
# *InteMat4PMS*

## Integration of material-science based performance models into life-cycle-analysis processed in the frame of pavement management systems

Technische Universität Braunschweig
PMS-Consult Vienna
University of Belgrade
ViaTec AG Winterthur

**Deliverable D2: Performance functions for road materials and pavement structures**

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# Deliverable 2: Performance functions for road materials and pavement structures

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| **Project Coordinator:** | Technische Universität Braunschweig<br>Institut für Straßenwesen (ISBS) | Germany |
| | | |
| **Other Partners:** | | |
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| University of Belgrade, Faculty of Civil Engineering | | Serbia |
| ViaTec AG | | Switzerland |

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| **Report coordinated by:**<br>M. Wistuba | **Contributors:**<br>M. Wistuba<br>A. Weninger-Vycudil<br>G. Mladenovic<br>A. Ringleb/Alisov | **Supported by:**<br>J. Litzka | **Reviewed by:**<br>J. Litzka |

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# 1 Introduction

## 1.1 Project framework

From technical, economic and environmental point of view, the objective of ***InteMat4PMS*** is to improve the accuracy of performance prediction in the frame of PMS. As advanced mechanistic material models are used in this project, rather than empirical performance relationships, the application of innovative construction materials and advanced pavement structures is encouraged. Hence, this project has the potential for better performances of pavements, for an overall improvement in pavement durability, and thus, for most cost-effective road maintenance decisions.

Inter alia, InteMat4PMS meets research needs for

- specifying performance models based on material characteristics and structural data,
- incorporating structural parameters in probabilistic performance prediction models,
- employing enhanced structural probabilistic prediction models in PMS,
- and evaluating probabilistic models and advanced PMS tools on structural data from a test section.

***InteMat4PMS*** will result in a limited number of demonstration case studies where "sophisticated analysis solutions" are used that integrate material science, performance modelling, and decision tools for cost-benefit analysis. As the number of test sections available is limited and because of the short time periods of observation no final solutions will be given. Validation and calibration of the developed methods are omitted.

Within ***InteMat4PMS*** no new material models or LCA/LCCA models will be developed. The project focuses on project level analysis and consequently on primary response and structural performance models.

Even though research in ***InteMat4PMS*** is restricted to asphalt materials, the effect of supporting (granular, cement treated, innovative, etc.) layers on pavement lifetime is not left disregarded, as the principal layout of the PMS approach will be designed such, that any information on other layers can be integrated in the performance prediction analysis.

## 1.2 Objectives and Outline of Deliverable D2

The overall objective of Deliverable D2 is to increase the predictability of pavement performance models and to increase the possibility of conducting an efficient road management process through PMS.

Pavement evolution over time basically depends upon four major variables, i. e. traffic load repetitions, climatic conditions, pavement age (counted from date of construction or recent rehabilitation), and structural capacity. The loss in structural capacity over time can be characterized by means of mechanistic analysis based on laboratory testing of pavement materials and on derivation of physical performance functions.

The laboratory study realized in the frame of ***InteMat4PMS*** provides all data, needed for information and feedback regarding the installation, usage and fitness of the developed PMS algorithms. Conventional and performance-based lab tests are conducted to identify target parameters that are most influencing structural properties of asphalt pavements. Detailed description of the employed testing procedures is presented in Deliverable D1. Permanent deformation testing is realized by means of cyclic triaxial stress tests (CTST) according to EN 12697-25. Top-down cracking caused by thermal and traffic loads is derived from Thermal Stress Restrained Specimen Test (TSRST) and Uniaxial Cyclic Tensile Stress Test (UCTST) according to EN 12697-46. Bottom-up cracking caused by structural fatigue is derived from fatigue tests according to EN 12697-24, structural pavement distress is calculated based on asphalt stiffness modulus according to EN 12697-26. Materials are delivered from real test sections. This enables to check developments within real conditions to find design flaws, which have to be eliminated in an iterative design and development process. Test site description and test results will be presented in an internal report of WP 5. Final results of laboratory study and interpretation of test results will be presented in the final report of ***InteMat4PMS*** (Deliverable D5).

In Chapter 2 of Deliverable D2 possible methodologies for incorporating physical performance functions into holistic PMS are discussed. The aim is to propose performance model calibration recommendations and to establish the most suitable and adequate calibration procedure generally applicable in any PMS solution. This requires general discussion of methods for material characterization and for modelling structural pavement performance. Physical performance functions depending on mix properties need to be identified which are potential candidates to be included in holistic PMS. Empirical performance functions need to be listed, where calibration can be realized. And the methodology of the calibration itself needs to be developed.

In Chapter 3, with a view to various possibilities for realizing the calibration procedure, the methodology rated to be most appropriate for improving performance prediction and judged as most adequate for various PMS solutions is discussed in detail. It is finally prepared for demonstration purposes demanded in Deliverable D3.

# 2 Connecting mechanistic with empirical performance functions

## 2.1 Approach

The major challenge in developing holistic PMS is to connect data obtained from empirical analysis to data obtained from mechanistic analysis. This requires that data arising from these approaches be compatible, always regarding specific road sections according to representative variables (structure, traffic, geometry, climate).

On the one hand, empirical analysis in the frame of PMS is based on long-term periodic observation of pavement condition in the field. From deterministic or probabilistic analysis approaches, an empirical performance function (EPF) is identified.

On the other hand, mechanistic analysis is based on coring pavement samples, on laboratory characterization and on damage analysis of the pavement structure. Material performance relationships in function of repetitive loads are identified such as fatigue damage accumulation law. However, distress stated in laboratory tests and real distress are not congruent, and thus, the results of material tests are not directly applicable to predict pavement lifetime. Using material performance laws directly within PMS is problematic.

This is equally valid for structural pavement data coming from in-field condition surveys, such as from measurements using Falling-Weight-Deflectometer, Thermal Mapping, or Ground Penetration Radar. Data from these investigations cannot directly be linked with appropriate performance functions.

In addition to laboratory testing, pavement design analysis is needed for development of material performance laws to be used within PMS, considering the full pavement structure, and using material parameters and material performance laws as input in structural stress-strain-analysis, in damage analysis, and in pavement performance prediction.

To date, various efforts are under way to enhance PMS by adjusting EPF through outputs from mechanistic analysis. Effective adjustment of EPF is of advantage, as by considering the mechanistic outputs important factors such as repetitive traffic loads, climate, material characteristics, layer thicknesses, and subgrade support, can be taken into account. The special importance of calibrating EPF is the economic impact, since consideration of substantial pavement loss will reduce prediction error, and hence, economic evaluation will become more accurate.

Most PMS are based on deterministic EPF displaying relationships for dimensional condition parameters (for general explanation of deterministic performance modelling see Deliverable D1). Therefore, ***InteMat4PMS*** focuses on linking deterministic EPF to mechanistic analysis. Moreover, ***InteMat4PMS*** pursues the objective to develop and recommend a most practicable methodology with highest possible operating advantages and flexibility for adaptation to different existing PMS solutions. A general approach is needed as performance functions in PMS are always calibrated according to the specific conditions of a country or region where they are to be used.

With regard to these requirements, in ***InteMat4PMS*** adjustment methodology is considered most suitable if data from mechanistic analysis are directly taken into account for EPF calibration. The calibration approach shall permit to relate the pavement condition per section to results from laboratory testing of pavement samples. It does not rely on long-term pavement observation, and consequently allows for theoretical performance prediction without any further material testing, which is of major advantage.

In order to calibrate EPF, a group of distress data obtained from mechanistic analysis is required which serve to find the adjusted performance curve. This group of data must represent a relatively long period of time. Then, the process of calibration consists of determining which adjustment factors achieve the best agreement between EPF and the data obtained from mechanistic analysis.

Figure 1 illustrates the principle of simple adjustment (or adaptation, or calibration) procedure. Initial EPF choice is related to the relevant performance indicator. Data obtained from mechanistic analysis (modelling behaviour over longer period) provide additional information on pavement condition. If these data deviate from EPF, EPF is adjusted. In doing so, the mathematical type of function is kept, but the initial EPF is shifted to fit the mechanistic data best. In this way, EPF is calibrated, and performance prediction is improved. In the example illustrated in Figure 1, adjustment leads to acceleration of damage accumulation and to early failure. However, adjustment can equally express retarded damage accumulation.

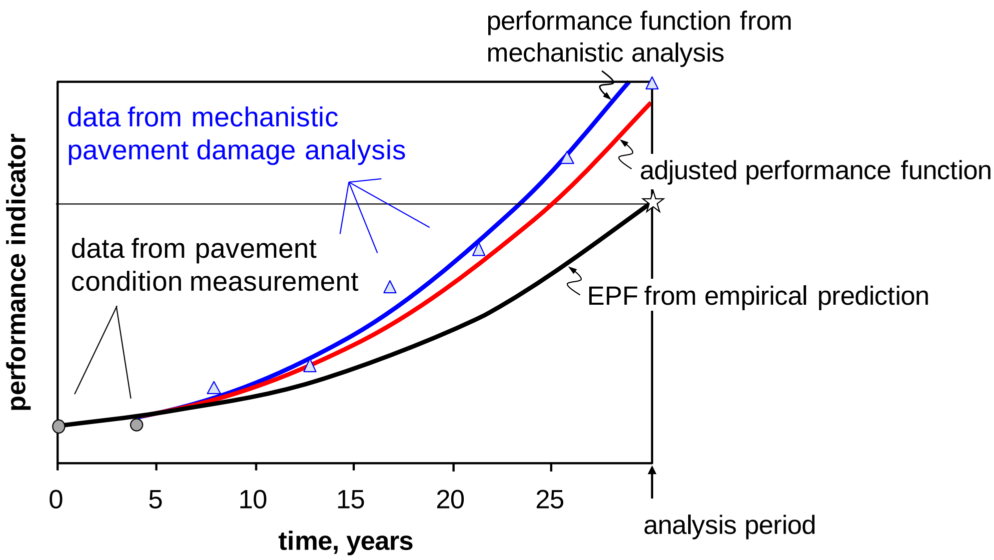


Figure 1. Principle for adjustment of empirical pavement performance function (EPF) using data obtained from mechanistic damage analysis.

EPF adjustment calls for suitable mechanistic data. This requires establishment of mechanistic data on the basis of incremental damage approach in function of time, or load repetitions respectively, where the number of load repetitions correlate with pavement durability.

Knowledge is limited on mechanistic performance functions that connect pavement distress with load repetitions for long time spans (of several decades) and that are thus suitable for the purpose of EPF adjustment. To-date, the arrogating issue of accurate modelling of pavement damage for a defined analysis interval with steady load, and of continuous accumulation of incremental damage over the total analysis period, is not satisfactorily solved for heterogeneous structures like asphalt pavements. This is primarily due to the complex visco-elasto-plastic asphalt properties and the impossibility of realistic modelling the complex distress modes observed in asphalt pavements for high numbers of load repetitions.

With respect to distress modes such as permanent deformation, thermal cracking, reflective cracking, no suitable mechanistic models of general applicability are available today.

Adjustment procedures developed in ***InteMat4PMS*** therefore primarily focus on fatigue-related performance indicators, i. e.

- traffic load related alligator cracking initiated at the bottom of the asphalt layers (bottom-up fatigue cracking),
- and thermal and traffic load related cracking initiated at the pavement surface (top-down low-temperature fatigue cracking),

as well as on mechanistic data considered in conventional pavement design.

As to conventional pavement design approach, mechanistic performance data are derived from stress-strain-analysis in multi-layered asphalt pavement structures, from cyclic testing of asphalt materials obtained from pavement samples and from incremental damage modelling for high number of load repetitions. A priori linear-elasticity is postulated, assuming that pavement reacts purely elastic to abrupt traffic loading and that temperature conditions at the bottom of the asphalt layers (considered as the critical point for fatigue crack initiation) is moderate.

As soon as the adjustment procedure works for fatigue modelling, calibration procedures shall be developed for further distress types in analogy.

## 2.2 Mechanistic pavement performance analysis

### 2.2.1 Pavement design approach

Pavement design based on pavement life-time estimation is generally realized by using mechanistic analysis tools. A multi-layer performance model for asphalt pavement structure is assumed taking into account the composite-like material morphology and the mix-design of asphalt mixtures. Pavement response is expressed in terms of stresses, strains and deflections under loads in order to assess pavement distress throughout its design life.

Micro-crack propagation due to fatigue damage is considered as a primary deterioration mechanism in asphalt materials. The bond energy between mastic and stone, the viscosity and the tensile/shear strength are parameters determining fatigue mechanisms of asphalt pavements. Loss in pavement substance value in consequence of material fatigue is targeted in mechanistic performance modelling and design analysis for asphalt pavements.

Best pavement performance is expected for design of structures which are sufficiently thick to resist to the design loads because they are made of materials lasting over the design period without any significant fatigue deterioration. However, as distress mechanisms in pavements are far more complex that can be investigated by means of laboratory testing or modelling, any mechanistic-empirical pavement design needs to be adjusted or calibrated to the real pavement performance. For consistency with empirical pavement design, an empirically derived shift-factor is introduced into the mechanistic design approach in order to adjust the output from mechanistic pavement design. Therefore, any mechanistic pavement design is in fact a mechanistic-empirical design approach.

The mechanistic design procedure used in ***InteMat4PMS*** is based on linear elastic multilayer theory. The calculation procedure is organized in an iterative approach as illustrated in the flow chart in Figure 2.

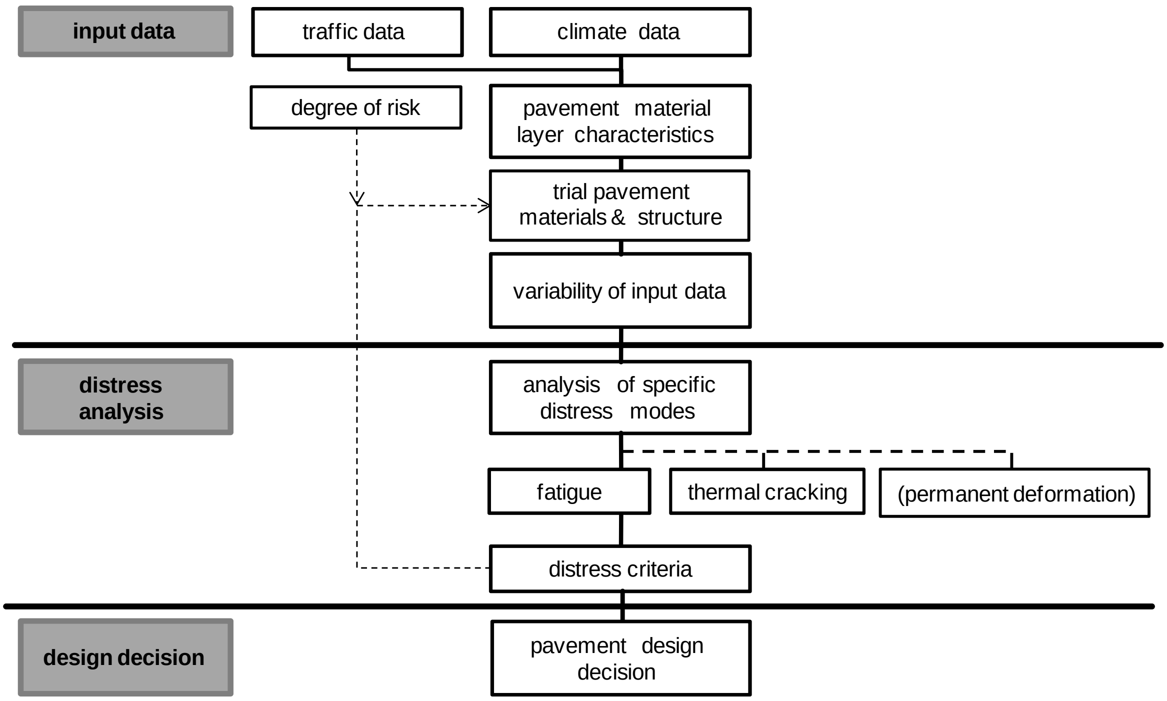


Figure 2. Flow chart of general design approach.

Input data needed for design analysis are related to climate, traffic, pavement structure and materials. The thermal analysis is required for determination of the temperature distribution in the pavement, providing both the thermal dilatation/shrinkage strains and the actual rheological behaviour of asphalt material for the subsequent mechanical analysis.

### 2.2.2 Climate and traffic inputs

Pavement surface temperature is determined by heat balance equation, where pavement surface is regarded as a closed thermodynamic system. In order to satisfy energy conservation law, all thermal effects due to radiation, soil heat flux, convective heat exchange to air, and heat flux due to evaporation and condensation of water equal to zero value at the pavement surface. By means of parameterization of the thermal budget, the resulting pavement surface temperature can be expressed in function of standard parameters of meteorological observation such as air temperature, global radiation, and wind velocity (for details see Wistuba et al., 2001; Wistuba, 2002, 2003).

Based on energy balance principles the relevant pavement surface temperature can be derived from the time variation of standard meteorological parameters. Exemplarily, Figure 3 illustrates the temperature variation at the pavement surface calculated for a period of 1 year.

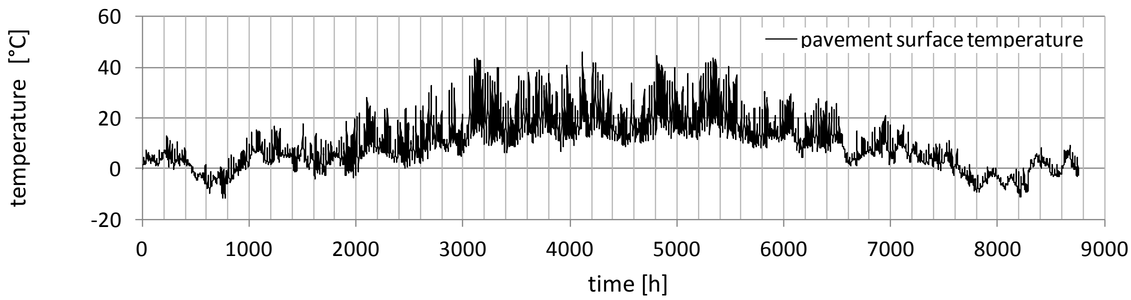


Figure 3. Variation of pavement surface temperature over time (Walther & Wistuba, 2012).

Consequently, Fourier heat transfer law can be used to calculate the temperature distribution in the pavement structure for every single hour of the design period (for details see Wistuba & Walther, 2012).

Pavement structure is decomposed in individual layers representing a multilayer system according to Burmister (1943), see Figure 4.

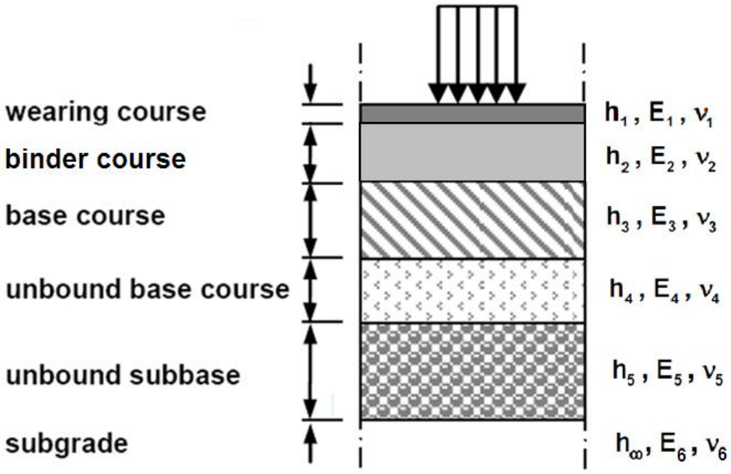


Figure 4. Multilayer pavement structure.

Young's modulus of each individual layer can be calculated in function of temperature for every single hour. Figure 5 shows temperatures on an hourly time scale within the pavement (left), and the corresponding distribution of derived Young's Modulus (right).

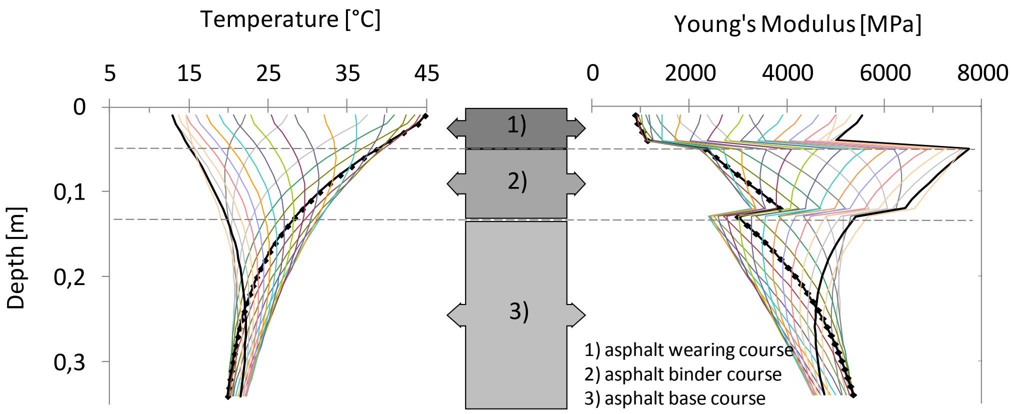


Figure 5. Pavement temperature distribution (left) and corresponding Young's modulus (right) for 24 hours (Walther & Wistuba, 2012).

In the design process, pavement temperature and Young's modulus of each individual layer is calculated for every single hour considering the whole design period. Consequently, stress-strain analysis is performed for every single hour.

Traffic loading is assumed as instantaneous. The tire load is considered by a uniform load with circular contact area.

Traffic loading and temperature induced stress are thus superimposed on a narrow time scale, and all stress situations – including extreme loading events such as traffic overload during hot summer periods – are incorporated into the design procedure. Figure 6 exemplarily shows resulting strains per hour for a period of 1 year (= 8760 h) arising at the bottom of the asphalt base course from 11 tons axle loading.

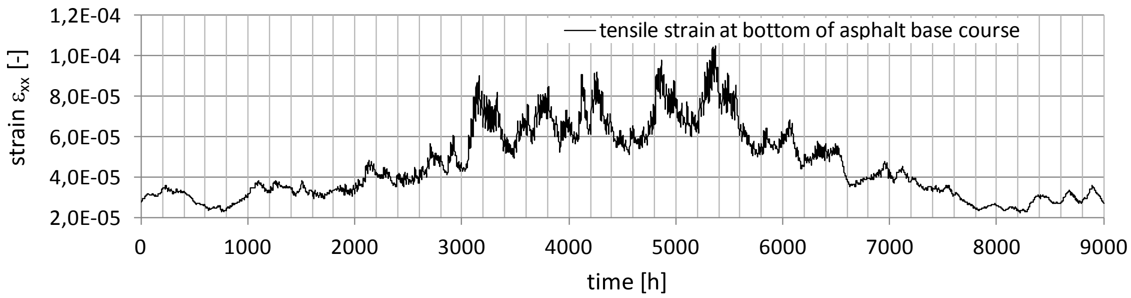


Figure 6. Resulting horizontal strains per hour at bottom of the asphalt base course (Walther & Wistuba, 2012).

### 2.2.3 Materials inputs

#### 2.2.3.1 Stiffness and fatigue

Material stiffness of the individual pavement layers are assessed in laboratory tests. Young's Modulus of pavement materials is derived in function of temperature distribution over pavement layers.

Fatigue strength evaluation of the asphalt layer is based on cyclic stress tests on asphalt mix samples. According to the European Standard for fatigue testing (EN 12697-24), the classical fatigue criterion is used, and determination of the number of load repetitions at failure $N_{f/50}$ is undertaken. The results of the tests end in a fatigue Wöhler line which is drawn by executing a linear regression between $N_{f/50}$ and $\varepsilon_i$, indicating the fatigue life duration in function of the applied load amplitude. The Wöhler line can be expressed in the general form

$$N = \alpha \cdot \varepsilon^{-\beta} \qquad \text{Equation 1}$$

where $\alpha$ and $\beta$ are experimentally derived material constants. Finally, the slope of the fatigue line and the initial strain amplitude $\varepsilon_i$ corresponding with a fatigue life of $10^6$ load cycles are determined, as required for CE-declaration of conformity by the European Standards (EN 13108).

#### 2.2.3.2 Low-temperature behaviour

For modelling low-temperature performance of asphalt, parameters determining low-temperature properties of asphalt materials are derived from laboratory testing.

Cooling and relaxation behaviour of the specific asphalt wearing course material is studied through thermal stress restrained specimen tests (TSRST). Test results with conventional binders are exemplarily shown in Figure 7.

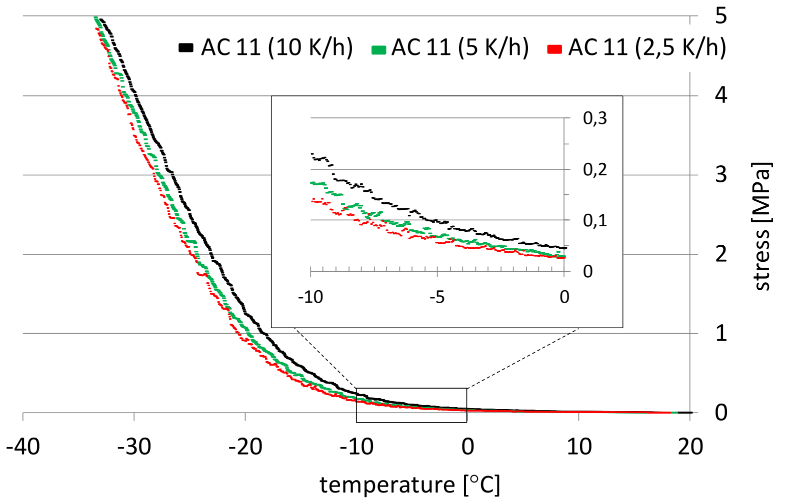


Figure 7. TSRST data (AC 11) at different cooling rates (Walther & Wistuba, 2012).

Low temperature fatigue behaviour is assessed through uniaxial cyclic tensile stress test (UCTST), where the lower stress level is derived from TSRST. The resulting Wöhler line can be expressed by

$$N = k_1 \cdot \Delta\sigma^{k_2} \qquad \text{Equation 2}$$

where $k_1$ and $k_2$ are experimentally derived material constants.

### 2.2.4 Stress-strain analysis

The solution of the coupled thermo-mechanical problem is performed in two steps: first, the temperature distribution in the road section is determined on the basis of the temperature scenarios at the road surface.

The obtained temperature distribution in the pavement structure serves as input for the second analysis, the mechanical analysis, considering the change of material parameters with temperature, and the traffic load. The temperature history obtained from the previous thermal analyses is considered at each point of the pavement structure. At the beginning of the simulation, the temperature is set equal to the initial (stationary) solution of the thermal problem and the road is assumed to be in a state free of stress.

Stress-strain analysis of the asphalt pavement layers is realized by linear elastic multilayer theory according to Burmister (1943). All layers are described by an elastic material model, characterized by elastic modulus (Young's modulus, Poisson's ratio). Hence, Young's modulus in function of temperature is as the major input parameter describing asphalt layer properties.

As to thermal stress analysis at the pavement surface, the cryogenic tensile stress $\sigma_{kry}$ can be analytically estimated from the following equations in function of

- initial stress $\sigma_0$,
- Young's modulus E [MPa],
- the temperature rate $\dot{T}$ [$K \cdot h^{-1}$],
- the coefficient of thermal expansion $\alpha_T$ [$K^{-1}$] as derived from thermal shrinkage test,
- the tensile viscosity λ [MPa·s], which is regarded as constant in certain temperature ranges and can be derived from relaxation test,
- and the relaxation time $t_R$,

reading

$$\sigma_{cry}(T,t) = \sigma_0 \cdot e^{-\frac{t}{t_R}} - \alpha_T \cdot \lambda(T) \cdot \dot{T} \cdot \left(1 - e^{-\frac{t}{t_R}}\right)$$ Equation 3

and

$$t_R(T) = \frac{\lambda(T)}{E(T, T_R)}.$$ Equation 4

As pavement surface temperatures are known in every hour, the cooling rate can be derived for the specific time variation of temperature.

### 2.2.5 Modelling loss in pavement substance value

Analysis of fatigue evolution requires a damage hypothesis. Linear damage law is assumed for constant loading conditions per analysis interval (Figure 8).

Incremental damage in the analysis interval i is calculated from the number $n_i$ of load repetitions accumulated during the interval, and the number $N_{f,i}$ of load repetitions until failure that is obtained from fatigue testing and Wöhler curve modelling (for details see Deliverable D1).

Accumulation of damage is realized by linear summation applying Miner's law (1945). Cumulative damage D over total analysis period is composed of individual incremental damage ratios, reading

$$D = \sum_{i=1}^{n} \frac{n_i}{N_i} \leq 1$$ Equation 5

where $n_i$ is the number of actual traffic load repetitions at strain/stress level i, and $N_i$ is the number of allowable traffic load repetitions to failure at strain/stress level *i*. This equation allows predicting fatigue life in terms of the number of theoretically allowable load repetitions (due to traffic and/or thermal load cycles). If cumulative damage value D equals 1, the pavement fails due to material fatigue, and hence, the pavement substance value has decreased to 0 % of the initial value.

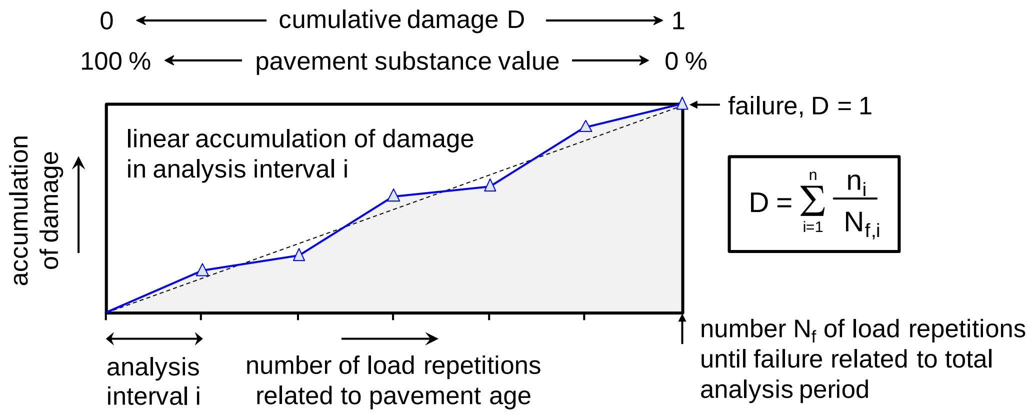


Figure 8. Linear accumulation of damage simulated in pavement design analysis.

Traffic and climate loading are distributed over time. Therefore, the result of pavement design can be expressed either in terms of time or in terms of the number of load repetitions.

Often, the theoretical residual fatigue life is expressed in the unit of years. The number of actual traffic load repetitions used for design considerations is formulated by a mean value for one design year (based on traffic counts and extrapolative estimations). Fatigue life is then calculated by the ratio of the number of allowed load repetitions to the mean number of traffic load repetitions of the design year, where the number of allowed load repetitions is derived from fatigue testing on the asphalt base- and wearing course material by superposition of the results that were obtained from tests at various strain/stress levels.

# 3 Calibrating empirical performance functions used in PMS

## 3.1 Definition of calibration

Empirical performance functions (EPF) used in pavement management systems should be transferable across different technological and climatic conditions. This can be achieved by combining mechanistic-empirical approach in deriving the underlying predictive relationships, theory of material and pavement behavior under traffic loading and rigorous statistical analysis using data gathered from a wide range of road and climatic conditions and vehicle types.

Empirical performance functions (EPF) simulate the future changes in the road system from current conditions and the reliability of results is dependent of two primary considerations (Bennett & Paterson, 2000):

- how well the data provided to the model represent the current condition and influencing factors, in the terms understood by the model, and
- how well the predictions of the model fit the real behavior and the interactions between various factors for the variety of conditions to which it is applied.

The first item is related to data input including correct interpretation of the data input requirements, and achieving a quality of input data that is appropriate for the desired reliability of the results.

The second item is related to the calibration of the model, i.e. adjusting the model parameters to enhance how well the forecast and outputs represent the changes and influences over time and under various interventions.

Environmental conditions, local construction materials, practices and quality, and the effectiveness of maintenance have the most significant impact on the road deterioration models. All these influences can be controlled through the EPF calibration factors.

The HDM-4 model recognizes three level of EPF calibration (Bennett & Paterson, 2000) depending on the required level of effort and resources:

- **basic application** that includes calibration of the most sensitive parameters based on desk studies, using minimal field surveys
- **calibration** that requires measurements of additional input parameters and moderate field surveys to calibrate key predictive relationships to local conditions, and
- **adaptation**, that assumes undertaking major field surveys and controlled experiments to enhance the predictive relationships or to develop new and locally specific relationships to be included in the model.

## 3.2 Empirical performance functions selected for calibration

### 3.2.1 Selection of performance indicators

Modern PMS is based on the LCC-analysis method. Actual pavement condition is needed for the initialization of LCC-analysis. Performance functions reflect the development of pavement condition over time and are important for estimation of remaining service life. Output of the LCC-analysis is pavement performance considering different maintenance and budgetary scenarios.

Pavement condition is expressed through performance indicators. Hence, performance indicators are the primary measure of pavement condition as they characterize key properties of pavements, such as longitudinal or transverse evenness, cracking, surface defects, macro-texture, friction, bearing capacity, environmental impacts (noise, air pollution).

- A performance indicator can be defined in the form of a single dimensional technical parameter, which is obtained from on-site investigation (through visual engineering judgement or special measurement device), such as rut depth or friction value. As to most PMS, dimensional performance indicators are used as input in performance functions. These performance functions are important for driving recommendations for maintenance needs, and for estimation of remaining service life.

Besides single dimensional indicators, also dimensionless, combined, and general (global) performance indicators are distinguished (cp. Litzka et al., 2008).

- Based on a standardization model, any performance indicator can also be expressed through a single dimensionless figure or index. Commonly, dimensionless indicators are ranked on a 0 to 5 scale, with 0 representing a pavement in best condition and 5 in worst condition. Once the predicted pavement condition falls within a trigger range, a suitable maintenance strategy is assigned to the pavement section.
- The use of dimensionless performance indicators is of special advantage, when pavement condition is composed of various forms of pavement distress. Then pre-combined dimensionless performance indices form a single value. Typically, visual condition surveys are combined into a single index to provide an overall measure of performance.
- Finally, general performance indicators such as safety index, serviceability/comfort index, structural index, and environmental index may be derived through mathematical combination and weighting procedure of individual dimensionless single and/or pre-combined performance indicators. General performance indicators are especially relevant to decision-makers for assessing the general condition of the network and to recommend future maintenance strategies and funding requirements.

Based on information collected within COST 354 the performance indicator for description of structural pavement properties (as focused in InteMat4PMS) is the structural index. This combined indicator can be calculated based on different single indicators like Bearing Capacity, Cracking, Evenness and Rutting (see Figure 9).

| **Level** | **Comfort Index** | **Safety Index** |
|---|---|---|
| Minimum | PI_E | PI_F |
| Standard | PI_E, PI_SD, PI_R | PI_F, PI_R, PI_T |
| Optimum | PI_E, PI_SD, PI_R, PI_T, PI_CR | PI_F, PI_R, PI_T, $PI\_SD_{cat1}$ *), $PI\_SD_{cat2}$ |
| **Level** | **Structural Index** | **Environmental Index** |
| Minimum | PI_B | - |
| Standard | PI_B, PI_CR | - |
| Optimum | PI_B, PI_CR, PI_R, PI_E | PI_E or air pollution, PI_T or noise labelling; $PI\_SD_{cat2}$ |

PI_E…PI evenness
PI_F…PI friction
PI_CR…PI cracking
PI_SD…PI surface defects (all categories)
$PI\_SD_{cat2}$…PI surface defects category 2
PI_R…PI rutting
PI_T…PI macro-texture
PI_B…PI bearing capacity
$PI\_SD_{cat1}$…PI surface defects category 1

*) including bleeding only

Figure 9. Input parameters for combined performance indices (Litzka et al., 2008).

For the development of a calibration procedure in order to adjust EPF the performance indicator for cracking was selected as the most suitable approach. The index for cracking is usually composed from different input variables.

Within COST 354 a method was developed which combines different appearance forms or types of cracking (linear, alligator, reflective, etc.) into one single cracking parameter ($TP_{cr}$). The combination procedure takes into account the significance of various forms of cracking in form of different weights.

The technical parameter for cracking ($TP_{cr}$) is defined as a weighted sum of different types and dimensions (area, linear, numbers) of cracking in reference to the investigated area. The different dimensions are converted into equivalent areas. The result is a cracking rate, which can be calculated through the use of the following equations. To simplify the calculation procedure the mathematical function for the cracking rate is split into 3 partial expressions (area, length, and cracked elements (e.g. concrete slabs)) which can be summarized as follows. Equation 9 considers cracks in concrete pavement slabs as well, however for asphalt pavements Equation 9 is used considering a factor of $TP_{cr,E} = 0$.

$$TP_{cr} = Min(100; TP_{cr,A} + TP_{cr,L} + TP_{cr,E}) \qquad \text{Equation 6}$$

<u>Area:</u>

$$TP_{cr,A} = Min\left(100; \frac{1}{A_{ref}} \cdot \sum_m \left[ W_m \cdot \sum_i (S_{cr,a,i} \cdot A_i) \right] \cdot 100 \right) \qquad \text{Equation 7}$$

where $TP_{cr,A}$ is the cracking rate area [%], $A_{ref}$ is the reference area, $W_m$ is the weight of cracked areas, $S_{cr,a,i}$ is the severity of crack type I, and $A_i$ is the cracked area of crack type i.

<u>Length:</u>

$$TP_{cr,L} = Min\left(100; \frac{1}{A_{ref}} \cdot \sum_n \left[ W_n \cdot l_{width,l} \cdot \sum_j (S_{cr,l,j} \cdot L_j) \right] \cdot 100 \right) \qquad \text{Equation 8}$$

where $TP_{cr,L}$ is the cracking rate length [%], $A_{ref}$ is the reference area, $W_n$ is the weight of cracked length, $l_{width,l}$ is the standard influence width of linear cracks, usually 0.5 m based on "OECD Full-scale Pavement Test", $S_{cr,l,j}$ is the severity of crack type j, and $L_j$ is the cracking length of crack type j.

<u>Element:</u>

$$TP_{cr,E} = Min\left(100; \frac{1}{A_{ref}} \cdot \sum_o \left[ W_o \cdot l_{area,k} \cdot \sum_k (S_{cr,E,k} \cdot E_k) \right] \cdot 100 \right) \qquad \text{Equation 9}$$

where $TP_{cr,E}$ is the cracking rate element [%], $A_{ref}$ is the reference-area, $W_o$ is the weight of cracked elements, $l_{area,k}$ is the standard area of elements with cracks (e.g. area of concrete slab), $E_{ref}$ is the total number of referred elements (e.g. number of concrete slabs), $S_{cr,E,k}$ is the severity of cracks on an element of crack type k, and $E_k$ is the number of elements with cracks of type k.

For the calculation of the cracking rate it is necessary to apply different weights for different types of cracking. Based on a statistical evaluation of collected information from the experts within COST 354 the following weights were recommended. However, these weights can be adapted individually by the user.

Table 2 presents the mean, median, and a proposed range (minimum and maximum) of weights for different types of cracking subject to flexible pavements. The range is defined by the second largest and second lowest value of the analyzed data volume. Similar tables are available for rigid and semi-rigid pavements.

Table 1: Weights for cracking types for flexible pavements

| Cracking type | Weight W' [0-1] (0 = lowest importance, 1 = highest importance) | | | |
|---|---|---|---|---|
| | Flexible pavements | | | |
| | min*) | max*) | median | mean |
| alligator cracking | 0.9 | 1.0 | 1.0 | 0.9 |
| longitudinal cracking | 0.7 | 0.8 | 0.7 | 0.7 |
| transverse cracking | 0.6 | 0.8 | 0.8 | 0.7 |
| block cracking | 0.6 | 1.0 | 1.0 | 0.8 |
| thermal cracking | 0.6 | 0.9 | 0.7 | 0.7 |
| reflective cracking | 0.8 | 0.9 | 0.8 | 0.8 |

*) second largest and second lowest value of statistical evaluation

The weights represent the influence of the different cracking types on a relative basis. The maximum weight in use is always equal to 1.0, independently from which cracking types are used in Equations 7 to 9. In case, the selected maximum weight as proposed by Table 2 is unequal to 1.0 all the weights proposed by the tables have to be multiplied by the following scaling factor x. The cracking types considered in the process of scaling weights must not be changed within one road network investigation, even if a specific cracking type does not occur on an individual road section (as a part of the network considered).

$$x = \frac{1}{\max\left(W_1^{'};W_2^{'};\ldots;W_n^{'}\right)} \qquad \text{Equation 10}$$

$$W_1 = x \cdot W_1^{'};\ W_2 = x \cdot W_2^{'};\ldots..;W_n = x \cdot W_n^{'} \qquad \text{Equation 11}$$

### 3.2.2 Selection of empirical performance functions

As concerns the structural index the following EPF are selected in order the exemplarily demonstrate the calibration procedure as described in chapter 4.3. The selection is based on the experiences of the project members using this function but also on the availability of data for the demonstration of the examples. The EPF selected cover the approaches applied (a) on an international level based on Highway Development and Management Model HDM-4 developed by University of Birmingham and supported by the World Bank, (b) in Germany, and (c) in Austria.

(a) HDM-4 cracking model for asphalt pavements

The Highway Development and Management model HDM-4 is the successor of the World Bank Highway Design and Maintenance Standards model HDM-III, which was used by various road agencies all over the world in the last 20 years. HDM-4 uses separate models for surface distress (cracking, raveling, potholing, and edge repair), deformation (rutting and roughness), and surface texture (texture depth and skid resistance) of asphalt pavements. To allow for local calibration each relationship contains a calibration coefficient or scaling factor.

HDM-III included models for structural cracking, as “all” and “wide” (wider than 3 mm) cracking, based on the relationships defined by Paterson (1987). Paterson developed models that were both time-based and traffic-based. Although the traffic-based model was “generally superior”, it was not applicable to all surface types and only time-based model was incorporated in HDM-III and later on used in HDM-4.

The first version of the HDM-4 in addition included models for transverse thermal cracking (Odoki & Kerali, 2000), while it is also proposed to include models for reflection cracking in future versions (Morosiuk et al, 2001), which has not been implemented yet.

For each type or cracking two distinct phases are identified, the time to the development of the distress (the initiation phase) and the progression phase (Figure 10).

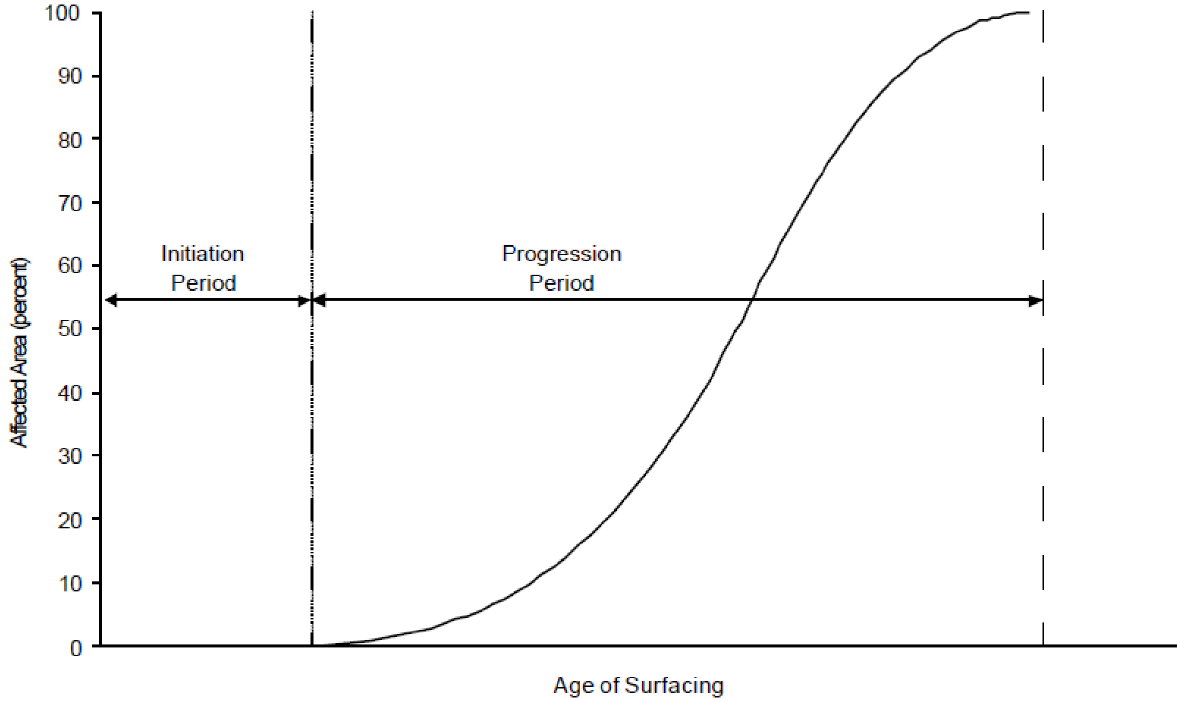


Figure 10. Initiation and progression phase of distress (Odoki & Kerali, 2000).

The cracking EPF in HDM-4 are based on the distressed area that is expressed in per cent of total carriageway area. The distressed area of linear cracks is calculated as the length of the crack in meters multiplied by 0.5 meters.

Total area of structural cracking and reflection cracking is given by equation 12.

$$ACAT = ACA + ACF \qquad \text{Equation 12}$$

where:
ACAT – total area of "all" structural and reflection cracking, (% of total carriageway area),
ACA – total area of "all" structural cracking, (% of total carriageway area), and
ACF – total area of reflection cracking, (% of total carriageway area).

Since it is proposed that in HDM-4 reflection cracking is treated as "wide" cracking, the total area of "wide" structural and reflection cracking (cracks wider than 3 mm) is provided by equation 13.

$$ACWT = ACW + ACF \qquad \text{Equation 13}$$

where:
ACWT – total area of "wide" structural and reflection cracking, (% of total carriageway area), and
ACW – total area of "wide" structural cracking, (% of total carriageway area).

The total area of cracking combines the structural, reflection and transverse thermal cracking and is defined as follows:

$$ACRA = ACA + ACF + ACT \qquad \text{Equation 14}$$

where:
ACRA – total area of carriageway cracked, (% of total carriageway area), and
ACT – total area of transverse thermal cracking, (% of total carriageway area).

Paterson (1987) defined the area of indexed cracking, ACX, as a weighted average of "all" and "wide" cracking, as defined by equation 15.

$$ACX = 0.62 \cdot ACAT + 0.39 \cdot ACWT$$ Equation 15 ([1])

The existing models for reflection cracking are based on the extensive study performed in Malaysia that showed that reflection cracking depends on traffic loading, existing structural strength and surface condition. However, no models for reflection cracking have been included in HDM-4 yet, since no studies were found that would isolate climatic variables, in particular the effects of the daily temperature range (Morosiuk et al., 2001).

Models for transverse thermal cracking have been introduced in HDM-4. The cracking intensity is modelled as the number of cracks per km.

The models for initiation and progression of structural, transverse thermal and reflection cracking are provided in the Annex.

(b) German cracking model for asphalt pavements

The German empirical performance function, which describes the development of cracking on asphalt pavement, is the result of a statistical analysis of pavement condition data. This data were collected within the two campaigns 1997/98 and 2001/02 on a uniform base. On the basis of plausible data, changes in condition variables between 1997/98 and 2001/02 are analyzed as target variables as a function of influential factors available network-wide (Hinsch et al., 2005).

The determined performance functions were used in an ex-post analysis for projecting condition data from 1997/98 to 2001/02. A comparison with the condition variables actually determined in 2001/2 at the level of the ZEB (condition measurement and assessment campaign) analysis sections shows that the derived performance functions have very satisfactory estimation accuracy. The performance functions derived for homogeneous performance groups are catalogued together with recommendations for applications so as to permit incorporation into existent guidelines/work papers. Homogeneous performance groups are defined as road elements or sections with the same pavement construction type (wearing course, bound base courses, sequence of layer types, new or already rehabilitated, etc.), the same design index category and on the same lane (heavy vehicle or other lane).

For the empirical performance functions (EPF) to be applied in the German pavement management system, it is first necessary to determine the cumulative transitions of equivalent 10-t unit axles belonging to heavy traffic AL for each section i (of example, homogeneous section, maintenance section) for the year tZEB of the current ZEB program, based on the year of the last maintenance measure. This can be used to select and apply – separately for each condition variable j – the catalogued, standard behavioural function of the associated homogeneous behavioural group k (Hinsch, et al., 2005):

$$z_{i,j,tZEB} = a_{j,k} + b_{j,k} \cdot AL_{i,tZEB}{}^{c_{j,k}}$$ Equation 16

where $z_{i,j,tZEB}$ is the estimated condition variable for section i and condition feature j in the year tZEB of the current ZEB program, $AL_{i,tZEB}$ is the cumulative axle transitions AL for section i in the year tZEB, and $a_{j,k}/b_{j,k}/c_{j,k}$ is the catalogued coefficients of the standard behavioral function for condition variable j and homogeneous behavioral group k.

In Figure 11, the curves for the standard EPF of cracking on asphalt pavements in Germany can be seen. For different types of pavements (new and already rehabilitated ones) the tables with the respective coefficients a, b and c are available.

[1] In the Equation, the sum of the weighting factors is above 1. The formula in the presented form is taken from HDM III, even if there may be a typing error (see Odoki J.B, Kerali, H. R. 2000. *Volume four: Analytical Framework and Model Descriptions, Highway Development and Management Series, HDM-4*. International study of Highway Development and Management (ISOHDM), World Road Association, PIARC, Paris, France, page C2-29, equation 5.25.).

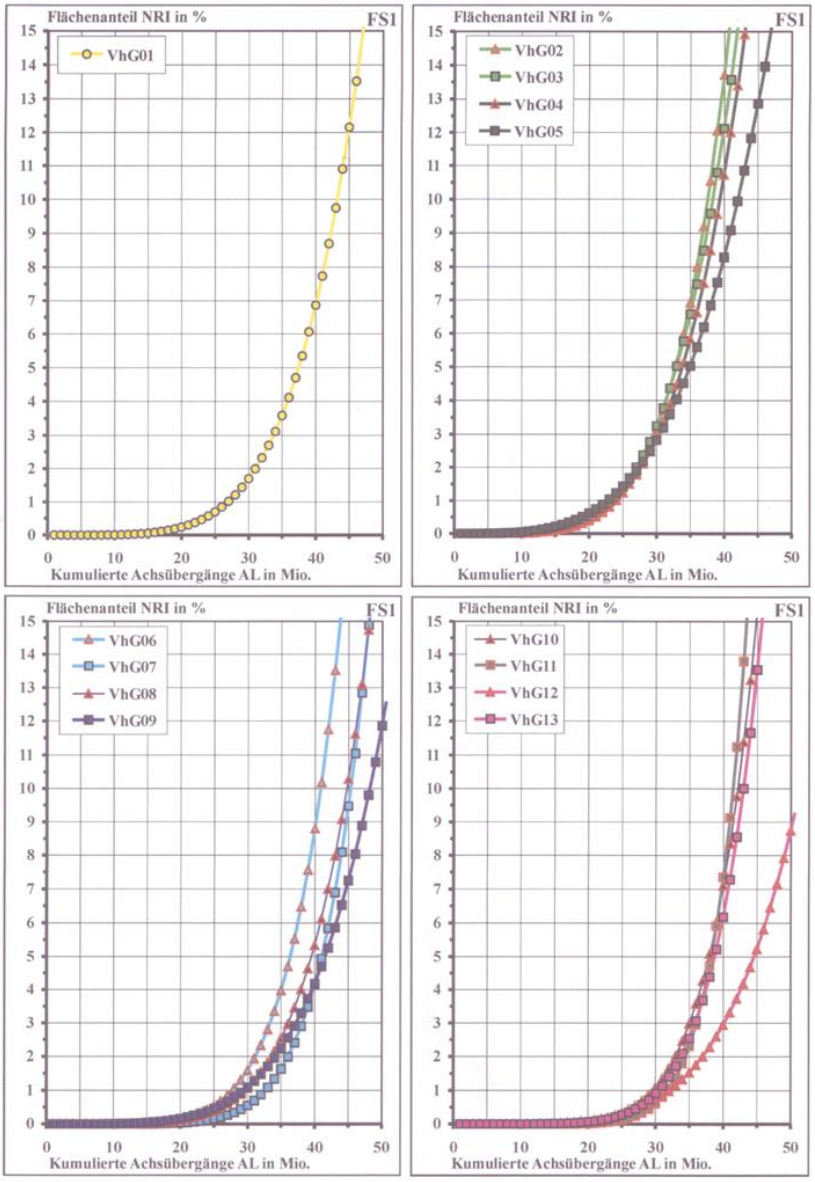


Figure 11. Standard EPF of German cracking model (alligator cracking NRI) used for the heavy vehicle lane (FS1) on asphalt pavements for 13 different pavement categories (labeled VhG01 to VhG13) (Hinsch et al., 2005).

The estimated condition variable $z_{i,j,tZEB}$ frequently deviates from the condition variable $z_{i,j,bZEB}$ determined in the current ZEB. In such cases, the catalogued coefficients of the standard EPF (non-calibrated function) for the parameter ($a_{j,k}$) and ($b_{j,k}$) need to be adapted as follows (Hinsch et al., 2005):

$$\alpha_{i,j,k} = a_{j,k} \cdot \frac{z_{i,j,bZEB}}{z_{i,j,tZEB}} \qquad \text{Equation 17}$$

$$\beta_{i,j,k} = \frac{z_{i,j,bZEB} - \alpha_{i,j,k}}{AL_{i,tZEB}{}^{c_{j,k}}} \qquad \text{Equation 18}$$

where $Z_{i,j,bZEB}$ is the observed condition variable for section i and condition feature j from the current ZEB program in the year tZEB, $\alpha_{i,j,k}$ is the adapted initial state (axle segment) for section i, condition variable j and homogeneous behavioral group k, $\beta_{i,j,k}$ is the adapted slope coefficient for section i, condition variable j and homogeneous behavioral group k, and $b_{i,j,k}$ is the slope coefficient for section i, condition variable j and homogeneous behavioral group k.

Transformation of the parameter $a_{j,k}$ into a section-specific value $\alpha_{k,j,k}$ is based on the assumption that the current condition is derived to a certain extent from the (individual, section-specific) initial state following production or a maintenance measure, so that the initially standardized axle segment needs to be matched retro-actively with the standard EPF. The need for matching the slope coefficient $b_{j,k}$ and determining a section-specific coefficient $\beta_{i,j,k}$ arises from the fact that the influencing factors comprising inventory and traffic available network-wide only make it possible to explain a small part of the scattering in condition trends, despite a formation of homogeneous behavioural groups. The coefficients adapted according to section are used for condition forecasting in the pavement management system for each year t of the forecasting and observation period as follows (Hinsch, et al 2005):

$$z_{i,j,t} = \alpha_{i,j,k} + \beta_{i,j,k} \cdot AL_{i,t}{}^{c_{j,k}} \qquad \text{Equation 19}$$

where $z_{i,j,t}$ is the estimated condition variable for section i and condition feature j in the year t, $AL_{i,t}$ is the cumulative axle transitions AL for section i in the year t, $\alpha_{i,j,k}$ is the adapted initial state (axle segment) for section i condition variable j and homogeneous behavioral group k, $\beta_{i,j,k}$ is the adapted slope coefficient for section i, condition variable j and homogeneous behavioral group k, and $c_{j,k}$ is the catalogued curve coefficients (powers) for condition variable j and homogeneous behavioral group k.

(c) Austrian cracking model for asphalt pavements

The Austrian cracking model for asphalt pavements is based on a detailed statistical analysis of pavement condition data from two large measuring campaigns, which were carried out in 1999 and 2001. The model is principally based on findings obtained under the research project “Statistical Methods for the Evaluation of Pavement Condition Data” and the application of Bayesian statistical methods (Molzer et al., 2000) and updated in 2002 (Molzer et al., 2002).

Beside the detailed regression models evaluated for various types of pavements, simplified performance models were derived for the ongoing development of the Austrian Pavement Management System. The regressors used include the age of the surface layer, the design index DI and a material specific coefficient a.

$$TP_{cracking} = \exp\left[-3.60517 + a \cdot Age_{Surflayer} + \ln\left(Age_{Surflayer} + 0{,}01\right) - 0.5 \cdot \ln\left(DI + 0.01\right)\right] \qquad \text{Equation 20}$$

where $TP_{cracking}$ is the technical parameter of cracking in form of a cracking rate [%], $Age_{Surflayer}$ is the age surface layer, DI is the Design index (DI ≤ 0.5…underdesigned pavement, 0.5 < DI < 2 is the properly designed pavement; DI ≥ 2…overdesigned pavement).

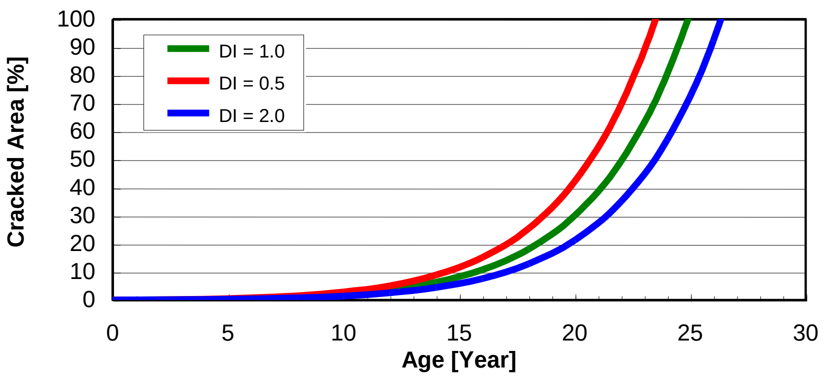


Figure 12. Performance prediction model for cracking on motorways and expressways in Austria according to (Molzer et al., 2002).

The Austrian cracking model is a general function, which was designed using the entirety of pavement performance information available (measurements and visual assessment), traffic data, pavement data and climate data and is representative of Austria. For the purposes of section-based analysis and the assessment of pavement performance it must be assumed, however, that the general model will most likely not reflect the conditions found on specific sections. In order to be able to make forecasts nonetheless, the individual model have to be calibrated to the conditions prevailing on specific sections. The performance function can be adjusted by changing the function through application of a "calibration factor" (change of slope) while observing certain boundary conditions. With this method, the last value in a series of measurements is necessarily an element of the section-based performance function. Compared with calibration based on a series of measurements (Method 1), the measuring point method is, however, significantly more sensitive to measuring or recording inaccuracies. The performance function of a performance characteristic *i* on section *j* calibrated by applying a calibration factor is defined by the following function (Weninger-Vycudil, 2003):

$$Y_{i,j}^{*} = K_{i,j} \cdot Y_{i,j} = \frac{CI_{i,j}^{*}}{Y_{i,j}(t^{*})} \cdot Y_{i,j} \qquad \text{Equation 21}$$

where $Y_{i,j}^{*}$ is the calibrated performance function on section j of characteristic i, $Y_{i,j}$ is the non-calibrated performance function on section j of characteristic i, $K_{i,j}$ is the calibration factor on section j of characteristic i, $CI_{i,j}^{*}$ is the condition value of characteristic i on section j at last inspection time $t^{*}$, and $Y_{i,j}(t^{*})$ is the function value of performance function of characteristic i at time $t^{*}$.

## 3.3 Calibration procedure based on deterministic analysis approach

### 3.3.1 Theoretical background

The deterministic analysis approach needs to integrate the output of the mechanistic analysis directly into the EPF. As basis for this approach deterministic EPF, which were developed on condition data assessment can be used in general. The essential input parameter for describing the structural deterioration of the road is the traffic load. In those cases, where the age is the main input parameter, a translation of age into number of loads by using an adequate traffic prediction model (which is usually available) is the recommended solution.

In general the approach can be categorized into the following 3 steps, i. e. (1) section based calibration of the selected EPF, (2) analysis of laboratory fatigue testing results, and (3) integration of laboratory results into the EPF. The approach can be used for different types of EPF. Of course, the selection of an adequate EPF has to be in coincidence with the local framework condition. A road section, which was already treated or overlaid with a new wearing course needs a different EPF in comparison to a road sections, where all layers were constructed at the same time. All three selected models (HDM 4, German and Austria) offer this opportunity by selecting different model parameters of the origin EPF. Nevertheless, the process described below is in all cases the same.

(1) Section based calibration of the EPF

The first step in the recommended approach is the calculation of the actual load repetitions and the starting point of the EPF. This will be carried out usually in form of section based calibration (or adaptation) of the EPF by using the section-specific traffic information, the respective model-parameters and condition data from actual condition inspections or measurements ($PI_{meas,t}$). The following results are the output of this process:

- number of load repetitions $N_{meas,t}$ (or $n_t$) at actual time t,
- calibrated EPF function $PI'(N_{meas,t},PI_{meas,t})$.

The calibration (or adaptation) of the EPF by using section specific input parameter causes usually a change in the slope of the function and moves the curve into the measured point (PI). The following Figure 13 shows this step schematically.

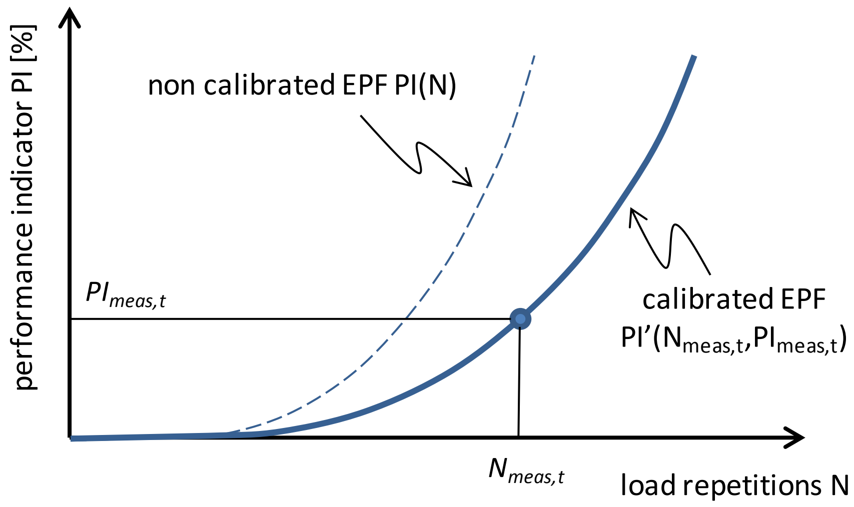


Figure 13: Calibration of EPF by using section specific input data (traffic, condition).

(2) Analysis of laboratory fatigue testing results

Within the next process-step the number of load repetitions $N_{f,D}$, where the pavement construction reaches a certain damage D has to be estimated by using laboratory fatigue testing. The number of load repetitions $N_{f,D}$ is the essential output of laboratory fatigue testing and structural analysis (see Chapter 3.2), and it is used for the definition of the laboratory calibrated EPF. For the testing of the samples it is important, that the coring and the last condition measurement have to be carried out at the same time, so that an additional damage caused by additional number of loadings will be minimized. The output of this process is the number of load repetitions $N_{f,D}$ which is related to a specific amount of damage D (%). The following Figure 14 shows schematically the relationship between increase of damage and increase of load repetitions, where the increase of damage starts at load repetition $N_{meas,t}$:

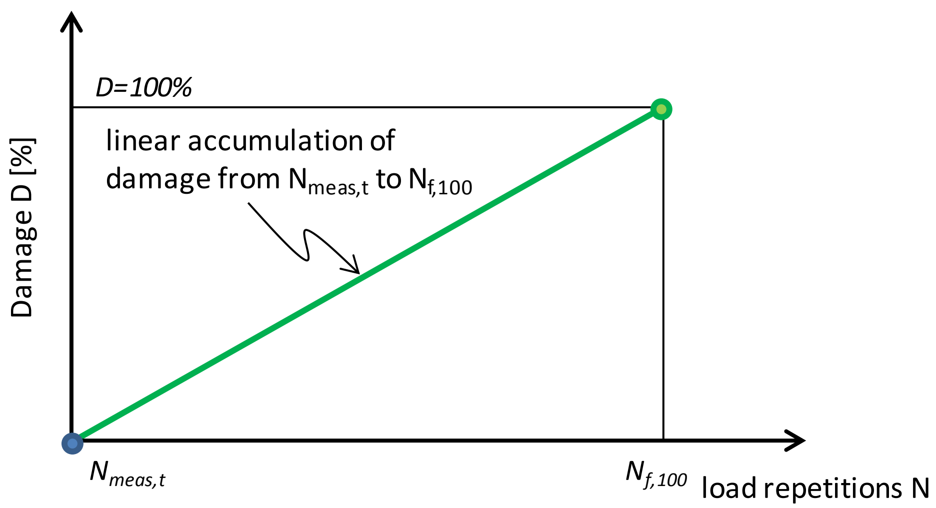


Figure 14: Output of laboratory fatigue testing. $N_{meas,t} = n_t$.

In case of using the age as the input parameter for the EPF (instead of load repetitions) the $N_{f,D}$ has to be translated into years (or intervals).

(3) Integration of laboratory results into the EPF

To integrate the results of the laboratory fatigue testing and analysis into the calibrated EPF the damage D with the corresponding load repetitions $N_{f,D}$ must be brought in line with the calibrated model $PI'(N_{meas,t},PI_{meas,t})$. Thus, it is necessary to calculate the number of load repetitions $N_{PI',D}$ where the PI of the calibrated EPF reaches a damage stage D.

In many cases this could be a D of 100% or a pavement substance value of 0%. For the integration of the fatigue testing and analysis results into the calibrated EPF a scaling factor $X_f$ can be used and which can be calculated as follows:

$$X_f = \frac{N_{f,D} - n_t}{N_{PI',D} - n_t} \qquad \text{Equation 22}$$

where $N_{f,D}$ is the number of load repetitions for a damage of D % during laboratory fatigue testing, and analysis, $N_{PI',D}$ is the number of load repetitions for a damage of D % derived from PI', and $X_f$ is the scaling factor for fatigue.

In practice, the scaling factor $X_f$ stretches or shrinks the PI'-curve along the N-axis and enables a new performance prediction of the PI for any time t+N of the analysis period. The result of the scaling is the new adjusted performance curve PI''. In the following equation the mathematical solution for the calculation of this PI'' at time t+N is shown:

$$PI''_{t+N} = PI_{meas,t} + PI'\left(X_f \cdot \Delta(n_{t+N} - N_{meas,t})\right) \qquad \text{Equation 23}$$

where $PI''_{t+N}$ is the laboratory calibrated EPF at time t+N, $PI_{meas,t}$ is the value calibrated EPF at time t (last measurement), PI' is the calibrated EPF function $PI'(N_{meas,t},PI_{meas,t})$, $n_{t+N}$ is the number of load repetitions at time t+N, $N_{meas,t}$ is the number of load repetitions at time t (last measurement), and $X_f$ is the scaling factor for fatigue.

The following Figure 15 shows schematically the integration of the results of laboratory fatigue testing and the attributed analysis of the whole pavement structure into the performance prediction model:

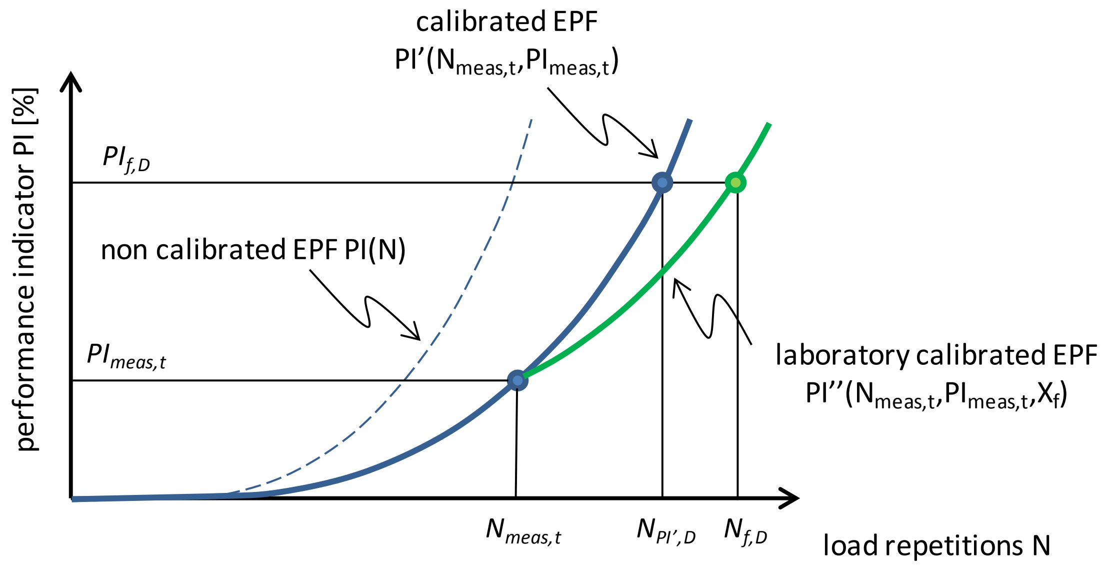


Figure 15: Laboratory calibrated EPF (schematically).

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# Annex A. The cracking models in HDM-4

# 1 Structural cracking

## 1.1 Initiation of structural cracking

The models for predicting the time to initiation of "all" structural cracking in asphalt pavements with stabilized base are different from models for pavements with other types of base. Also the models distinguish between pavements with original surfacings and those that have been resealed or overlaid. For the later ones the amount of cracking in the previous bituminous surface layer is taken into account and separate EPFs are provided for cold mix, slurry seal, cape seal, and other surface types. Models also include the construction defects indicator CDS to enable user to distinguish between pavements that are more likely to crack from those that are more prone to plastic deformation.

Models for structural cracking initiation were developed based on the assumption that initiation happens when 0.5% of carriageway area is cracked.

Time to initiation of "all" structural cracking (ICA) and "wide" structural cracking (ICW) is estimated based on the following expressions, depending on the base and surface type:

- stabilized base, original surfacing (HSOLD = 0):

$$ICA = K_{cia} \cdot \left\{CDS^2 \cdot a_o \cdot exp\left[a_1 \cdot HSE + a_2 \cdot In(CMOD) + a_3\, In(DEF) + a_4 \cdot YE4 \cdot DEF\right] + CRT\right\} \qquad \text{Eq. 24}$$

- stabilized base, overlay or reseal (HSOLD > 0): Eq. 25

$$ICA = K_{cia} \cdot \left\{CDS^2 \cdot \begin{bmatrix}(0.8 \cdot KA + 0.2 \cdot KW)(1 + 0.1 \cdot HSE) + (1 - KA)(1 - KW) \cdot \\ \left[a_o \cdot exp\left(a_1 \cdot HSE + a_2 \cdot In(CMOD) + a_3\, In(DEF) + a_4 \cdot YE4 \cdot DEF\right)\right]\end{bmatrix} + CRT\right\}$$

- other base, original surfacing (HSOLD = 0):

$$ICA = K_{cia} \cdot \left\{CDS^2 \cdot a_o \cdot exp\left[a_1 \cdot SNP + a_2 \cdot \frac{YE4}{SNP^2}\right] + CRT\right\} \qquad \text{Equation 26}$$

- other base, overlay or reseal (HSOLD > 0), for material type other that cold mix, slurry seal and cape seal:

$$ICA = K_{cia} \cdot \left\{CDS^2 \cdot a_o \cdot \left[MAX\left(\begin{matrix} exp\left[a_1 \cdot SNP + a_2 \cdot \frac{YE4}{SNP^2}\right] \cdot MAX\left(1 - \frac{PCRW}{a_3}, 0\right), \\ a_4 \cdot HSNEW \end{matrix}\right)\right] + CRT\right\} \qquad \text{Eq. 27}$$

- other base, overlay or reseal (HSOLD > 0), for material type: cold mix, slurry seal and cape seal:

$$ICA = K_{cia} \cdot \left\{CDS^2 \cdot a_o \cdot \left[MAX\left(\begin{matrix} exp\left[a_1 \cdot SNP + a_2 \cdot \frac{YE4}{SNP^2}\right] \cdot MAX\left(1 - \frac{PCRW}{a_3}, 0\right), \\ a_4 \end{matrix}\right)\right] + CRT\right\} \qquad \text{Eq. 28}$$

- Initiation of "wide" structural cracking:

$$ICW = K_{ciw} \cdot MAX\left[(a_5 + a_6 \cdot ICA), a_7 \cdot ICA\right] \qquad \text{Equation 29}$$

where:

ICA – time to initiation of "all" structural cracking (years)

ICW – time to initiation of "wide" structural cracking (years)

CDS – construction defects indicator for bituminous surfacing:

| Surfacing condition | | CDS |
|---|---|---|
| Dry (brittle) | Normally about 10% below design optimal binder content | 0.5 |
| Normal | Optimal binder content | 1.0 |
| Rich (soft) | Normally about 10% above design optimal binder content | 1.5 |

YE4 – annual total number of equivalent standard axles (millions per lane)

SNP – average annual adjusted structural number of the pavement. SNP represents the structural contribution of surface, base and subbase layers, and subgrade

HSOLD – thickness of previous underlying surfacing layers (mm)

DEF – mean Benkelman beam deflection in both wheel paths (mm)

CMOD – resilient modulus of soil cement (GPa) (in the range between 0 and 30 GPa for most soils)

HSNEW – thickness of the most recent surfacing (mm)

HSOLD – total thickness of previous underlying surfacing layers (mm)

PCRA – area of “all” cracking before latest reseal or overlay (% of total carriageway area)

PCRW - area of “wide” cracking before latest reseal or overlay (% of total carriageway area)

KW = MIN (0.05 MAX (PCRW-10,0), 1)

KA = MIN (0.05 MAX (PCRA-10,0), 1)

HSE = MIN (100,HSNEW+(1-KW) HSOLD)

$K_{cia}$ – calibration factor for initiation of “all” structural cracking

$K_{ciw}$ – calibration factor for initiation of “wide” structural cracking

CRT – crack retardation time due to maintenance (years)

Default values of model coefficients are provided in tables 3, 4 and 5.

Table 2: Default coefficients for models for initiation of "all" structural cracking

| Pavement type | Surface material | HSOLD | Equation | $a_o$ | $a_1$ | $a_2$ | $a_3$ | $a_4$ |
|---|---|---|---|---|---|---|---|---|
| Asphalt mix on granular base | All | 0 | 14 | 4.21 | 0.14 | -17.1 | | |
| | All except cold mix | >0 | 15 | 4.21 | 0.14 | -17.1 | 30 | 0.025 |
| | Cold mix | >0 | 16 | 13.2 | 0 | -20.7 | 20 | 1.4 |
| Asphalt mix on asphalt base | All | 0 | 14 | 4.21 | 0.14 | -17.1 | | |
| | | >0 | 15 | 4.21 | 0.14 | -17.1 | 30 | 0.025 |
| Asphalt mix on asphalt pavement | All | >0 | 15 | 4.21 | 0.14 | -17.1 | 30 | 0.025 |
| Asphalt mix on stabilized base | All | 0 | 12 | 1.12 | 0.035 | 0.371 | -0.418 | -2.87 |
| | | >0 | 13 | 1.12 | 0.035 | 0.371 | -0.418 | -2.87 |
| Surface treatment on granular base | All | 0 | 14 | 13.2 | 0 | -20.7 | | |
| | All except Slurry and Cape Seal | >0 | 15 | 13.2 | 0 | -20.7 | 20 | 0.22 |
| | Slurry Seal and Cape Seal | >0 | 16 | 13.2 | 0 | -20.7 | 20 | 1.4 |

Table 3: Default coefficients for models for initiation of "all" structural cracking, continued

| Pavement type | Surface material | HSOLD | Equation | $a_o$ | $a_1$ | $a_2$ | $a_3$ | $a_4$ |
|---|---|---|---|---|---|---|---|---|
| Surface treatment on asphalt base | All | 0 | 14 | 13.2 | 0 | -20.7 | | |
| | All except Slurry and Cape Seal | >0 | 15 | 4.21 | 0.14 | -17.1 | 20 | 0.12 |
| | Slurry Seal and Cape Seal | >0 | 15 | 4.21 | 0.14 | -17.1 | 20 | 0.025 |
| Surface treatment on asphalt pavement | All | >0 | 15 | 4.21 | 0.14 | -17.1 | 20 | 0.12 |
| Surface treatment on stabilized base | All | 0 | 12 | 1.12 | 0.035 | 0.371 | -0.418 | -2.87 |
| | | >0 | 13 | 1.12 | 0.035 | 0.371 | -0.418 | -2.87 |

Table 4: Default coefficients for model for initiation of "wide" structural cracking

| Pavement type | Surface material | HSOLD | $a_5$ | $a_6$ | $a_7$ |
|---|---|---|---|---|---|
| Asphalt mix on granular base | All | 0 | 2.46 | 0.93 | 0 |
| | All except cold mix | >0 | 2.04 | 0.98 | 0 |
| | Cold mix | >0 | 0.70 | 1.65 | 0 |
| Asphalt mix on asphalt base | All | 0 | 2.46 | 0.93 | 0 |
| | | >0 | 2.04 | 0.98 | 0 |
| Asphalt mix on asphalt pavement | All | >0 | 2.04 | 0.98 | 0 |
| Asphalt mix on stabilized base | All | 0 | 1.46 | 0.98 | 0 |
| | | >0 | 0 | 1.78 | 0 |
| Surface treatment on granular base | All | 0 | 2.66 | 0.88 | 1.16 |
| | All except Slurry and Cape Seal | >0 | 1.85 | 1.00 | 0 |
| | Slurry Seal and Cape Seal | >0 | 0.70 | 1.65 | 0 |
| Surface treatment on asphalt base | All | 0 | 2.66 | 0.88 | 1.16 |
| | All except Slurry and Cape Seal | >0 | 1.85 | 1.00 | 0 |
| | Slurry Seal and Cape Seal | >0 | 0.70 | 1.65 | 0 |
| Surface treatment on asphalt pavement | All | >0 | 1.85 | 1.00 | 0 |
| Surface treatment on stabilized base | All | 0 | 1.46 | 0.98 | 0 |
| | | >0 | 0 | 1.78 | 0 |

## 1.2 Progression of structural cracking

The HDM-4 EPFs for predicting the progression of structural cracking are based on time-based models (Paterson, 1987), used in HDM-III. All models are incremental where progression rate depends on the condition at the beginning of analysis period (typically one year).

The following parameters are used in the HDM-4 models for progression of "all" and "wide" structural cracking:

dACA – incremental change in area of "all" structural cracking during the analysis year (%)

dACW – incremental change in area of "wide" structural cracking during the analysis year (%)

$ACA_a$ – area of "all" structural cracking at the start of the analysis year

$ACW_a$ – area of "wide" structural cracking at the start of the analysis year

$\delta t_A$ – fraction of analysis year in which "all" structural cracking progression applies

$\delta t_W$ – fraction of analysis year in which "wide" structural cracking progression applies

AGE2 – pavement surface age since last reseal, overlay, reconstruction, or new construction (years)

ICA – time to initiation of "all" structural cracking (years)

ICW – time to initiation of "wide" structural cracking (years)

$K_{cpa}$ – calibration factor for progression of "all" structural cracking

$K_{cpw}$ – calibration factor for progression of "wide" structural cracking

CRP – retardation of cracking progression due to preventative treatment, given by
CRP = 1 – 0.12 CRT

The general form for the progression of "all" structural cracking is given by equation 30.

$$dACA = K_{cpa} \cdot \frac{CRP}{CDS} \cdot Z_A \cdot \left[ \left( Z_A \cdot a_o \cdot a_1 \cdot \delta t_A + SCA^{a_1} \right)^{1/a_1} - SCA \right] \qquad \text{Equation 30}$$

Progression of "all" structural cracking happens when the area of "all" structural cracking at the beginning of year is greater than 0 ($ACA_a > 0$) or fraction of year when "all" structural cracking progression applies is greater than 0 ($\delta t_A > 0$).

If $ACA_a > 0$, then $\delta t_A = 1$, otherwise $\delta t_A = MAX\{0, MIN[(AGE2 - ICA), 1]\}$.

If $ACA_a > 50$, then $Z_A = -1$, otherwise $Z_A = 1$.

The following assumptions are used in Equation 30:

$ACA_a$ = MAX ($ACA_a$, 0.5)

SCA = MIN ($ACA_a$, (100-$ACA_a$))

If $Y = Z_A \cdot a_o \cdot a_1 \cdot \delta t_A + SCA^{a_1}$, then:

If Y < 0, then $dACA = K_{cpa} \cdot \frac{CRP}{CDS} \cdot (100 - SCA)$ Equation 31

If Y ≥ 0, then $dACA = K_{cpa} \cdot \frac{CRP}{CDS} \cdot Z_A \cdot \left( Y^{1/a_1} - SCA \right)$ Equation 32

If the area of "all" structural cracking at the beginning of year is lower or equal to 50 % ($ACA \le 50$) and at the end of year is greater than 50 % ($ACA_a + \Delta ACA > 50$), then

$$dACA = K_{cpa} \cdot \frac{CRP}{CDS} \cdot \left( 100 \cdot c_1^{1/a_1} - ACA_a \right) \qquad \text{Equation 33}$$

where:

$c_1 = MAX\left[ \left( 2 \cdot 50^{a_1} - SCA^{a_1} - a_o \cdot a_1 \cdot \delta t_A \right), 0 \right]$.

The general form for the progression of "wide" structural cracking is given by equation 34.

$$dACW = K_{cpw} \cdot \frac{CRP}{CDS} \cdot Z_W \cdot \left[ \left( Z_W \cdot a_o \cdot a_1 \cdot \delta t_W + SCW^{a_1} \right)^{1/a_1} - SCW \right] \qquad \text{Equation 34}$$

where:

*dACW = MIN ($ACA_a$+dACA-$ACW_a$, dACW).*

Progression of "wide" structural cracking commences when fraction of year when "wide" structural cracking progression applies is greater than 0 ($\delta t_W > 0$) or the area of "wide" structural cracking at the beginning of year is greater than 0 ($ACW_a > 0$) or

If $ACW_a > 0$, then $\delta t_W = 1$, otherwise $\delta t_W = MAX\{0, MIN[(AGE2 - ICW), 1]\}$.

The initiation of "wide" structural cracking is constrained so that it does not start before the area of "all" structural cracking exceeds 5 % ($ACA_a > 5$):

If $ACA_a \le 5$ *and* $ACW_a \le 0.5$ *and* $\delta t_W > 0$, then $\delta t_W = 0$.

If $ACA_a > 50$ *and* $ACA_W > 50$ , then $Z_W = -1$, otherwise $Z_W = 1$.

The following assumptions are used in Equation 34:

$ACW_a$ = MAX ($ACW_a$, 0.5)

SCW = MIN ($ACW_a$, (100-$ACW_a$))

If $Y = Z_W \cdot a_o \cdot a_1 \cdot \delta t_W + SCW^{a_1}$, then:

If Y < 0, then $dACW = K_{cpw} \cdot \frac{CRP}{CDS} \cdot MIN\left[ (ACA_a + dACA - ACW_a), (100 - ACW_a) \right]$ Equation 35

If Y ≥ 0, then $dACW = K_{cpw} \cdot \frac{CRP}{CDS} \cdot MIN\left[\left(ACA_a + dACA - ACW_a\right), Z_W\left(Y^{1/a_1} - SCW\right)\right]$ Equation 36

If the area of “wide” structural cracking at the beginning of year is lower or equal to 50 % (*ACW* ≤ *50*) and at the end of year is greater than 50 % ($ACW_a$ + $\Delta ACW > 50$), then

$$dACA = K_{cpa} \cdot \frac{CRP}{CDS} \cdot MIN\left[\left(ACA_a + dACA - ACW_a\right), \left(100 - c_1^{1/a_1} - ACW_a\right)\right] \quad \text{Equation 37}$$

where:

$$c_1 = MAX\left[\left(2 \cdot 50^{a_1} - SCW^{a_1} - a_o \cdot a_1 \cdot \delta t_W\right), 0\right].$$

Table 5: Default coefficients for models for progression of “all” and “wide” structural cracking

| Pavement type | Surface material | HSOLD | “All” cracking | | “Wide” cracking | |
|---|---|---|---|---|---|---|
| | | | $a_o$ | $a_1$ | $a_o$ | $a_1$ |
| Asphalt mix on granular base | All | 0 | 1.84 | 0.45 | 2.94 | 0.56 |
| | All except cold mix | >0 | 1.07 | 0.28 | 2.58 | 0.45 |
| | Cold mix | >0 | 2.41 | 0.34 | 3.40 | 0.35 |
| Asphalt mix on asphalt base | All | 0 | 1.84 | 0.45 | 2.94 | 0.56 |
| | | >0 | 1.07 | 0.28 | 2.58 | 0.45 |
| Asphalt mix on asphalt pavement | All | >0 | 1.07 | 0.28 | 2.58 | 0.45 |
| Asphalt mix on stabilized base | All | 0 | 2.13 | 0.35 | 3.67 | 0.38 |
| | | >0 | 2.13 | 0.35 | 3.67 | 0.38 |
| Surface treatment on granular base | All | 0 | 1.76 | 0.32 | 2.50 | 0.25 |
| | | >0 | 2.41 | 0.34 | 3.40 | 0.35 |
| Surface treatment on asphalt base | All | 0 | 1.76 | 0.32 | 2.50 | 0.25 |
| | All except Slurry and Cape Seal | >0 | 2.41 | 0.34 | 3.40 | 0.35 |
| | Slurry Seal and Cape Seal | >0 | 1.07 | 0.28 | 2.58 | 0.45 |
| Surface treatment on asphalt pavement | All | >0 | 2.41 | 0.34 | 3.40 | 0.35 |
| Surface treatment on stabilized base | All | 0 | 2.13 | 0.35 | 3.67 | 0.38 |
| | | >0 | 2.41 | 0.34 | 3.40 | 0.35 |

In addition to time-based cracking progression model the use of traffic-based model was proposed to be incorporated in HDM-4, but due to apparent anomalies in the model the original time-based models are still used. More details on the proposed modified traffic-based model for structural cracking can be found in Morosiuk et al. (2001).

## 1.3 Reflection cracking

The existing models for reflection cracking are based on extensive study performed in Malaysia that showed that reflection cracking depends on traffic loading, existing structural strength and surface condition. However, no models for reflection cracking have been included in HDM-4 yet, since no studies were found that would isolate climatic variables, in particular the effects of the daily temperature range (Morosiuk et al., 2001).

The relationship for predicting the time to initiation of reflective cracking, based on Malaysia study, depending on thickness of the overlay and the pavement deflection before overlay is:

$$ICF = K_{cif} \cdot \frac{a_o}{ADH} \cdot DEF^{a_1} \cdot \left(1 - \frac{min\left[HSNEW, \left(a_2 - 1\right)\right]}{a}\right)^{a_3} \quad \text{Equation 38}$$

where:

ICF – time to initiation of reflection cracking (years)

ADH – average daily number of heavy vehicles in both directions

DEF – Benkelman beam deflection (mm)

HSNEW – thickness of most recent surfacing (mm)

$K_{cif}$ – calibration factor for initiation of reflection cracking.

The default values of model coefficients for initiation of reflection cracking are provided in Table 7.

Table 6: Coefficient values for initiation of reflection cracking

| Pavement type | Model coefficients | | | |
|---|---|---|---|---|
| | **$a_o$** | **$a_1$** | **$a_2$** | **$a_3$** |
| All | 685 | -0.5 | 200 | -2.0 |

The model for predicting the rate of progression of reflection cracking is:

$$dACF = K_{cpf} \cdot a_o \cdot ADH \cdot DEF^{a_1} \cdot max\left[0{,}1 - \frac{HSNEW}{a_2}\right]^{a_3} \cdot \delta t_F \quad \text{Equation 39}$$

and

$$ACF_b = min[(ACF_a + dACF), PCRA] \quad \text{Equation 40}$$

where:

dACF - incremental change in area of reflection cracking during analysis year, (% of total carriageway area)

$ACF_a$ – area of reflection cracking at start of analysis year, (% of total carriageway area)

$ACF_b$ – area of reflection cracking at end of analysis year, (% of total carriageway area)

PCRA – area of cracking before latest reseal or overlay, (% of total carriageway area)

$\delta t_F$ – fraction of analysis year in which reflection cracking progression applies

$K_{cpf}$ – calibration factor for progression of reflection cracking.

The default values of model coefficients for progression of reflection cracking are provided in Table 8. The reflection cracking model has been derived from observations of previous wide cracking reflecting through an overlay.

Table 7: Coefficient values for progression of reflection cracking

| Pavement type | Model coefficients | | | |
|---|---|---|---|---|
| | **$a_o$** | **$a_1$** | **$a_2$** | **$a_3$** |
| All | 0.0182 | 0.5 | 200 | 2.0 |

## 1.4 Transverse thermal cracking

Models for transverse thermal cracking have been introduced in HDM-4. The cracking intensity is modelled as the number of cracks per km.

### 1.4.1 Initiation of transverse thermal cracking

Two different models are defined for initiation of transverse thermal cracking. One is made for original surfacing, while another is made for overlays and reseals:

- original surfacing (HSOLD = 0)

  $$ICT = K_{cit} \cdot MAX[a_o, CDS \cdot CCT] \quad \text{Equation 41}$$

- overlays and reseals (HSOLD > 0)

$$ICT = K_{cit} \cdot MAX\left[a_o, CDS \cdot \left(CCT + a_1 + a_2 \cdot HSNEW\right)\right]$$ Equation 42

### 1.4.2 Progression of transverse thermal cracking

Progression of transverse thermal cracking commences when $\delta t_T > 0$

Separate models are also provided for progression of transverse thermal cracking. Cracking intensity is modelled as number of cracks per kilometre. Transverse thermal crack is assumed to traverse the full width of carriageway.

- original surfacing (HSOLD = 0)

$$dNCT = K_{cpt} \cdot \frac{1}{CDS} \cdot MAX\left\{0, MIN\left[\left(NCT_{eq} - NCT_a\right), \frac{2 \cdot NCT_{eq} \cdot \left(AGE3 - ICT - 0.5\right)}{T_{eq}^2}\right]\right\} \cdot \delta t_T$$ Eq. 43

- overlays and reseals (HSOLD > 0)

$$dNCT = K_{cpt} \cdot \frac{1}{CDS} \cdot MIN\left\{\left(NCT_{eq} - NCT_a\right), MAX\left[\begin{array}{l} MIN\left(a_o, PNCT, \left(PNCT - NCT_a\right)\right), \\ \dfrac{2 \cdot NCT_{eq} \cdot \left(AGE3 - ICT - 0.5\right)}{T_{eq}^2}, 0 \end{array}\right]\right\} \cdot \delta t_T$$ Eq. 44

The distressed area covered by transverse thermal cracks is obtained by equation 45.

$$dACT = \frac{dNCT}{20}$$ Equation 45

where:

ICT – time to initiation of transverse thermal cracks (years)

dNCT – incremental change in number of transverse thermal cracks during the analysis year ($n^o$/year)

CDS – construction defects indicator for bituminous surfacings

dACT – incremental change in area of transversal thermal cracking during the analysis year (% of total carriageway area)

CCT – coefficient of thermal cracking, given in the following table:

| Climate zone | Tropical | Sub-tropical | | Temperate | |
|---|---|---|---|---|---|
| | | Hot | Cool | Cool | Freeze |
| Arid | 100 | 5 | 100 | 100 | 2 |
| Semi-arid | 100 | 8 | 100 | 100 | 2 |
| Sub-humid | 100 | 100 | 100 | 100 | 1 |
| Humid | 100 | 100 | 100 | 100 | 1 |
| Per-humid | 100 | 100 | 100 | | |

PNCT – number of transverse thermal cracks before latest overlay of reseal ($n^o$/year)

$NCT_a$ – number of (reflected) transverse thermal cracks at the start of the analysis year ($n^o$/year)

$NCT_{eq}$ – maximum number of thermal cracks ($n^o$/year), given in the following table:

| Climate zone | Tropical | Sub-tropical | | Temperate | |
|---|---|---|---|---|---|
| | | Hot | Cool | Cool | Freeze |
| $NCT_{eq}$ | 0 | 100 | 0 | 0 | 20 |
| $T_{eq}$ | 50 | 7 | 50 | 50 | 7 |

$T_{eq}$ – time since initiation to reach maximum number of thermal cracks (years)

HSNEW – thickness of the most recent surfacing (mm)

$K_{cit}$ – calibration factor for initiation of transverse thermal cracking

$K_{cpt}$ – calibration factor for progression of transverse thermal cracking

The default coefficient values for transverse thermal cracking are provided in Table 9.

Table 8: Default coefficients for models for thermal transverse cracking

| Pavement type | Innovation | | | Progression |
|---|---|---|---|---|
| | $a_o$ | $a_1$ | $a_2$ | $a_o$ |
| All pavement types except surface treatment on granular or stabilized base | 1.0 | -1.0 | 0.02 | 0.25 |
| Surface treatment on granular or granular base | 100 | -1.0 | 0.02 | 0.25 |