gravel (fr.0/16) E_{V_2} = 120 MPa			
	Frost resistant layer	Sand (fr. 0/11) E_{V_2} = 100 MPa	Sand (fr. 0/11) E_{V_2} = 120 MPa			
	Aggregate base	Crashed dolomite (fr. 0/56) E_{V_2} = 150 MPa				
	Asphalt mix (bitumen) for base layer	AC 32 PN (50/70)	AC 32 PS (50/70)	AC 32 PS (50/70)	AC 32 PS (50/70)	AC 32 PS (50/70)
	Asphalt mix (bitumen) for binder layer	AC 16 AN (50/70)	AC 16 AS (PMB 45/80-55)	AC 16 AS (PMB 45/80-55)	AC 16 AS (PMB 45/80-55)	AC 16 AS (PMB 45/80-55)
	Asphalt mix (bitumen) for wearing layer	AC 11 VN (50/70)	SMA 11 S (PMB 45/80-55)	SMA 11 S (PMB 45/80-55)	SMA 11 S (PMB 45/80-55)	SMA 11 S (PMB 45/80-55)
Pavement construction expenditure comparison	Pavement structures paving costs, EUR/m^2	97.54	92.68	89.76	89.76	86.83
	Limit number of 10 t axle load, mln.: Resistance to fatigue of asphalt base (Σ_{Minner})	2.74 (1.09)	3.99 (0.75)	14.51 (0.77)	14.51 (0.77)	16.28 (0.48)
	Limit number of 10 t axle load, mln.: Resistance to permanent deformation of aggregate base	1.86E+08 (0.0001)	8957 (<0.001)	1.61E+10 (3.29)	1.61E+10 (3.29)	1.99E+09 (1.91)
	Limit number of 10 t axle load, mln.: Resistance to permanent deformation of subgrade	1.33E+38 (<0.001)	5.85E+66 (<0.001)	31.79E+79 (<0.001)	31.79E+79 (<0.001)	1.05E+75 (<0.001)

performance. The economical evaluation was conducted analysing pavement design and material tests costs and pavement construction costs. Five pavement structures (matching limit number of 10 t axle load of class III) were design for economical evaluation under (Table 4):

1. standard pavement design catalogue *KPT SDK 07* and material properties from theory;

2. standard pavement design catalogue *KPT SDK 07* and experimentally tested materials properties at one temperature (T = 20 °C);

3. advanced pavement design considering pavement elastic response and by experiment tested materials properties under 14 temperature intervals (T = (–17.5 °C, … 47.5 °C)):

3.1. asphalt mixture mechanical properties determined performing test in the laboratory;

3.2. asphalt mixture mechanical properties predicted applying Hirsch model;

4. advanced pavement design considering pavement visco-elastic response and by experiment tested materials properties under 14 temperature intervals (T = (–17.5 °C, … 47.5 °C)) when asphalt mixture mechanical properties predicted applying Hirsch model.

Pavement structure response (stress and strain) calculated from 10 t axial load and pavement performance subjected with asphalt base layer resistance to fatigue, aggregate base layer resistance to permanent deformation, and subgrade resistance to permanent deformation analysed. Table 4 presents the comparison criteria of pavement structures and design for economical evaluation.

Considering differently designed pavement structure the evaluation of layer material properties enables to present 8% less expensive pavement with the same performance as standard class III. Figure 9 shows the costs comparison of pavement construction and pavement design.

6. Conclusions

Summarizing analytical and experimental results, the following conclusions are formulated:

1. Literature review has shown the absence of universal tests and analytical methods for visco-elastic asphalt properties calculation, pavement response estimation and pavement performance prediction. In practice empirical methods are mainly used for pavement modelling and design.

2. Pavement design methods, based on analytical and empirical analysis, allow combine elastic and visco-elastic properties of materials and pavement performance prediction under certain temperatures and loading conditions simulating more realistic pavement operation through design life.

3. According to the pavement response and performance analysis was determined that the visco-elastic properties of asphalt mixture influence on responses decrease increasing pavement age. The higher horizontal strains were estimated for aged pavement and were caused of bitumen ageing and asphalt fatigue.

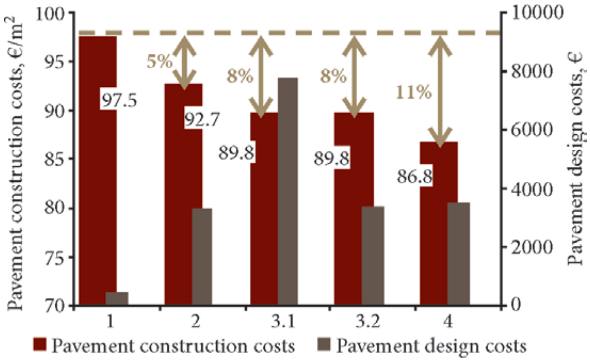

Fig. 9. The costs comparison of pavement construction and pavement design

4. Evaluation of visco-elastic asphalt properties allows design of optimized pavement structure with the same performance as standard one. Cost benefit analysis showed price reduction of optimized class III pavement structure by 8%.

5. It is recommended to use advanced pavement design procedure with evaluation of visco-elastic asphalt properties in cases where the Design Equivalent Axle Load A is over 3.0 million. In such cases, asphalt visco-elastic behaviour under different load and temperature conditions should be evaluated as the main parameter.

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