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RESEARCH ARTICLE

Simple Overlay Design Method for Thick Asphalt Pavements Based on the Method of Equivalent Thicknesses

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Abstract

The focus on the construction of new roads will certainly face a shift to the widening, maintenance and strengthening of the existing roads and pavements. However, the Hungarian standard referring to asphalt pavement and overlay design is outdated in many ways, and to efficiently use national resources a proper analytical design must be urgently developed.

The paper introduces a method based on the linear elastic multi-layer theory which is suitable for overlay design of considering the actual condition the existing structure, actual material parameters and various technological possibilities, provided some boundary conditions – primarily true in cases of highways and heavy duty pavements – are met.

The method is capable to assess the mechanical properties of the existing layers and the overlay, as well as to consider excess performance achieved by technologies already commonly used – but neglected in the current design methods –in Hungary, such as various remix technologies, special additives and mixes or innovative structural solutions.

In light of the affordable compromise regarding accuracy the proposed method gives a straightforward option for overlay design in the everyday practice.

Keywords

Overlay design, indirect tensile fatigue, equivalent thicknesses

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1 Introduction

As a result of vigorous developments in the past decades the primary road network in Hungary is close to having an optimal density, shifting the focus from the construction of new roads to managing the existing roads and pavements, with a great emphasis on strengthening and overlays. Unfortunately, however, there was no substantial development done in the field of pavement and overlay design in Hungary since the nineties. Although advanced at the time, the current standard for design of asphalt pavements and overlays became outdated over the past 25 years, since its development – similarly to several other Hungarian technical regulations – did not follow the developments used in the everyday practice. The current method cannot consider the mechanical parameters achieved using technologies and materials developed in the last decades that are otherwise commonly used in Hungary today as e.g. the use of modified binders, high modulus asphalt mixes, new additives and performance modifiers, remixed layers, asphalt nets, or compromises when using local materials.

The use of the current standard is mandatory on state roads and, in lack of alternatives, it is exclusively used for design of local roads as well, leading to overdesign in several cases, which, similarly to poor design, should be avoided from a financial and design perspective as well.

Such reasons – with the remark that road administrations are expected to face a shrinking budget relying increasingly on national sources – demand the best use of the modern scientific methods in new pavement and overlay design.

Along these considerations, the need to develop national design methods that are able to satisfy the needs of professionals – both governmental and contractor – of the 21st Century is undeniable. The use of this method must enable the objective comparison of technological possibilities, the consideration of material parameters and a degree of freedom for engineers to balance aspects of financial and environmental sustainability during design.

Although the outlines of several approaches to overlay design have been published in Hungary, in the past years as well [2, 3, 4] the details of these methods – similarly to the one presented in this paper – could not be developed particularly due to the lack of funds, moral and financial commitment of the regulatory bodies.

This paper presents the theoretical considerations and main phases of the proposed method for overlay design, proving the applicability of the method by presenting a design of an actual motorway section based on actual data. For this study, the method will be used to determine the required thickness of an overlay with a given modulus, but various technologies or other solutions can be analogously considered.

2 Theoretical considerations

The current overlay design method relies on deflection measurements (static deflections or dynamic deflections converted into static deflections) of the existing pavement as result of an equivalent single axle's (ESAL, 100 kN) wheel (50 kN). Based on measurement results, the road is divided into homogenous segments which are assumed to have approximately uniform load bearing capacity. Design deflections are calculated based on the measurements to a given reliability level (90 or 95% according to traffic load) for all homogenous segments. The allowable deflections are calculated considering the actual design traffic, in ESALs, and fatigue curves of the existing pavement, which are pre-defined in the standard for three pavement types (super flexible, flexible and semi-rigid).

The required overlay thickness is determined using predefined diagrams given in the standard by the comparison of the allowable and the design deflection. In cases when the design deflections are low, i.e. the overall load bearing capacity of the existing pavement is relatively high, but – due to surface conditions – the pavement still needs an overlay, the comparative method is used. In this case, the required overlay thickness is determined by comparing the effective asphalt thickness of the existing pavement – total asphalt thickness reduced according to results of a visual survey – and the asphalt thickness of a similar, but new pavement type according to the pavement catalogue for the calculated design traffic, naturally considering also milling of layers, if necessary.

The current design methods rely on the assumption that even if the asphalt layers work together in the first period after construction, the weakened lower layers will eventually crack, the bond between new and old layers will weaken and the layers will bend partly separately under the loads of heavy vehicles. Consequently, there will be high horizontal strains at the bottom of the overlay which therefore must be thick enough to endure most of the fatigue loads by itself. This assumption, in favour of security, is true in most cases.

Although in cases when the existing total asphalt layer is thick, built on a base layer and a subbase with good load bearing capacity – i.e. in Hungary primarily at main highways and motorways – and the overlay is required mainly due to surface conditions, this assumption is not necessarily true. These structures will have relatively low dynamic deflections, leading to the use of the comparative method, which, according to a broad professional consent, leads to a significant overdesign of the overlay in these cases.

Because of the rehabilitation the design life is extended for 15–20 years, so the structure must endure excess traffic loads as compared to the original design. Besides, the structure is naturally damaged to some extent due to its past service life and there is particularly some fatigue damage in the asphalt layers. Therefore, e.g. only the change of the wearing course, or simple surfacing to improve surface properties is inadequate even if the overlay is only required due to a poor surface condition, not because of inadequate load bearing capacity. Thus, the mechanical design of the strengthening of the structure is also necessary.

Regarding overlay design of such structures, there may also be significant remaining fatigue life available. It should be considered, because tack coats today are capable of providing a good bond between the overlay and the existing layers, thus the existing asphalt layers will have a significant role in the force system and the required overlay thickness can be significantly lower.

The boundary condition of the presented method is that the pavement to be overlaid consists of thick asphalt layers that work together, built on a good load bearing base and subbase, having low deflections and significant remaining fatigue life.

3 The proposed overlay design method

The proposed mechanical overlay design method is based on the multi-layer theory as shown on Fig. 1. In the infinite halfspace model, the pavement layers are represented by their moduli, thicknesses and Poisson's ratios, while the thickness of the subbase is infinite. The layers are assumed to be linear elastic, thus after some simplifications the Boussinesq-equations will be valid and adaptable to calculate stresses and strains in arbitrary points in the model.

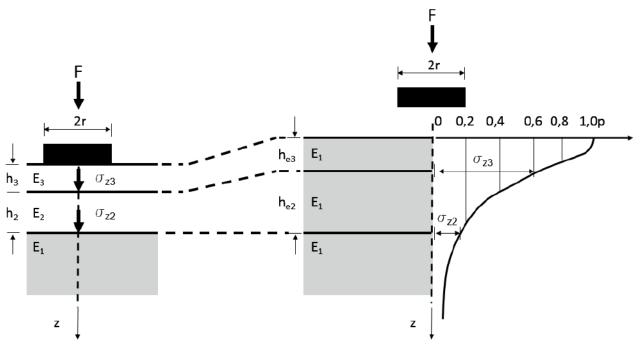

Fig. 1 Multilayer elastic model and equivalent thicknesses

The method relies on dynamic deflection measurements which serve a dual purpose. First, using the centre-of-load deflection the road is divided into homogenous segments, according to the current standard. Second, the overall weakest section within each homogenous segment is determined where core samples are taken. Samples are used to determine actual layer thicknesses. Although samples may also be used to determine material parameters, authors suggest the use of back-calculation to determine moduli of the layers, based on the deflection basin as described in Section 3.4.

The allowable (maximal) strain at the bottom of the total asphalt layer is determined based on the fatigue performance of the lowest, existing asphalt layer and the design traffic calculated for the extended design life of the strengthened pavement. Critical strains occurring at the bottom of the total asphalt layer in cases of various renovation techniques, overlay materials and thicknesses can be calculated using multi-layered elastic model. A rehabilitation scenario will be adequate provided the critical strains are lower than the allowable strains, that is, the fatigue criterion of the overlaid structure is met.

3.1 Determination of homogenous segments

Similarly to the current Hungarian practice, the analysed road is divided into homogenous segments based on either the measured deflections or parameters derived from measurement results. According to the Hungarian standard, a given section may be considered to be homogenous if the coefficient of variation ν is ≤ 0.5 It should be stated that the standard is quite indulgent regarding variation, in light of, e.g., the Swiss regulation stating $\nu \leq 0.35$ [5], however, this is out of the scope of the current study.

There are multiple accepted methods suitable to determine homogenous segments. The method of the absolute differences is often used. In this method, data are further processed after "smoothening", and based on the relationship of a given data and defined critical values, data can be sorted into homogenous groups with a rough approximation, which have the same borders as the sought homogenous segments [6].

A quite precise method, the Bayesian segmentation algorithm is an elegant approach to defining an arbitrary number of "similar" sections within a data set. However, data must meet strict requirements for the method to actually work, which is often not satisfied by field data in the practice [7].

Due to a good approximate accuracy and simplicity in use, the method of the cumulative sums is almost exclusively used in Hungary. The method is based on observing the changes in variance within a data set, by calculating the sums of the difference between individual data and the mean of the data set [8].

The alternation of homogenous sections can be identified conveniently by plotting the function against chainage, by observing or calculation of major changes in the trends of the function. The $S(x)$ cumulative sum function is calculated according to Eq. (1).

$$S_1 = d_{01} - \bar{d}_0$$
$$S_i = d_{0i} - \bar{d}_0 + S_{i-1} \tag{1}$$

where:

d_{01}: centre-of-load deflection in section i [mm],

$\bar{d}_0$: mean of centre-of-load deflections [mm],

S_i: cumulative sum of differences between the deflection in section i and the mean of measured deflections [mm].

This method is naturally valid on any other type of data series, e.g. for the analysis of area parameter data sets, as described in Section 3.2. Experience showed that homogenous sections determined using this method, based on centre-of-load deflections and based on area parameters, are practically the same.

3.2 Representative sections

There are several pavement indicators developed based on the shape and size of the deflection basin or the relationship of given sensors – such as the curvature radius, curvature indices, shape parameters, slope indices etc. Based on these indicators assumptions may be taken regarding the condition of the layers and the structure itself.

The characterization of the entire deflection basin based on area parameter was presumably introduced into the AASHTO design guide [9] based on the PhD thesis of Newcomb, as shown in Eq. (2).

$$AREA = \frac{6 \cdot \left(d_0 + 2 \cdot d_{300} + 2 \cdot d_{600} + d_{900}\right)}{d_0} \tag{2}$$

where:

d_i: deflection at distance i from centre-of-load [mm].

The area parameter developed for Hungarian circumstances, using KUAB FWD device, was given by Adorjányi according to Eq. (3) [10].

$$TP = \frac{1}{12} \cdot \left[\begin{aligned} & d_0 + 1{,}25 \cdot d_{300} + 2{,}25 \cdot d_{600} + \\ & 1{,}25 \cdot \left(d_{200} + d_{450} + 2 \cdot d_{900} + d_{1200}\right) \end{aligned} \right] \tag{3}$$

The significant difference between the two parameters lies in the normalization of the *AREA* using the centre-of-load deflection, d_0. Apart from this, when comparing non-normalised *AREA* and *TP*, a coefficient of determination even higher than 0.99 was found, indicating the parameters to be quite similar (at least in case of the chosen highway section).

The theoretical basis of such parameters is that the size of the deflection basin is proportional to the work done by the force impulse of the falling load of deflectograph [11]. Subsequently, however, this also means that bigger measured deflection basins, using the same load, indicate overall weaker sections, smaller basins indicate stronger sections.

Figure 2 shows the dynamic deflections measured every 50 meters along the analysed 12.5 km road. Parameter *TP* was calculated based on the deflection basins according to Eq. (3) in all sections, and the function of cumulative sums was determined according to Eq. (1). As seen, the road is to be divided into six homogenous segments.

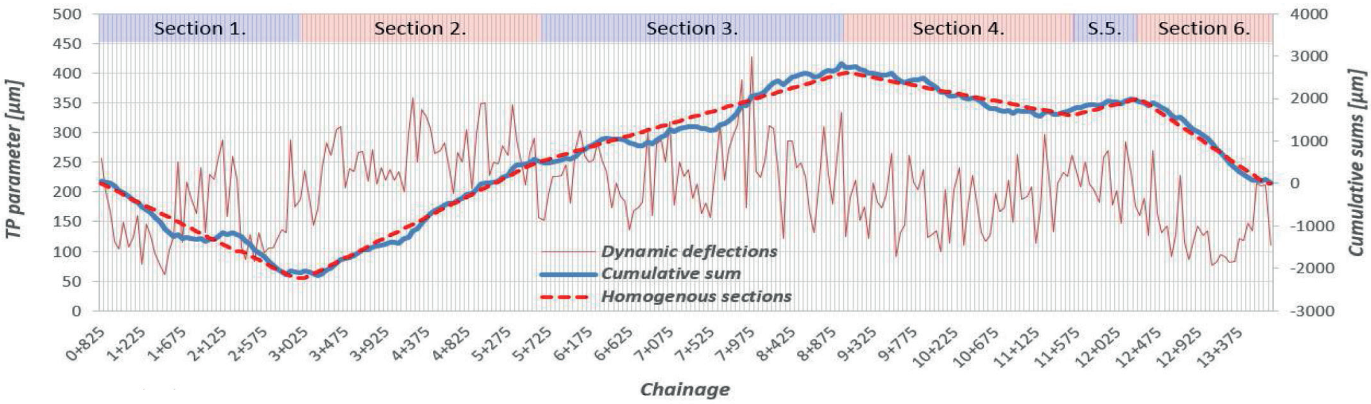

Fig. 2 Deflections, Cumulative sums and Homogenous sections of the analysed road

The third homogenous segment (5 + 575 – 9 + 025) was selected for further analysis. Figure 3 shows the deflection basins measured within this segment. As seen, the highest TP values do not necessarily coincide with the highest deflections.

The section having the 95% relative frequency of TP parameter is considered to be representative. Higher TP values are assumed to indicate local defects in the pavement subject to special attention during construction, not issues to be dealt with at the overlay design stage.

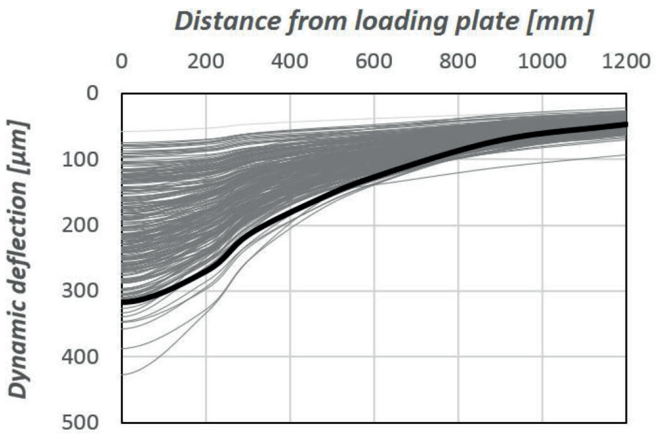

Fig. 3 Deflection basins: all sections and at the representative section

The representative section is located at 7 + 075, the measured deflections are 291, 248, 203, 163, 132, 83, 55 µm, respectively. In this section core samples were taken to determine actual layer thicknesses required for the model. Samples showed an average of 167 mm asphalt thickness (33 mm wearing course, 54 mm binder course and 80 mm base course), and a granular base layer of about 210 mm thickness was found.

3.3 Theoretical model

The analytical design of pavements is based on the response of the structure to physical loads calculated according to an assumed mechanical model. The fundamentals of calculation of stresses and strains in a linear elastic infinite half-space were worked out by Boussinesq, for point loads. However, his original model has no exact solution in case of multi-layer models, such as pavement structures. To solve this problem several methods were developed, which can be discussed divided into four types: methods based on equivalent thicknesses, based on layer-by-layer analytical calculations, based on finite element methods and discrete element methods. Method based on finite and discrete elements are rarely used in practice due to the various numbers of inputs needed and due to their complexity [12, 13]

In practice, the multi-layer models are most commonly used. In this case the pavement structure is defined by the moduli, thickness and Poisson's ratio of the layers. To solve the model, equivalent layer theories or layer-by-layer calculations are used. The methods using equivalent layer theories are based on the work of Odemark [14], while the methods using a layer-by-layer analytical calculation are mostly based on the fundamentals by Burmister [15].

In the proposed method, the multi-layer model can be managed using the Boussinesq-equations after combining the layers. For this step, there are several solutions, from which, again, Odemark's original method [16] should be highlighted, which was later upgraded by Ullidtz [17].

Although Ullidtz used a correctional factor to extend the scope of the Odemark's method, further studies showed that the factors actually depend on the layer thicknesses as well, and for this reason the originally straightforward method became quite difficult to use, requiring difficult individual calibrations [18]. Later, Pronk revised the method and corrected it to be able to consider the circular loads and solved the issue of calibration as well [19]. The essence of the method proposed by Pronk is to merge two layers into a single layer with a virtual – equivalent – thickness and the moduli of the lower layer, but having the same EJ bending moment as the original layers have. The equivalent layer thickness of the two layers is calculated according to Eq. (4)

$$h_{eq} = \left[\frac{A^4 + 4 \cdot N \cdot A^3 + 6 \cdot N \cdot A^2 + 4 \cdot N \cdot A + N^2}{(A+1)^3 \cdot (A+1)} \right]^{\frac{1}{3}} \cdot (h_1 + h_2) \tag{4}$$

where:

$$A, N \quad A = \frac{h_2}{h_1}; \quad N = \frac{E_1}{E_2}$$

h_i thickness of the layers (1-upper, 2-lower layer) [mm],
h_{eq} total equivalent thickness, with moduli E_2 [mm],
E_i modulus of layer i [MPa].

Merging is done on two layers at a time. After the "simplification" of the model to a two-layer system (a linear elastic layer on an infinite halfspace) the altered Boussinesq equations will be capable to calculate – among others – the horizontal strains occurring at the bottom of the asphalt layer. This calculated strain will be the basis of this method being the critical strain used to check the fatigue criteria of the asphalt structure.

3.4 Input parameters

Material parameters required by the model may be determined, by default, by conducting laboratory tests or by back-calculation of FWD deflection basins [20, 21, 22]. As laboratory tests refer to a given layer sawed from core samples, some essential parameters, as layer bond strength, have to be determined and directly considered in the model which considerably complicates calculations. Back-calculation is a method being is able to refer to the overall condition of the pavement, and – indirectly – address all relevant parameters, such as the extent of the working together of the layers, which is obviously important for the fatigue mechanism [23].

Traditional back-calculation methods consist of mainly simplified methods (e.g. based on equivalent thicknesses or surface curvature), gradient relaxation methods or direct interpolation methods. These methods – with some considerations and simplifications – iterate the calculated deflection basin by varying input parameters to an extent where the difference between the calculated and measured basin reaches a tolerance level.

Besides classical techniques, computational capacity today enables the simple and fast use of generic algorithms and various numerical approaches [24, 25, 26], however, these are less used in everyday practice.

In this method, the use of classical methods is proposed, for which in some cases there are even free online software available as well, e.g. EVERCALC which is also based on the multi-layer elastic theory. EVERCALC was shown to give (sufficiently) accurate results provided the iteration seed values are set properly [27].

The standard procedure to consider temperature is in most cases to correct the deflection basin, i.e. the measured deflections. The current Hungarian standard also follows this method, but provides corrections only for the centre-of-load deflections, which are solely used for overlay design (further sensor data are not). Performing back-calculation on a previously corrected deflection basin would result in corrected modulus values, however, there are no proven Hungarian corrections to address the whole basin. As deflections are highly dependant on the environmental properties of a given area, adapting a foreign method without throughout investigation cannot be done. Therefore back-calculation is suggested to be done on the original deflection basin, and back-calculated moduli are suggested to be calculated according to temperature, in case of asphalt layers.

Performing back-calculation on the deflection basin measured in the representative section 7 + 075, the iterated moduli for the asphalt layer, valid for the temperature measured at the surface of the pavement, $T_{surf} = 23°C$, was $E_{Tsurf} = 5426\ MPa$, moduli of the base and subbase were $E_2 = 97.08\ MPa$ and $E_3 = 285.3\ MPa$, respectively (latter two practically independent on temperature).

It must be noted that the moduli of bituminous layers are only interpretable together with the test temperature, and as temperature has a severe effect on moduli corrections must be considered. For this, T_{mid} temperature at depth $z = h/2$ of the asphalt layer with thickness h must be estimated based on the measured T_{surf} surface temperature. This temperature may be extended for the total asphalt layer to the sake of safety. For this estimation from several techniques, Eq. (5) shows a method used by the new German analytical pavement design method [28], developed for a climatic environment which is similar to the one in Hungary.

where:

$$T_{mid} = a \cdot \ln\left(0{,}01 \cdot z + 1\right) + T_{surf} \tag{5}$$

y asphalt temperature at depth x [°C],
x depth below pavement surface [mm],
T surface temperature [°C],
a parameter as a function of T_{surf} [-].

Using Eq. (5), the temperature at the middle of the asphalt layers is estimated in $T_{mid} = 22.03°C \sim 22°C$ based on measured surface temperature $T_{surf} = 23°C$. Consequently, the back-calculated modulus is $E_{Tsurf = 23°C} = E_{Tmid = 22°C} = 5426\ MPa$.

As known temperature not only effects moduli of bituminous layers but also fatigue performance, i.e. during winter and summer the fatigue damage done by heavy vehicles is relatively low as compared to normal *(~20°C)* service temperatures. To consider this, the equivalent temperature is to be introduced into the model, which, based on Hungarian data was found to be by Fi and Pethő about *17.7°C* [29].

Backcalculated moduli valid for $T_{mid} = 22°C = 71.6°F$ must be converted to this $T_{eq} = 17.7°C = 63.86°F$. From several studies proven, considering specific boundary conditions [30], the method used by EVERCALC is applicable, as shown in Eq. (6)

$$E_{Teq} = E_{Tmid} \cdot 10^{-0.000147362 \cdot \left(T_{eq}{}^2 - T_{mid}{}^2\right)} \tag{6}$$

where:

E_{Teq} moduli on the equivalent temperature [MPa],

E_{Tmid} backcalculated modulus on the temperature at the middle-depth of the asphalt layer [MPa],

T_{eq} equivalent temperature [°F],

T_{mid} temperature at middle-depth of the asphalt layer [°F].

Correction of the asphalt modulus according to Eq. (6) in E_{Teq} = *7744 MPa*. Using the corrected asphalt modulus, the backcalculated moduli for the base and subbase, and thickness of all layers, the model can be defined.

4 Calculation of the required overlay thickness

4.1 Calculation of critical strains

Using the model, stresses and strains can be calculated at desired depths beneath the surface for various rehabilitation techniques and/or overlay thicknesses considering material parameters. After merging all asphalt layers according to Eq. (4) in several steps as the bending moment of the virtual and the existing layer will be equal, the horizontal strain at a desired depth z is calculated according to Boussinesq's equation as shown in Eq. (8).

$$\varepsilon_z = \frac{(1+\nu)\cdot\sigma_0}{E}\cdot\left[\frac{\frac{z}{a}}{\left(\sqrt{1+\frac{z^2}{a}}\right)^3}-(1-2\cdot\nu)\cdot\left(\frac{\frac{z}{a}}{\left(\sqrt{1+\frac{z}{a}}\right)^2}-1\right)\right] \tag{7}$$

where:

ε_z strain at depth z [μm/m],

ν Poisson's number [-],

σ_0 uniform circular load under loading plate [MPa],

E modulus of the layer [MPa] ,

z depth below surface [mm],

a radius of the circular loading plate[mm].

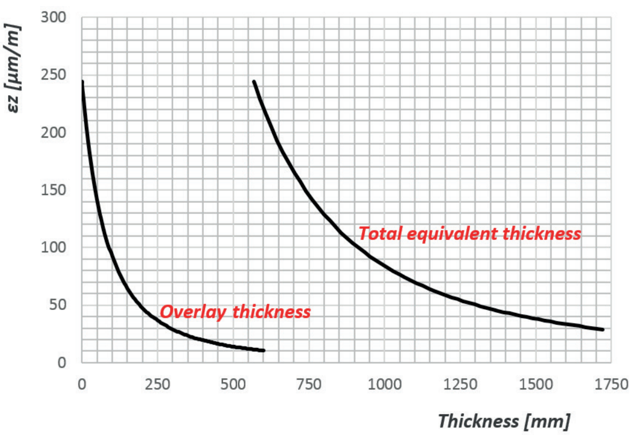

Fig. 4 Strains at the bottom asphalt line as a function of total equivalent thickness and overlay thickness

From the fatigue criterion point of view, assuming the layers work together, the strains will be critical at the bottom of the asphalt layers, i.e. at depth z calculated as shown in Eq. (7).

$$z = h - h_m + h_0 \tag{8}$$

where:

h existing total asphalt thickness [mm],

h_m thickness of milled layers (optional) [mm],

h_o overlay thickness [mm].

In the suggested model, depth z is characterised by the total equivalent asphalt thickness, $z = h_{eq}$. Figure 4 shows strains calculated at the bottom of the asphalt layers as a function of the total equivalent asphalt thickness. The corresponding overlay thickness can be determined by deducting the equivalent thickness of the existing asphalt layers from the total h_{eq} thickness.

4.2 Allowable strains

Allowable strains at the bottom of the asphalt layers are calculated based on the design traffic in ESALs and the fatigue performance of the asphalt layers. Since the analysis of the method for the determination of design traffic is out of the scope of this paper, design traffic may be calculated according to the valid Hungarian standard [1]. Design traffic N of the analysed motorway for a design life of 20 years resulted in 35,917,546 ESALs.

There are several methods to determine or estimate the remaining fatigue life of existing pavements. Provided the original fatigue performance of the asphalt base layer material is known or can be reliably estimated, the remaining life can be inferred based on the ratio of the realised traffic and the original design traffic, perhaps based on the design life and the time past since construction. In this case the actual condition of the base layer is not considered, and this negligence results in a high degree of uncertainty. There are some methods that use multiple empirical functions based on the visual survey of the pavement to estimate remaining service life [31]. Even more methods are accepted to determine the fatigue reserve of the existing pavement based on parameters derived from the deflection basin, although these usually need individual empirical calibration and are therefore difficult to implement [32, 33].

Also empirical correlations are known for estimating the fatigue performance of a given asphalt mix based on its material composition (gradation, binder content and type, binder properties, volume and weight properties). Inputs required for these functions could be easily determined by laboratory tests of the core samples, however, most of these relationships were originally derived from tests of new asphalt mixes, thus there may be again, some uncertainty in their adaptation to in-service pavement layers.

The most straightforward way to determine the fatigue performance of in-service layers while considering their actual condition – especially the fatigue damage already taken – is to adapt laboratory fatigue tests based on mechanical principles. These tests require samples taken from the road itself. For most of these tests, such as the two- or four point bending fatigue tests common in Hungary, the required samples may be prepared from slabs cut from the pavement, or from large diameter core samples drilled. As previous experience showed, both methods are viable [34].

Such solutions are preferable as there is no need for assumptions and simplifications as in the case of other available methods, the fatigue curve of the partly fatigued material can be relatively easily composed by fatigue tests of 18 specimens, according to the relevant standard. However, obtaining the number of proper samples to achieve a reliable fatigue curve can be a challenge, requiring several extensive cuts or drills from the pavement.

The method described by the standard EN 12697-24:2005, Annex E, describes the indirect tensile fatigue (ITFT) test for bituminous materials done on cylindrical specimens. Apart from some research works, the method is quite unknown and not used in Hungary. In this case, the drilling of at least 10–18 samples of 100 or 150 mm diameter is enough.

During the test, the horizontal displacement of the specimen as a result of a cyclic vertical indirect tensile stress is measured in a stress-controlled test mode. Horizontal strain occurring at the middle of the specimen is calculated in function of load cycles. For each test, the cycles required to reach a given failure criteria is registered. In this research, break of the specimen was used as failure criteria. The test results of 10-18 specimens on various stress levels will define the fatigue curve of the specimen. The fatigue curve obtained this way by testing core samples is directly used for the overlay design.

Indirect tensile fatigue tests were conducted on 15 specimens cut from the lowest layer of 15 core samples, at *20°C* temperature and *15 Hz* frequency. The fatigue curve of the asphalt base course is defined by Eq. (9), $R^2 = 0.86$. Figure 5 shows the fatigue curve of the existing base layer.

$$\varepsilon = 1357 \cdot N^{-0.1875} \tag{9}$$

This method was used previously in Hungary for research by Adorjányi. As the link between laboratory and field fatigue failure, based on his results, Adorjányi also suggested shift factors for this fatigue test type, as shown in Table 1. [35].

Table 1 Shift factors for ITFT fatigue tests

Load Class	A	B	C	D	E	K	R
N [10^6 ESALs]	≤0,10	0,1-0,3	0,3-1,0	1,0-3,0	3,0-10	10-30	30-100
υ	12		14		24		30

Considering design traffic of N=35,917,546 ESALs, and shift factor of v = 30 according to Table 1, the allowable horizontal strain at the bottom of the asphalt layer, based on a fatigue criteria equals $\varepsilon_{allowed} = 98.38\ \mu m/m$ (microstrain).

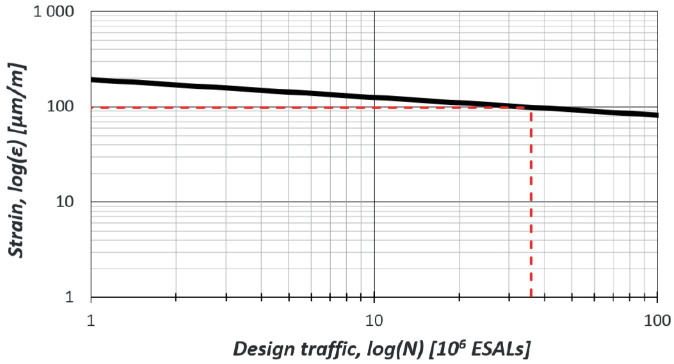

Fig. 5 Fatigue curve of the existing asphalt base course

4.3 The required overlay thickness

The required overlay thickness can be easily determined based on the critical strains and allowable strains at the bottom of the total asphalt layer. Critical strains are calculated according to Eq. (8) for various overlay thicknesses, resulting in the h_{eq}-ε diagram of the structure shown on Fig. 4.

The allowable strains are calculated based on the fatigue curve obtained from samples taken in the representatively weak section in each homogenous segment and the design traffic. The (minimal) h_{eq} total equivalent thickness of the overlaid pavement is adequate when $\varepsilon_{crit} \leq \varepsilon_{allowed}$ is met, see Fig. 6. The required overlay thickness is easily calculated by deduction the equivalent thickness of the existing pavement from the total equivalent thickness.

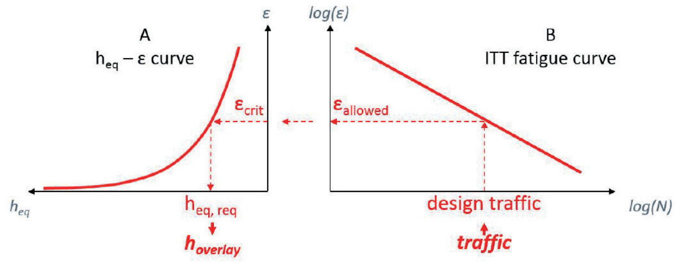

Fig. 6 Determination of the required overlay thickness

Based on $\varepsilon_{allowed}$*=98.38* $\mu m/m$ allowable strain in the case of *N = 35,917,546 ESALs* design traffic, h_{eq}*= 913.4* mm is found as a minimal total equivalent thickness required. The equivalent thickness of the existing pavement (after the milling of the wearing course) equals 568.4 mm, thus the required equivalent thickness of the overlay is about 345 mm (virtual thickness). In reality the virtual thickness of 345 mm, assuming a stiffness for the overlay asphalt of 5500 MPa, is equal to 89.8 mm thickness in reality.

5 Summary and conclusions

The current Hungarian methods for pavement and overlay design are outdated from several perspectives, as the methods, due to the use of catalogue and the predefined curves and assumptions, cannot consider actual material properties and the effects of given technologies, nor can they manage the sometimes affordable compromises regarding trade-offs between durability and financial aspects.

Hungarian highways and heavy duty roads usually consist of thick asphalt pavements built on good load bearing bases and subbases, having substantial remaining fatigue life and layers that work together with each other and the overlay. Such roads often need rehabilitation due to surface conditions instead of structural ones, however, due to the fact of construction itself, the design life is extended and excess traffic loads must be endured as compared to the original design. Because of this, strengthening is also required to extend fatigue life of the overlaid structure.

The proposed method – considering the presented boundary conditions – is suitable for the mechanistic overlay design of such pavements. The paper presents the theoretical considerations of the method, and a case study of the method on an actual highway design.

The proposed mechanistic method resulted in a required overlay thickness of 90 mm overlay with 5500 MPa modulus. The design was carried out using the methods according to the current Hungarian standard as well. Based on the deflections, an overlay requirement of 78 mm was found (allowable deflection: 0.28 mm, design deflection: 0.53 mm). Applying the comparative method due to the low deflections resulted in an overlay thickness of 185 mm (pavement from catalogue consisting of 200 mm granular base and 290 mm total asphalt thickness compared to the existing one). The latter two methods only depict layer thicknesses, no material properties are stated. According to the valid standard [1], the higher thickness should be chosen, as seen there are considerable differences. Comparing the results of various methods, the mechanistic design resulted as expected, between the method based on deflection and the comparative method.

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