ISSN 1392-3730
JOURNAL OF CIVIL ENGINEERING AND MANAGEMENT
http:/www.vtu.lt/english/editions
2002, Vol VIII, No 3, 214–220

ADAPTATION OF PAVEMENT DETERIORATION MODELS TO LITHUANIAN AUTOMOBILE ROADS

Aivaras Braga[1], **Donatas Čygas**[2]

Dept of Roads, Vilnius Gediminas Technical University, Saulėtekio al. 11, LT-2040 Vilnius, Lithuania.
[1]*E-mail: braga@centras.lt* [2] *E-mail: dcyg@ap.vtu.lt*

Received 25 Apr 2001; accepted 12 June 2002

Abstract. The article presents three-system modelling of road pavement deterioration used in Lithuania: HDM-III (Highway Design and Maintenance Standards Model), HDM-4 (Highway Development and Management System) and DAVASEMA – Lithuanian pavement management system developed by Lithuanian Road Administration and based on HDM-III models. Using research data gathered in four years of the programme the authors analyse possibilities of adapting the pavement deterioration models to Lithuanian conditions. The article describes suggested procedures for calculating of calibration coefficients for the pavement deterioration models of the highest importance: road roughness component incremental model, structural cracking initiation model and structural cracking progression model.

Keywords: road pavement distresses, deterioration model, calibration coefficients.

1. Introduction

Planning of road maintenance and development activities, setting priorities for construction, reconstruction and repair of automobile roads, preparing economic evaluation and project justification are inevitably connected with the need to forecast road pavement condition. For this purpose the pavement deterioration models are used, which are often integrated into more complex computing systems [1]. These are pavement management systems and other highway development and management tools. The currently used Lithuanian Pavement Management System DAVASEMA, developed by the Lithuanian Road Administration (LRA), is based on pavement deterioration and road user effect models suggested by the international HDM-III (Highway Design and Maintenance Standards Model) [2, 3]. Lithuania still uses the HDM-III software for economic calculations of road projects. In 2000, the Lithuanian Transport and Road Research Institute (TRRI) purchased the new software of Highway Development and Management Model HDM-4, which will be used in the near future.

The first HDM systems were developed by the initiative of the World Bank in 1971-1981 and based on long-term research data collected in the countries of South America and Asia [4]. At present the systems are widely used by different consulting companies, financial and governmental institutions to investigate the economic consequences of investments in the road infrastructure. The HDM models are universal, easy to describe by mathematical equations, allowing to modify them and to adapt to local conditions [5].

Each country has unique local conditions, as they include climatic, technological, operational and traffic conditions, as well as standards and measuring units; therefore, adaptation and calibration of pavement deterioration models to local conditions are indispensable [1, 6].

In order to adapt pavement deterioration models to Lithuanian conditions and to determine a maximum accuracy of pavement distress models, it is necessary to conduct long-term (five-year or longer) observations of pavement condition, to accumulate and analyse observation data, to identify and to gradually calibrate mathematical relationships and models describing pavement deterioration. For this purpose, in 1997 the TRRI started a long-term Pavement Performance Research Programme. The research data collected in four years of the programme gave the opportunity for the authors of this article to make certain conclusions on pavement performance modelling possibilities in Lithuania.

2. Modelling pavement distresses

The asphalt concrete pavement as well as other bituminous pavement distresses, used in HDM-III, HDM-4 and DAVASEMA systems, are given in Table 1 (marked by "+"). All of them are closely related and determining the main unit of pavement condition, evaluation-pavement roughness [7], measured by the International Roughness Index (IRI) scale. Interaction between different pavement distresses is shown in Fig 1.

Table 1. Models of pavement distresses [2, 3]

Pavement distress	HDM-III	HDM-4	DAVA-SEMA
Structural cracking	+	+	+
Thermal cracking		+	+
Reflection cracking		+	
Ravelling	+	+	
Delamination		+	
Potholing	+	+	
Edge break		+	
Rutting	+	+	+
Pavement crossfall		+	
Roughness (IRI)	+	+	+

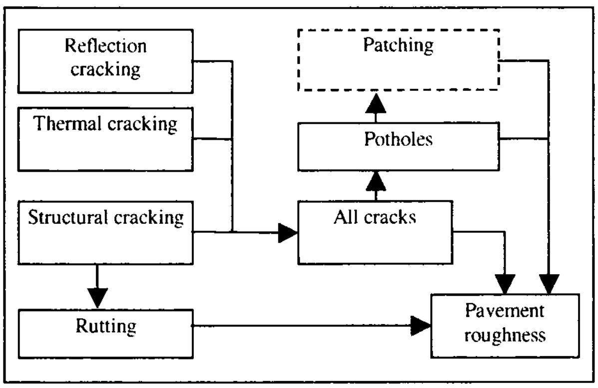

Fig 1. Interaction between pavement distresses [2, 3]

2.1. Sensitivity classes of pavement parameters

When adapting various pavement deterioration models it is very important to be aware of the impact of each pavement parameter on a final outcome, ie to know the level of sensitivity. As a rule, the final results of calculations, using the pavement management systems is an economic factor, which reflects economic benefits, generated by the implementation of a study project (eg Internal Rate of Return, IRR, or Net Present Value, NPV, of the project).

Sensitivity analysis of pavement parameters and their models was conducted by the developers of HDM-4 themselves. Sensitivity is quantified by the so-called *impact elasticity,* the ratio of change in the calculation results to the input parameter change, when other parameters are constant [1].

Four classes of model sensitivity have been established, according to the function of the impact elasticity [1]. The higher the elasticity, the higher is sensitivity class (Table 2).

Table 2. Sensitivity classes according to HDM-4 [1]

Impact on the final result	Sensitivity class	Impact elasticity
High	S-I	> 0,50
Moderate	S-II	0,20 – 0,50
Low	S-III	0,05 – 0,20
Negligible	S-IV	< 0,05

Table 3 gives the sensitivity classes of the main pavement parameters and models according to their impact elasticity. Calculation results, mostly impacted by those parameters, are marked by "+".

Before using the HDM systems under Lithuanian conditions, it is necessary to adapt their pavement deterioration models, having high or moderate impact on the final outcome (impact elasticity value > 0,2). The models of lower sensitivity do not require special adaptation, since they are likely to have only a slight impact on the accuracy of calculation results.

According to Table 3, the major pavement deterioration models are:

- cracking initiation model,
- cracking progression model,
- pavement roughness model.

Table 3. Sensitivity classes of pavement parameters and models according to HDM-4 [1]

Sensitivity class	Impact elasticity	Pavement parameters and deterioration models	Results mostly impacted		
			Predicted pavement roughness	Predicted pavement distresses	Economic evaluation factors
S-I	> 0,50	Pavement strength	+	+	+
		Traffic volume			+
		Pavement roughness	+		+
S-II	0,20 – 0,50	Traffic volume	+	+	+
		Pavement age		+	+
		All cracking area		+	+
		Pavement roughness model (environmental component)	+		+
		Cracking initiation model	+	+	+
		Cracking progression model		+	
S-III	0,05 – 0,20	Potholing area	+	+	
		Average rut depth	+		
		Rutting progression model	+		
		Pavement roughness model (other components)	+		+
S-IV	< 0,05	Ravelling area		+	
		Ravelling model		+	

2.2. Structural cracking initiation and progression models

Structural or fatigue cracking is the most widely spread type of distresses on asphalt concrete pavements, and it has the largest effect on pavement roughness. Structural cracks are generated by traffic loads and insufficient road pavement strength [7, 8]. It is assumed that the most common type of structural cracking is crocodile and longitudinal wheelpath cracking [2, 9].

In HDM systems, structural cracking is modelled as having two distinct phases: the initiation phase and the progression phase.

Structural cracking initiation model is an exponential relationship from the annual traffic volume per traffic lane and a square of pavement strength ratio (here and further in this article the original markings, suggested by the authors of the models, are used) [2, 5, 10, 11]:

$$ICX = a0 \cdot \exp\left(a1 \frac{YE4}{SNC^2}\right); \quad (1)$$

ICX is the time from the pavement surfacing to structural cracking initiation, in years (it is assumed that the initiation of structural cracking is a moment in time when 0,5% of pavement area is cracked); *YE*4 – average annual traffic volume per traffic lane expressed in millions of Equivalent Standard Axles (ESA) with a standard load of 8,2 t, *SNC* – road pavement strength expressed by the modified structural number SNC; $a\,0$, $a1$ – coefficients, depending on the type of pavement and foundation.

Under Lithuanian conditions the recommended coefficient values are [2]:

for asphalt concrete pavements: $a\ 0$=8,61; $a1$= -24,4;
for other bituminous pavements: $a\ 0$=13,2; $a1$= -20,7.

Progression of structural cracking is defined by a structural cracking annual increment model [2-5]:

$$CRX_t = Kcp\left\{50(1-Z) + Z\begin{bmatrix} Z \cdot a\,0 \cdot a1 \cdot NEci + \\ + Z\,0{,}5^{a1} + (1-Z)^{a1} \end{bmatrix}^{\frac{1}{a1}}\right\}. \quad (2)$$

where *Kcp* is model calibration coefficient; CRX_t – incremental area of indexed cracking at time t, in %; the indexed structural cracking area *ACX* is equal to the sum of the total area of all cracking *ACA* and the total area of wide cracking (>3 mm wide) *ACW*, multiplied by a crack width designator: *ACX*=0,62 *ACA* + 0,39 *ACW*; z=1, if $TCI < t_{50}$; otherwise z=-1; $a\,0$, $a1$ – coefficients; *TCI* – time since cracking initiation, in years; *TCI*=*AGE*2-*ICX*; *AGE*2 – pavement age, in years; *ICX* – structural cracking initiation period, in years; *NEci* – the cumulative traffic per traffic lane since the initiation of cracking, in millions ESA; t_{50} – time to 50% of area is cracked: $t50 = 50^{a1} - 0{,}5^{a1}/a\,0\ a1$.

Under Lithuanian conditions the recommended coefficient values for modelling structural cracking progression are [2]:

for asphalt concrete pavement: $a\ 0 = 3330\ SNC^{-4,25}$; $a1$=0,25;
for other bituminous pavements: $a\ 0 = 1530\ SNC^{-2,51}$; $a1$=0,41.

The relationship becomes more simple, when structural cracking progression is modelled before the moment when 50% of total pavement area is cracked (not likely to occur in practice), ie if $TCI < t_{50}$, and z=1. Then

$$CRX_t = Kcp\left[a0 \cdot a1 \cdot NEci + 0{,}5^{a1}\right]^{\frac{1}{a1}}. \quad (3)$$

After introduction of coefficient values for a 0 and a1 (for asphalt concrete pavements):

$$CRX_t = Kcp\left[3330 \cdot 0{,}25 \frac{NEci}{SNC^{4.25}} + 0{,}5^{0.25}\right]. \quad (4)$$

2.3. The component incremental roughness model

In the HDM-4 system, the change in road pavement roughness in a certain period of time is expressed by the component incremental pavement roughness model [3; 4]:

$$\Delta IRI = k0\begin{pmatrix} k1 \cdot \Delta IRIs + k2 \cdot \Delta IRIc + k3 \cdot \Delta IRIv + \\ + k4 \cdot \Delta IRIr + k5 \cdot \Delta IRIt + k6 \cdot \Delta IRId + \\ + k7 \cdot \Delta IRIh + k8 \cdot \Delta IRIe \end{pmatrix}, \quad (5)$$

where ΔIRI is total increment in IRI during a study period, m/km; $\Delta IRIs$ – the structural component of the IRI increment, $\Delta IRIc$ – the cracking component of the IRI increment; $\Delta IRIv$ – the ravelling component of the IRI increment; $\Delta IRIr$ – the rutting component of the IRI increment; $\Delta IRIt$ – the potholing component of the IRI increment; $\Delta IRId$ – the delamination component of the IRI increment; $\Delta IRIh$ – the patching component of the IRI increment; $\Delta IRIe$ – the environmental component of the IRI increment; $k0 \ldots k8$ – calibration coefficients (defaults = 1,0).

$\Delta IRIs$ – **the structural component** – describes the effect of reduction in pavement strength on the increment in pavement roughness:

$$\Delta IRIs = 134 e^{mAGE2}(1 + SNCK)^{-5} YE4\,, \quad (6)$$

where $\Delta IRIs$ is the structural increment of pavement roughness, m/km; m – an environmental coefficient; *AGE*2 – the years since the last resurfacing; *YE*4 – average annual traffic volume in million ESA per lane; *SNCK* – the modified structural number, reduced for the effect of cracking in a study period. If SNC is derived from field measurements at the end of ΔIRI determination period, there is no need for its recalculation, since *SNCK* = *SNC* [4].

$\Delta IRIc$ – **cracking component** – describes the effect of cracking on pavement roughness increment:

$$\Delta IRIc = 0{,}0066 \cdot \Delta ACA\,, \quad (7)$$

ΔACA – the increment in all cracking area in a study period, %.

$\Delta IRIv$ – **ravelling component** – describes the effect of ravelling on pavement roughness. Due to the fact that it is attributed to the lowest sensitivity class S-IV, it has not been studied when adapting the model.

$\Delta IRIr$ – **rutting component** – describes the effect of rutting on pavement roughness, when the change in rut depth is calculated as the increment in standard deviation of rut depth. The component is attributed to the sensitivity class III, therefore, it has not been studied when adapting the model.

$\Delta IRIt$ – **potholing component** – describes the effect of potholes on pavement roughness. Based on the strategy of automobile road use the occurrence of potholes on main and national roads is not allowed, thus, this component has not been studied when adapting the model.

$\Delta IRId$ – **pavement delamination component** – describes the effect of asphalt concrete pavement delamination area on pavement roughness. Delamination is the distress of thin overlays (mostly after surface treatment) in separate areas. Since the road sections with surface treatment applications had not been measured, the component has not been studied when adapting the model.

$\Delta IRIh$ – **patching component** – describes the effect of the reduced area of potholes on pavement roughness. Similar to a potholing component it has not been studied when adapting pavement roughness model.

Environmental component describes the environmental effect on the increment in pavement roughness:

$$\Delta IRIe = m\ IRI\ \Delta T; \qquad (8)$$

where m is environmental coefficient; IRI – pavement roughness at the beginning of the study period, m/km; ΔT – the length of a study period (a year or a fraction of a year).

For the initial adaptation of pavement roughness model it is necessary to determine the environmental coefficient m. In the second stage, for achieving a higher accuracy, the effect of the remaining factors on roughness increment should be studied and the coefficients $k0...k8$ should be determined.

3. Experimental studies

Based on the above chapters, the authors have come to a conclusion that for a comprehensive adaptation of pavement deterioration models for Lithuanian conditions the following data should be obtained or otherwise collected on:

- pavement strength expressed in the modified structural number SNC, which is recommended to be calculated based on the resilient pavement deflections, measured by a falling weight deflectometer (FWD) [1];
- road pavement roughness, measured in IRI units;
- traffic volume expressed in a number of equivalent axles ESA with a standard load of 8,2 t;
- the age of pavement surfacing;
- the cracking area for all types of cracks (structural, thermal and reflection);

In the period 1997–2000 the specialists of TRRI and the authors of this article carried out special investigations of 35 selected test sections on main and national Lithuanian roads. The length of each test section – 300 m, type of pavement – asphalt concrete on a granular base. Other characteristics (thickness of the structural layers, strength, age, traffic loading and geographical position) differed from section to section.

Pavement deflections on each study section were measured twice a year in a May–October period. Measurements were taken in two cross-sections – 50 m from the beginning and the end of a section, on the right wheel path of each traffic lane in a driving direction. Measurements were carried out by a falling weight deflectometer FWD Dynatest 8000. This is a standard pavement deflection-measuring device, used in many countries of the world. A number of worldwide methods have been developed to compute the modified structural number SNC from FWD deflection testing results. Since SNC is a relative dimensionless value, when using different methods, different values can be obtained. Reference [4] describes seven methods to determine the SNC, the correlation coefficients of which vary from 0,920 to 0,453. It is also recommended there to carry out pavement performance evaluation and pavement deterioration modelling by using Jameson's method. Another method, developed by the American Association of State Highway and Transportation Officials (AASHTO), is used by the TRRI to determine the need for pavement strengthening [6]. If compared to other methods, the advantage of the above-mentioned SNC estimation methods is that they do not use structural pavement layer thicknesses and material properties. It is enough to know the total thickness of pavement structure and the thickness of asphalt concrete layers, thus, the accidental errors could be avoided when determining thicknesses of pavement structural layers. Since the SNC values, calculated by these methods for the test road sections, in certain cases vary up to 20% and their correlation is 0,776 [4], the possibilities for modelling pavement deterioration parameters are studied separately for each SNC estimation method.

Pavement roughness was measured with a vehicle-mounted laser profilometer DYNATEST 5051 RSP, corresponding to the international IRI measuring standard. Annually two measurements were taken (mostly in a summer season) and their average value was used for the estimation.

Since 1998 a visual inspection of pavement distresses has been carried out each year. The main objective of a visual inspection is to observe pavement cracks, to register their initiation moment and further progression. The distresses identified were registered in special books and since 2000 photo pictures have been taken too. A visual inspection book describes the category and type of distress, its extent, severity and location, according to "Methods for identifying asphalt concrete distresses" [9].

Data on the average annual traffic volume and average annual ESA were obtained from the reports on traffic counts on main and national Lithuanian roads, carried out by the TRRI, or from the Lithuanian Road Database according to the results from traffic counting stations in a relative year.

Construction year of the pavement structure or surfacing layers was obtained from the Lithuanian Road Database.

4. Main investigation results

4.1. The effect of pavement strength on roughness

Based on the investigation results, the effect of SNC calculated by different methods on the increment of pavement roughness was determined. For this purpose equation (5) was rewritten, taking into consideration only those components, which have a certain effect on the roughness increment of the test sections in a study period:

$$\Delta IRI = \Delta IRIs + \Delta IRIc + \Delta IRIe. \tag{9}$$

The equation can be expressed as (6; 7; 8):

$$\Delta IRIs = 134e^{mAGE2}(1+SNCK)^{-5}YE4 + 0{,}0066 \cdot \Delta ACA + m \cdot IRI \cdot \Delta T. \tag{10}$$

In this equation, parameter *SNC*, re-calculated into *SNCK*, and the annual traffic loading *YE*4, expressed in ESA, are the main variables of a structural component $\Delta IRIs$. The effect of cracking component on pavement roughness increment on test sections is minimal (for $\Delta IRIc$ to be >0,05, the annual progression of cracking area ΔACA should be > 7,6%, however, only on one test section a total cracking area comes to 10%). According to HDM-4 [1], the SNC and a cumulative traffic loading NEci have the effect on the progression of all types of cracks, thus, on the component $\Delta IRIc$, too. Therefore it can be asserted that road pavement strength and traffic loading are the main parameters determining pavement roughness increment in a certain period of time. For the purpose of conviction the relationship was derived between the annual incremental change in pavement roughness ΔIRI per 1 million ESA (ΔIRI and *YE*4 relation, based on 1997–2000 measurement data) and pavement strength, expressed in an average SNC of a test section at the end of one-year period (Fig 2). Also, the relationship curves were derived according to a mathematical equation:

$$Y = aX^b\ ; \tag{11}$$

assuming that $Y=\Delta IRI/YE4$; m/km million ESA; X – average SNC of a test section at the end of one year period; b=5, assumed according to (2, 10) equations; a – average coefficient assumed for all n test points:

$$a = \frac{\left(\sum_{n=1}^{n} SNC_i^5 \dfrac{\Delta IRI_i}{YE4_i}\right)}{n}. \tag{12}$$

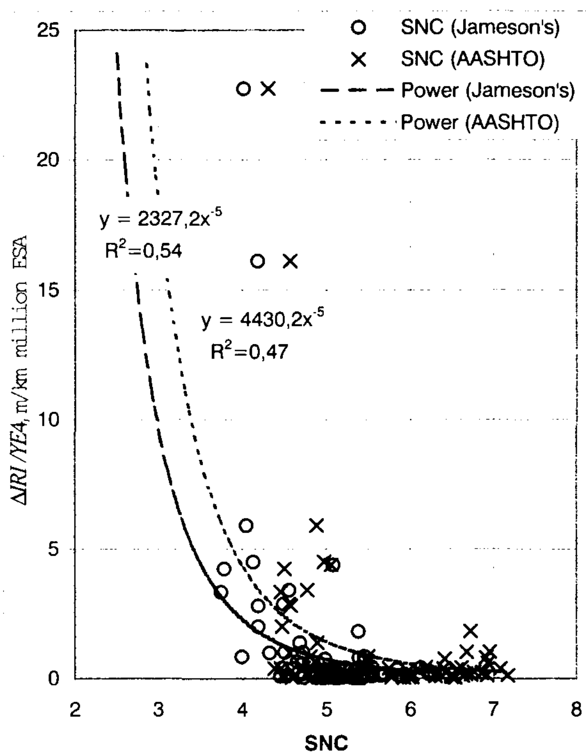

Fig 2. Relation between road roughness increment and SNC, calculated by Jameson's or AASHTO method obtained from the data on test road sections

The results showed that, when SNC was calculated by Jameson's method, a = 2327,2; by AASHTO method – a = 4430,2.

4.2. Evaluation of environmental effect

Environmental coefficient m is the main parameter in valuing environmental effect on pavement performance (sensitivity class S-II). To determine this coefficient the obtained road roughness increment is compared to the calculated one, using a mathematical roughness model (10). Calculations were made by an approach method, ie by comparing a real progression of pavement roughness in a certain time with that of the calculated.

Environmental coefficient m was calculated for one-year period (ΔT=1), starting from the middle of one year to the middle of another. At the same time ΔIRI was estimated as a difference between the values of pavement roughness of a study year, and *SNCK* – as an average actual pavement strength of a final year, expressed by SNC and calculated by Jameson's or AASHTO method. Thus, each individual road section is described by two coefficients m (for different SNC calculation methods) in each study year. A final m value was determined as the average of all values obtained.

Investigations showed that the average three-year environmental coefficient m under Lithuanian conditions is:

a) when SNCK is calculated by Jameson's method – 0,071;
b) when SNCK is calculated by AASHTO method – 0,072.

For the comparison, in HDM-4 the recommended value of environmental coefficient m for Lithuanian conditions is 0,065 [2; 4].

4.3. Adapting structural cracking models

Based on equation (1), the period of structural cracking initiation depends on the annual traffic volume per one traffic lane YE4, expressed by a ratio of ESA and SNC square. The original HDM-4 relationship for asphalt concrete pavements with the recommended coefficients for Lithuanian conditions is shown in Fig 3.

The figure also gives the determined values of structural cracking initiation on test road sections (for both SNC calculation methods – Jameson's and AASHTO). Knowing that the equation of cracking initiation period is exponential, the exponential regression curves were derived across the groups of points. The graph reflects their mathematical expression.

When the exponential relationship between the structural cracking initiation period and the ratio $YE4/SNC^2$ is derived, it is possible to determine the coefficients $a\,0$ and $a1$. The relationships obtained are still not very accurate (correlation – 0,15–0,19) due to a low number of points (and the lack of data since the structural cracking have started their progression only on nine test road sections).

Coefficients $a\,0$ and $a1$ can be determined from a mathematical expression of exponential curves derived:

$$Y = A \cdot e^{BX} \; ; \qquad (13)$$

Y and X – variables, $Y=ICX$; $X=YE4/SNC^2$; e=2,7182; A, B – coefficients, $A=a0$; $B=a1$.

If SNC is calculated by Jameson's method, the determined coefficient values are: $a\,0$=4,518; $a1$= -26,65.

If SNC is calculated by AASHTO method, the determined coefficient values are: $a\,0$ = 4,672; $a1$= -44,77.

Structural cracking progression depends on the annual traffic volume per one traffic lane YE4 in ESA, and SNC powered by 4,25 ratio (see equation (4). The original HDM-4 relationship and determined values for the test road sections are given in Fig 4.

The coefficient Kcp is calculated as a ratio between an actual area of structural cracking, registered on a test section, at time t and the calculated one by using equation (8).

The following average coefficients for adaptation of structural cracking progression model to Lithuanian conditions were obtained:

- In case if SNC is calculated by Jameson's method– Kcp = 1,304 (correlation coefficient R^2 = 0,67);
- In case if SNC is calculated by AASHTO method – Kcp = 2,277 (R^2 = 0,87).

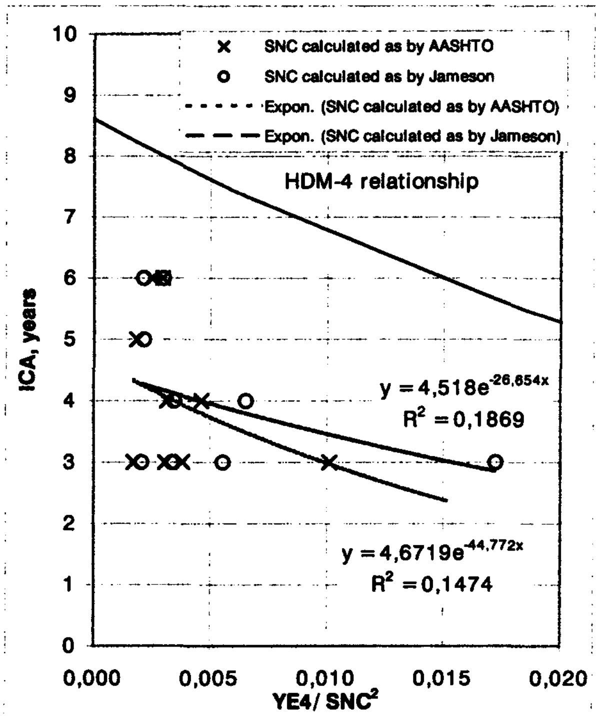

Fig 3. Relation between a structural cracking initiation period ICA and $YE4$ / SNC square ratio

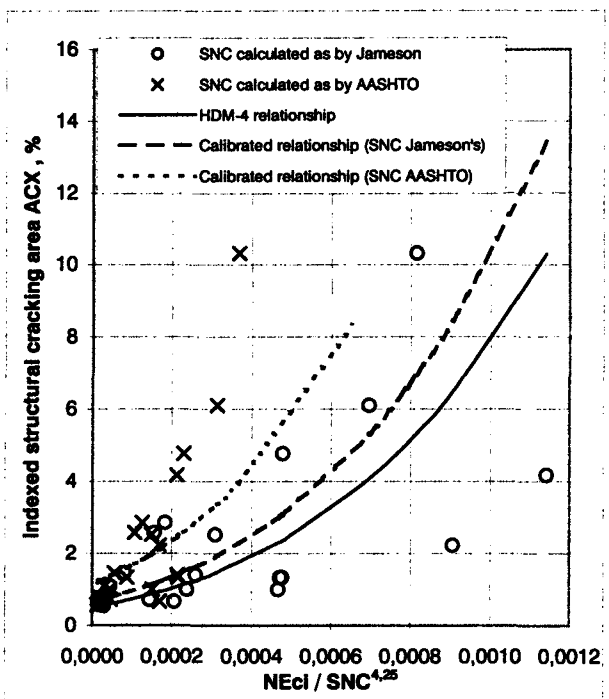

Fig 4. Relation between structural cracking progression and $YE4$ / SNC powered by 4,25 ratio

5. Conclusions

1. The analysis of pavement roughness and pavement strength showed that pavement roughness is more rapidly progressing on the road sections where pavement condition is worse and traffic loading is higher (Fig 2). The effect of pavement strength, expressed by SNC, on this relation is appraised by power 5. Taking into consideration the graphs obtained and the equations of component parameters (6; 7; 8) of pavement regression model (5), the effect of SNC on this relation could be appraised by even higher power.

2. The selection of SNC calculation method has a very slight influence on the calibration of a component incremental pavement roughness model, but this is important for individual components of the model and for other pavement deterioration models. The analysis of pavement deterioration parameters and models did not allow to identify which of the methods is more accurate or at least more acceptable; therefore in further investigations we would suggest to rely on HDM-4 recommendations and to use Jameson's method for SNC calculations.

3. The derived average three-year environmental coefficient $m = 0,07$. It is slightly higher than that suggested by HDM-4 for Lithuanian conditions ($m = 0,065$), and, correspondingly, shows a higher environmental effect on the pavement roughness change.

4. Structural cracking progression was observed on nine sections from those 31 studied. Therefore, the derived relations of structural cracking initiation and progression are rather unreliable. A large influence of SNC calculation method on the derived calibration coefficients of the models should be mentioned. Thus, when modelling pavement deterioration, the calibration coefficients should be used, corresponding to the SNC calculation method selected.

5. To get reliable relations of asphalt concrete pavement deterioration parameters and to determine the accurate calibration coefficients of HDM-4 pavement deterioration models for Lithuanian conditions this kind of investigations should be continued for additional 3-5 years. Based on the results obtained, it is only possible to make an assumption that pavement distresses on Lithuanian roads are initiating earlier and progressing more rapidly than in the other countries with similar climatic conditions (or than it is predicted in HDM-III and HDM-4 models).

References

1. Bennett C. R., Paterson W. D. O. HDM-4. Volume 5. A Guide to Calibration and Adaptation. PIARC, 1999. 198 p.
2. Tebodin Consultants and Engineers O'Sullivan and Graham. Enhancement of the DAVASEMA Pavement Management System. Pavement Performance Modelling. Technical Assistance to Lithuanian Road Administration, 1996. 43 p.
3. TRRI. Enhancement of pavement management system. Development of road maintenance economic evaluation subsystem, etc. (Dangų valdymo sistemos tobulinimas. Kelių priežiūros darbų ekonominio pagrindimo posistemio dangų valdymo sistemoje sukūrimas ir t.t.). Kaunas: Transport and Road Research Institute, 1999. 51 p (in Lithuanian).
4. NDLI. Modelling Road Deterioration and Maintenance Effects in HDM-4. Vancouver: N.D.Lea International, 1995. 220 p.
5. Peterson W. D. O. Road Deterioration and Maintenance Effects. Models for Planning and Management. The Johns Hopkins University Press, Baltimore, 1987. 454 p.
6. Odoki J. B., Kerali H. G. R. HDM-4. Volume 4. Analytical Framework and Model Descriptions. PIARC, 1999. 634 p.
7. TRRI. Pavement distress identification manual (Asfaltbetonio dangų defektų nustatymo metodika). Kaunas: Transport and Road Research Institute, 1994. 45 p (in Lithuanian).
8. Petkevičius K., Sivilevičius H. Necessary characteristics and rational life duration of asphalt pavements and road pavement structures. *Transport* (Transportas), Vol 15, No 4, 2000, p 184-195 (in Lithuanian).
9. Puodžiukas V., Kamaitis Z. Strengthening of asphalt pavements and modeling of the strengthening influence on road roughness. *Civil Engineering* (Statyba), Vol 5, No 6, 1999, p 386-393 (in Lithuanian).
10. TRB (Transportation Research Board). Transportation research record No 1397. Pavement management systems. National Academy press. Washington, D.C. 1993. 119 p.
11. Per Ullidtz. Modelling Flexible Pavement Response and Performance. - Polyteknisk Forlag, Gylling, 1998. 205 p.