Assessment of the Remaining Life of Bituminous Layers in Road Pavements

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***Abstract* – In this paper, a mechanistic-empirical approach is presented for the assessment of bearing capacity condition of asphalt pavement layers by Falling Weight Deflectometer measurements and laboratory fatigue tests. The bearing capacity condition ratio was determined using past traffic data and the remaining fatigue life which was determined from multilayer pavement response model. The traffic growth rate was taken into account with finite arithmetic and geometric progressions. Fatigue resistance of layers' bituminous materials was obtained with indirect tensile fatigue tests. Deduct curve of condition scores was derived with Weibull distribution.**

***Keywords*–remaining life, fatigue, backcalculation, condition rating, analytical model**

1. Introduction

The evaluation of pavements' remaining service life (*RSL*) is the most important part of Pavement Management System (PMS) providing information for decision-making at network and project levels. For a pavement section, the *RSL* can be expressed in years between the evaluation year and the year when the pavement reaches a threshold value of the pre-determined service quality [1]. The remaining service life of existing pavements can be assessed with a functional or structural approach as well as with a combined functional-structural approach. For estimation of *RSL* with a functional approach, the failure of the pavement is determined using performance outputs (e.g. roughness, serviceability, safety, etc.).

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The structural failure concept applies structural capacity parameters (e.g. deflections, fatigue resistance, distress, etc.). The remaining structural life was determined from fatigue criterion and elastic layer principles [2]. The structural approach was found more suitable at project level for the design of rehabilitation works [3]. It was found that methods combining material test data as inputs for fatigue analysis and pavement performance evaluation could provide a more comprehensive solution [4].

For evaluation of the structural condition, the Structural Adequacy Index (*SAI*) was proposed using the maximum tolerable deflection (*MTD*) as a function of load repetitions [4].

A statistically based Structural Strength Index (*SSI*) was established corrected with traffic factor and rainfall factor [5]. The tensile strain at the bottom of asphalt layer, the vertical compressive strain at the top of subgrade, the rutting and cracking remaining life were determined by regression relationships derived from deflections and surface curvature index measured with Falling-Weight-Deflectometer (FWD) [5].

In a further research using FWD data directly, the structural index was defined as the ratio of existing structural number (SN_{eff}) and the required structural number (SN_{req}) of the pavement. The SN_{eff} depended on the pavement thickness and the Structural Index of the Pavement (*SIP*). The *SIP* value is the difference between the central deflection and the deflection measured at the offset of 1.5 times of the total pavement thickness [6].

Relationships were established between the structural number (*SN*) and the deflection bowl parameters of pavement with an asphalt base. It was found that with the application of the Base Layer Index (*BLI*), the Middle Layer Index (*MLI*), the Lower Layer Index (LLI) and the Area under pavement profile (*AUPP*) the calculation of *SN* can be improved. These parameters can be used for evaluation of the structural condition without any other information on layer thicknesses or materials [7].

An alternative multiple Structural Number (adjusted structural number, *SNP*), was proposed which can be created from mechanistically derived independent separate structural indexes (*SI*) of rutting, roughness, flexure, and shear stability [8].

A sigmoidal relationship was found between *RSL* and the FWD centre deflection which can be used to predict *RSL* at the network level [9]. In further research, it was determined that the structural number can be predicted from FWD centre deflection and pavement condition data for network-level [10].

For overall structural evaluation, the Strength Structural Index (*SSI*) can be determined without backcalculation with a regression equation using indicators of the upper layer and subgrade calculated from selected FWD deflections corrected with temperatures [11].

Certain structural evaluation methods compare measured deflection (or measured surface curvature index) with initial permissible deflection (or with initial permissible surface curvature index). The initial permissible deflection is usually calculated with empirical fatigue relationship depending on the design traffic [12]. For discrimination of different pavement behaviour, empirical fatigue equations for initial permissible deflection can be assigned to the separate pavement family (e.g. very flexible, flexible, semi-rigid) [12]. Stresses, strains, and deflections can be calculated using multilayer analytical models backcalculated from deflections measured with FWD. In these models, permissible criteria are often determined with empirical equations in general form regardless of pavement types or material parameters.

Several authors used probabilistic models for describing pavement condition as a function of time. In a review of PMS at the network level for prediction of the Pavement Condition Index (*PCI*) the Weibull distribution was applied [4]. Several probabilistic functions have been investigated for describing the hazard rate of the pavement. It was found that the failure hazard of the pavement condition can be modelled best by the Weibull distribution [3]. The remaining service life can be predicted with Kaplan-Meier survival curves approximated with Weibull distribution. This method is not able to show the difference between the life of a thin pavement with high traffic volume and a thick pavement with low traffic volume. [13].

The fatigue life of the pavement can be expressed with cumulative passes of Equivalent Standard Axle Load (ESAL) or with years. The end of the actual fatigue life can be defined when the relative area of fatigue cracking reaches the pre-defined very poor (undesirable) level. For example, this very poor level was defined as the amount of alligator cracking reaches 40% of the total pavement area [14]. Fatigue relationships were determined in a study of thin asphalt layers for a different amount of cracking (under 10% of cracking, and above 30% of cracking). In these fatigue equations, the number of load repetitions depends on the maximum tensile fatigue strain at the bottom of asphalt layer and the complex modulus of asphalt concrete [15].

In this paper, the remaining fatigue life as shown in Figure 1. is the time span between the evaluation year and the final year of the actual fatigue life. In this work it is assumed that:

– the initial year (year of opening the traffic) of the layer's fatigue life is known from the road data bank;
– the annual average daily traffic volumes of the commercial vehicles and buses between the initial year and the evaluation year are known;
– the function of the traffic growth rate is given or can be determined by regression analysis from past traffic data;
– factors for calculating the cumulative ESAL number (in one direction, in the wheel path of the design traffic lane) are known.

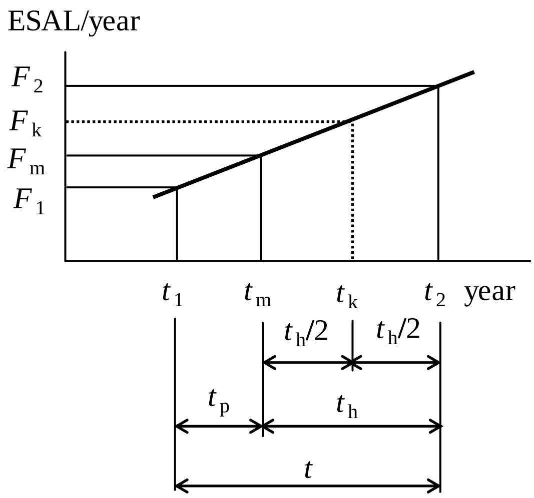

Figure 1. The remaining fatigue life in the case of linear traffic growth

Legend:
(t– the actual fatigue life of the layer, t_1–the initial year (year of opening the traffic), t_m–the evaluation year, t_p–period of time between the initial year and the evaluation year, t_h–the remaining fatigue life of the layer, t_k–the middle year in the remaining fatigue life period, t_2–the final year of the actual fatigue life, F_2–cumulative number of *ESAL* passes during the final year of the actual fatigue life, F_k–cumulative number of *ESAL* passes during the middle year of the remaining fatigue life, F_m–cumulative number of *ESAL* passes during the evaluation year, F_1–cumulative number of *ESAL* passes during the initial year.

The year of bearing capacity measurements and the year of fatigue resistance tests are regarded as the evaluation year. It is desirable that bearing capacity measurements and sampling of specimens from layers be made nearly at the same time, probably within a month. The determination of remaining fatigue life also depends on the forecasting method of the commercial traffic. In this work, two traffic growth models were considered. The traffic growth of commercial vehicles can be given with a linear function [16]. Most specifications apply the annual traffic growth rate in percent [17].

2. The remaining fatigue life

The linear traffic growth corresponds to a finite arithmetic progression. The series of the finite arithmetic progression corresponds to the cumulative number of *ESAL* passes over the remaining fatigue life period in one direction, F_h in the wheel path of the design traffic lane:

$$F_h = M \cdot t_h \cdot F_k, \tag{2.1}$$

where M is the reliability factor. The difference between the pavement's design period and the actual performance period can be taken into account with reliability factor derived from probability distribution theory [18]. Here, for second class rural major roads, M=1.56 is taken at 90% confidence level. For calculating F_h, traffic data from the evaluation year were used applying the slope λ of the traffic growth straight line (λ>0):

$$F_h = M \cdot t_h \cdot (F_m + \frac{t_h}{2} \cdot \lambda), \tag{2.2}$$

with

$$t_h = [(F_m^2 + 2 \cdot \lambda \cdot F_h / M)^{0.5} - F_m] / \lambda. \tag{2.3}$$

The value of λ can be determined from past traffic data by linear regression or by:

$$\lambda = (F_m - F_1) / (t_m - t_1). \tag{2.4.}$$

When the traffic yearly growth rate is given as a constant rate p (percent), the cumulative traffic over the design period is determined as the series of a finite geometric progression [19]. Using this approach the cumulative number of *ESAL* passes over the remaining life period can be obtained as

$$F_h = F_m \cdot \frac{(1+r)^{t_h} - 1}{r}, \tag{2.5}$$

with

$$r = p/100. \tag{2.6}$$

From 2.5 the remaining fatigue life in years is expressed as:

$$t_h = \frac{\log(1 + r \cdot F_h / M \cdot F_m)}{\log(1+r)}. \tag{2.7}$$

The working fatigue tensile strain value in the bituminous layer (mostly at the bottom of the layer) is determined by analysis of FWD measurement's data. After backcalculation of layers moduli, working stresses and strains can be calculated using the mechanical analysis of a multilayer pavement model. At a given confidence level S, the lower (ε_l), and upper (ε_u) levels as well as the confidence interval, $\Delta K(\varepsilon)$ of the fatigue tensile strain are

$$\varepsilon_l = \varepsilon_a - t_{S,n} \cdot \frac{s(\varepsilon)}{\sqrt{n}}, \tag{2.8}$$

$$\varepsilon_u = \varepsilon_a + t_{S,n} \cdot \frac{s(\varepsilon)}{\sqrt{n}}, \tag{2.9}$$

$$\Delta K(\varepsilon) = 2 \cdot t_{S,n} \cdot \frac{s(\varepsilon)}{\sqrt{n}}, \tag{2.10}$$

where ε_a is the mean value of the initial tensile working strain (microstrain), s(ε) is the corrected standard variation of the sample, n is the sample size, and $t_{S,n}$ is the value of Student's t–distribution. Here the Wöhler's fatigue law is used for describing fatigue behavior of bituminous materials:

$$\varepsilon = a \cdot N^{-b} \tag{2.11}$$

where ε (microstrain) is the maximum horizontal tensile strain in the bituminous layer, N is the number of load cycles to failure, a is a material parameter and b is the slope of the fatigue line. The values of a and b can be determined with laboratory fatigue tests. Combining fatigue relationship (2.11) with F_h the remaining fatigue life of the layer is

$$F_{ha} = SF \cdot (a/\varepsilon_a)^{1/b}, \tag{2.12}$$

$$F_{hl} = SF \cdot (a/\varepsilon_u)^{1/b}, \tag{2.13}$$

$$F_{hu} = SF \cdot (a/\varepsilon_l)^{1/b}, \tag{2.14}$$

where, F_{ha}, F_{hl}, F_{hu} are the mean value, the lower level and the upper level of the actual remaining fatigue life respectively. The shift factor SF allows to convert the laboratory fatigue life to the fatigue life of the layer in field conditions. Note that the F_{hl} corresponds to the upper strain level and F_{hu} corresponds to the lower strain level.

3. Bearing capacity rating

If the bearing capacity rating is made by observation of the cracked area then the rating score depends on the amount of alligator cracking. A relationship for the deduct curve is proposed below where the score value is determined on a five-level scale as

$$R = 1 + 4 \cdot \left[1 - \left(1 - \frac{CRA}{CRA_f} \right)^n \right], \qquad (3.1)$$

where R ($1 \le R \le 5$) is the actual score value, CRA (%), ($0 \le CRA \le CRA_f$) is the amount of cracking in percent of the total area, CRA_f (%) is the amount of cracking corresponding to the end of the actual fatigue life in percent of the total area. For general practice n=2.1-2.5 and CRA_f =35%–40% can be used. For evaluation of bearing capacity the mechanistic-empirical approach uses FWD measurements, multilayer analysis, and fatigue charasteristics of the layer's materials. The mean condition ratio CR_a, ($0 \le CR_a \le 1$) of the bearing capacity is

$$CR_a = F_p \,/\, (F_p + F_{ha}), \qquad (3.2)$$

F_p is the past traffic, equal to the number of cumulative *ESAL* passes between the initial year and the evaluation year. For the equation (3.3) of the deduct curve the Weibull distribution was applied. The cumulative value of this distribution function is

$$\mathrm{S}(CR_a; \eta, \beta) = 1 - e^{-(CR_a / \eta)^{\beta}}, \qquad (3.3)$$

where η=0,85 is the scale parameter and β=2,4 is the shape parameter. Score classes and the intervals of the mean condition ratio are given in Table 1. and demonstrated with deduct curve in Figure 2.

Table 1. Score classes of the bearing capacity condition

Description	Score	Mean condition ratio
Very good	1	0.000 – 0.050
Good	2	0.051 – 0.150
Fair	3	0.151 – 0.340
Poor	4	0.341 – 0.620
Very poor	5	0.621 – 1.000

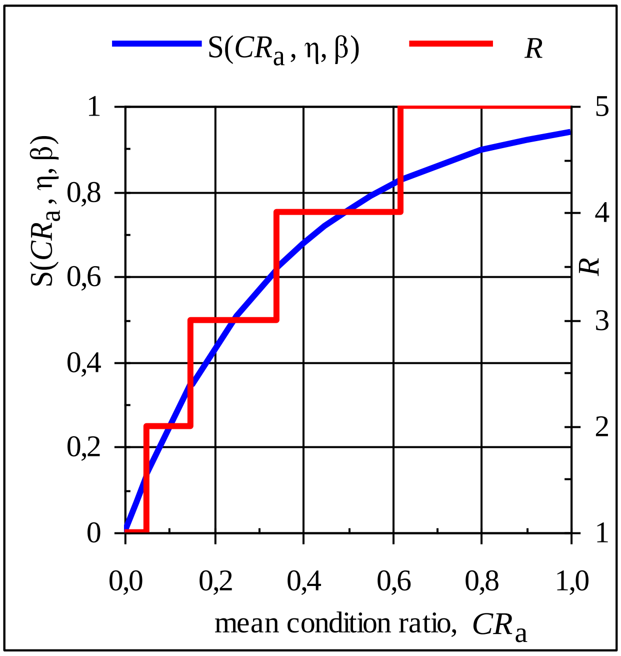

Figure 2. The deduct curve and score classes of the bearing capacity condition

4. Tests and analysis

The following tests were conducted on a selected homogeneous section of the second class two-lane major road No. 83 between chainage of 31250 m–33050 m:

– bearing capacity measurements with FWD, (KUAB);

– identification of homogeneous test sections with the method of Cumulative Sum of Differences (CuSum), [20];

– taking cores from the homogeneous test sections and identification of pavement layers;

– measuring layer thicknesses on cores; the pavement structure data are given in Table 2. where the first layer is a new asphalt concrete wearing course, and the underlying layers compose the existing pavement structure before the resurfacing;

Table 2. The pavement structure

Layer No.	Material	Thickness, mm
1	asphalt concrete	40
2	asphalt concrete	40
3	asphalt concrete	40
4	asphalt concrete	60
5	penetration macadam	160
6	crushed stone	200
	subgrade	

– backcalculation of layers' moduli;

– laboratory fatigue resistance tests on cylindrical test specimens cut from cores;

– calculation of stresses and strains in the pavement layers [21];

– determination of the actual remaining fatigue life of the bituminous layers.

Moduli of layers were backcalculated with BAKFAA code [22]. The backcalculated moduli of the first five asphaltic layers were corrected for temperature and loading time. The temperature correction was made to 20 °C with equation [23]:

$$\frac{E_{T1}}{E_{T2}} = \frac{2.6277 - 1.384 \cdot \log T_2}{2.6277 - 1.384 \cdot \log T_1} \qquad (4.1)$$

where E_{T1} is the backcalculated modulus at the temperature T_1 (°C) of the layer at the FWD measurement and E_{T2} is the corrected modulus at the reference temperature T_2 (20 °C). The correction of modulus from the FWD loading time (50 ms) to the traffic loading time was made with [24]:

$$E_{1TRAFFIC} = E_{1FWD} \cdot \left(\frac{t_{FWD}}{t_{TRAFFIC}} \right)^{0.0,9+0.006 \cdot T} \qquad (4.2)$$

where E_{1FWD} is the modulus at the FWD's loading time t_{FWD} while $E_{1TRAFFIC}$ is the modulus at the wheel loading time $t_{TRAFFIC}$ and T is the reference temperature (20 °C). For calculation of $t_{TRAFFIC}$ the Brown's formula was used where the loading time t (s) of the vehicle's wheel depends on the depth h (m) and the speed V (km/h) [25]:

$$\log(t) = -0.2 + 0.5 \cdot h - 0.94 \cdot \log(V) \qquad (4.3)$$

The laboratory fatigue tests were performed using Indirect Tensile Fatigue Test (ITFT) method according to BS DD ABF procedure [26]. This test method estimates the crack initiation on cylindrical specimens applying controlled vertical repeated force pulse with rise time of 124±4 ms and 1.5 s pulse repetition period. The maximum tensile stress and the maximum horizontal tensile strain at the centre of the specimen were calculated from test data according to applied test method [26].

Three asphalt concrete (AC) layers were identificated on cores, here named as first layer (top layer), second layer and third layer. Cores were cut by layers then geometrical dimensions and bulk specific gravities of the test specimes were determined. The ITFT test were conducted at the temperature of +20 °C which is the weighted annual fatigue temperature for asphalt pavements in Hungary. The fatigue equations were determined with linear regression analysis in form:

$$\varepsilon_{x,\max} = a \cdot N_f^{-b}. \qquad (4.4)$$

where $\varepsilon_{x,\max}$ is the maximum horitontal tensile strain at the centre of the specimen, N_f is the number of cycles to failure of the specimen, a is the material parameter and b is the slope of the fatigue life. The values of a and b as well as the correlation coefficient (R^2) are given in Table 3. and the fatigue lines of the tested materials are illustrated in Figures 3–5.

Table 3. Material constants and correlation coefficient of the AC layers derived from ITFT tests

Layer	a	b	R^2
1st layer	1095	0,1890	0,95
2nd layer	1713	0,2526	0,86
3rd layer	1247	0,2114	0,87

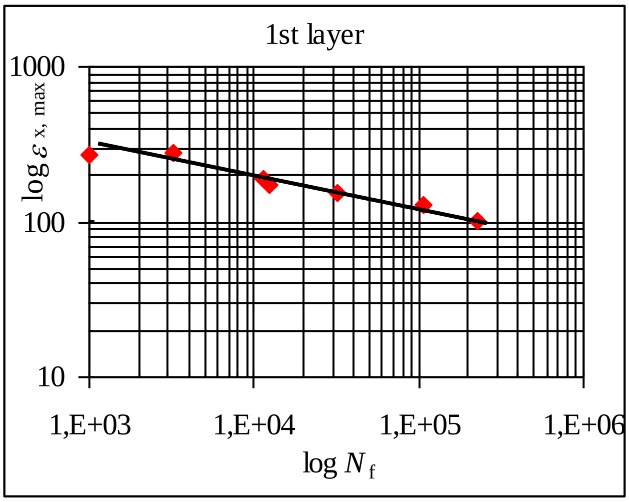

Figure 3. Fatigue line of the first layer material

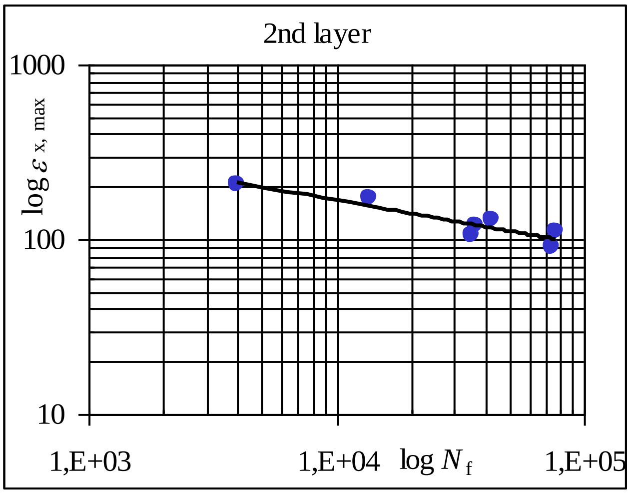

Figure 4. Fatigue line of the second layer material

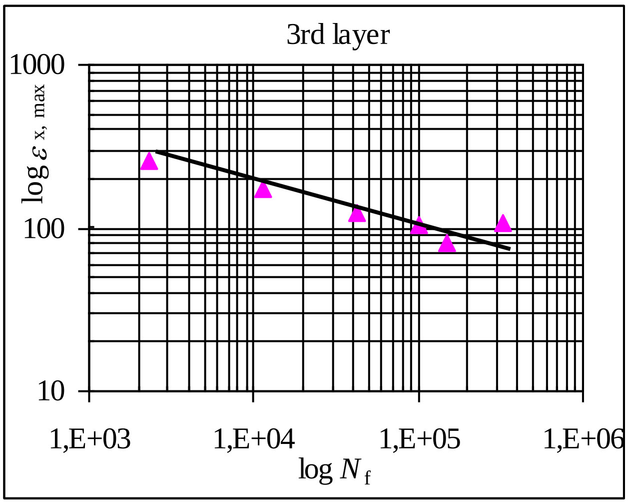

Figure 5. Fatigue line of the third layer material

A new asphalt concrete wearing course was laid on the existing pavement structure (the1st layer in Table 2. and Table 3.). The FWD measurements and fatigue tests were conducted after a year of new layer's construction. It reveals from Table 4. that the second layer is critical with shortest remaining life. Results of analysis of the first three layers are given in Table 4. The traffic data for calculation are: F_m=207632; λ=33201; F_1=174431; number of FWD measurement points=20.

Table 4. Remaining lifes of the AC layers

Layers	1st layer	2nd layer	3rd layer
ε_α (μm/m)	84	94	99
s(ε) (μm/m)	17	22	28
F_{ha} (ESAL)	8.20x10^6	0.99x10^6	1.64x10^6
t_{ha} (year)	12.6	2.6	3.9

Conclusion

A mechanistic-empirical procedure is given for the estimation of remaining fatigue life of the asphalt pavement layers at project level. A bearing capacity condition ratio was derived using traffic growth rate, fatigue resistance of layers' materials and the working tensile strains in the asphalt layers. The pavement response model shall correspond to the layer structure of the existing pavement. The combination of two or three layers into one layer leads to inefficient results. The confidence interval of the remaining fatigue life is influenced by the tensile strain in the asphalt layer. The remaining fatigue life is very sensitive for the standard deviation of the tensile strain. The interface parameter representing the bond between the pavement layers highly influences the backcalculated moduli. Due to this the bearing capacity condition ratio can be strongly effected through the calculated tensile strains. In further research, it is suitable to determine separate deduct curves by pavement families (very flexible, flexible and semi-rigid). The presented methodology for assessment of remaining fatigue life helps to select and customise pavement rehabilitation alternatives at project level.

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