Characterisation of the climatic temperature variations in the design of rigid pavements

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ABSTRACT
The design of rigid pavements in Austria is currently based on a catalogue of standard structures, which can be chosen depending on traffic-related parameters. This approach does not allow the consideration of real material characteristics, detailed traffic load information, local climate conditions, and other boundary conditions. To overcome these limitations, a new mechanistic-empirical pavement design method for rigid pavements (MEPDR) has been developed. The focus of this paper is laid on the proper characterisation of climatic boundary conditions and their impact on the design results. To simulate the actual temperature distribution within a concrete slab, a temperature prediction model was proposed and validated with measured weather data. Furthermore, representative temperature gradients were established using the proposed model and surface temperature data from measuring stations distributed over the Austrian highway pavement network. Additionally, the effect of these temperature gradients on the design life of a typical pavement structure was demonstrated using the MEPDR.

1. Introduction
The old Austrian design method for rigid pavements RVS 03.08.63 (FSV 2016) defines standard pavement structures for different load classes in a design catalogue. The method is based on semi-analytical models, using physical and mechanical principles to describe the reaction of a rigid pavement to external loads. However, this method has some limitations regarding the consideration of real material characteristics, detailed load information and various climatic boundary conditions. The drawbacks of the design process lead to ticker and less economic pavement structures. To resolve these limitations, a new mechanistic-empirical pavement design method for rigid pavements (MEPDR) has been developed in Austria (Eberhardsteiner et al. 2016, 2018; FSV 2020). The new Austrian MEPDR method considers several input parameters and allows for the prediction of service life or estimation of allowable load cycles until failure. Moreover, the design method offers more realistic modelling of traffic loads by incorporating actual data of traffic volume, vehicle gross weight and axle load distribution, so that traffic loads can be taken into account as realistic as possible. It also considers the load transfer in transverse joints by using a dowel effectiveness number (DEN) and the actual concrete material behaviour by using experimental results of tensile bending tests or splitting tensile tests. Another important factors affecting the pavement performance are the climatic boundary conditions. The subgrade bearing capacity is strongly influenced by local climatic and hydrological conditions. Hence, four periods during one year with varying subgrade modulus representative for bearing capacities of soils are considered in Austria. Additionally, the temperature distribution within the concrete slab (temperature difference between the top and the bottom of the slab) can force the slab to warp upward or downward, which will affect the pavement’s performance under traffic load. Therefore, the fatigue behaviour of concrete is described by Smith’s criterion (Eisenmann 1979), which takes into account traffic and curling stresses.

To establish representative temperature gradients for the estimation of curling stresses and to account for seasonal and local temperature variations in the pavement design, the temperature distribution in rigid pavements has been investigated using actual weather data (Eberhardsteiner et al. 2016; Bayraktarova et al. 2017). Therefore, a temperature prediction model based on the Forward Time Centered Space (FTCS) Finite Difference Method (FDM) was implemented in a tool and validated using actual temperature measurements. Furthermore, the impact of the temperature gradients on the design life of rigid pavements was demonstrated using the Austrian MEPDR (FSV 2020).

2. Literature review
The investigation of temperature and moisture variations in rigid pavement structures is an important task, as these fluctuations generate volumetric changes of the concrete slabs and induce curling or warping stresses. Consequently, these stresses cause upward or downward slab curvature, which, in combination with the traffic loading, can lead to fatigue damage. To create a proper perspective, the following
section reviews state of the art models for the prediction of temperature profiles in pavements as well as different approaches for the calculation of curling stresses in rigid pavements.

### 2.1. Prediction of temperature profiles

As temperature variations play a crucial role in the development of critical stress and distress, temperature distribution in a pavement is considered in most Mechanistic Empirical Pavement Design Models (NCHRP 2004). Wang et al. (2009) summarised three types of solution methods to predict pavement temperature profiles: statistics-based, analytical and numerical approaches. A comprehensive review of existing models for predicting pavement temperature, including empirical, analytical, and numerical models is given in Chen et al. (2019).

Statistics-based methods use simplified correlations between pavement temperature and environmental factors. These methods can be categorised in linear regression models, nonlinear regression models, and neural network models. The linear regression models calculate extreme or average temperature in pavement at a prescribed depth within an original sample database and should not be extrapolated outside the original solution database (Sherif and Hassan 2004; Diefenderfer et al. 2002; Krsmanc et al. 2013). The nonlinear regression models are more complex and allow the prediction of pavement temperature with time and depth (Chandrappa and Biligiri 2015). The prediction of pavement temperature with neural network models is based on machine learning using extensive data.

Analytical and numerical models can be formulated by solving the partial differential equation (PDE) for heat conduction under given boundary conditions to simulate time-dependent temperature profiles. These methods use pavement geometry, climatic and thermal parameters of the layered materials.

Barber (1957) was among the first researchers to employ an analytical solution to the heat conduction problem for prediction of pavement temperatures. In his solution, a thermal diffusion theory was applied to a semi-infinite pavement in contact with air temperature. However, the model operates with total daily radiation instead of hourly radiation and is therefore inaccurate.

Dempsey and Thompson (1970) developed an approach that uses 1D heat transfer model and an explicit finite difference method (FDM) to simulate temperature profiles as a function of time. His approach was implemented in the Mechanistic Empirical Pavement Design Guide (MEPDG) (NCHRP 2004) as part of the Enhanced Integrated Climatic Model (EICM). EICM considers hourly climatic data that include air temperature, wind speed, relative humidity, precipitation, percentage of sunshine and location of ground water to calculate the temperature and moisture profile, frost heave and other climate-related phenomena.

Later, Solaimanian and Kennedy (1993) considered energy balance at the pavement surface and developed an equation for calculating the maximum pavement surface temperature. Wang proposed analytical models developed by solving the PDE for heat conduction using the separation of variables method (Wang 2012; Wang and Roesler 2014) and eigenfunction expansion technique (Wang 2016). Qin (2016) introduced a prediction model for pavement surface temperature based on the assumption that the daily surface temperature follows sinusoidal waves.

Many researches (Hermansson 2002; Yavuzturk et al. 2005; Huang et al. 2017) developed numerical models with FDM for prediction of temperature profiles, that use solar radiation, long-wave radiation and convection to define the surface boundary condition.

Besides FDM, the Finite Volume Method (FVM) and the Finite Element Method (FEM) can also be applied to predict the pavement temperature. These models are appropriate for complex surface boundary conditions because the PDE of heat conduction is solved over the nodes or volumes numerically. In the literature, there are plenty of FEM-models for predicting temperature profiles (McCullough and Rasmussen 1999; Minhoto et al. 2005; Jeong and Zollinger 2006; Teltayev and Koblanbek 2015). Alavi et al. (2014) proposed a temperature prediction model using the finite control volume approach (FCVM) with fully implicit scheme, that solved some of the known limitations in current temperature prediction models.

### 2.2. Calculation of the curling stresses in rigid pavements

The most common way to consider temperature distribution in the estimation of curling stresses is to use linear temperature gradients $\Delta T$, even though the nonlinearity of the temperature distribution throughout the slab has long been recognised (Choubane and Tia 1995; Mohamed and Hansen 1996; Siddique et al. 2005). A temperature gradient arises from the temperature difference on the top and on the bottom of the concrete slab divided by its thickness.

The Eisenmann ‘s and Westergaard–Bradbury’s approximate analytical solutions are the most commonly used and are based on Kirchhoff’s plate theory. They are based, for simplicity, on the assumption that the temperature variation in the concrete slab from top to bottom is linear.

Bradbury (1938) solution assumes linear temperature differential, Winkler foundation and full bonding between the slab and the foundation. The curling stress can be estimated for any pavement structure by taking into account the effective thickness of the slab with the corresponding radius of relative stiffness values.

Eisenmann’s solution (Eisenmann 1979) is based on the concept of a critical slab length $L_{\text{crit}}$, which is defined as the length where a concrete slab, equally heated at the surface, only touches the substructure at the four corners and in its centre. Three cases are considered depending on the actual length of the slab $L$. When $L > 1.1 \cdot L_{\text{crit}}$, the stress in the centre of the slab can be determined according to Kirchhoff’s plate theory. When $L \approx L_{\text{crit}}$, the stress increases by 20% and when $L < 0.9 \cdot L_{\text{crit}}$, the stress reduces depending on the actual length of the slab $L$. In the early nineties, it was found that the Eisenmann’s model for calculation of the curling stresses is not
accurate for estimation of the critical stresses resulting from great temperature gradients. Houben (1992) revised Eisenmann’s model and introduced a coefficient C for the support length for different temperature gradients.

While the Bradbury (1938) solution assumes full bonding between the slab and the foundation, Eisenmann’s solution (Eisenmann 1979) considers partial foundation contact, which corresponds to the real interface behaviour.

Thomlinson (1940) was the first researcher who developed a theory, accounting for the nonlinear temperature distribution in the estimation of curling stresses. According to him, the total curling stress can be divided into three components (Figure 1). The first uniform axial component causes the slab to expand or contract. The second is the linear component due to temperature changes that causes bending (upward or downward) and the third is the nonlinear component, which generates internal forces as a result from the temperature loading and the restriction of the deformations.

Another concept to estimate a temperature gradient is to convert the nonlinear temperature profile to an equivalent temperature difference $\Delta T_{eq}$ (Thomlinson 1940; Choubane and Tia 1995; Mohamed and Hansen 1996; Ioannides and Khazanovich 1998). This equivalent temperature difference would induce the same slab curvature as the original nonlinear temperature profile. Ioannides and Khazanovich (1998) established this concept of a non-uniform, multi-layered slab and implemented it into finite-element code ILSL2 and MEPDG (NCHRP 2004). The estimation of critical stresses in the MEPDG is based on Neural Networks matching prediction of the Westergaard solution and considers a variety of design factors (Khazanovich et al. 2001): surface and base layer parameters, interface conditions between the concrete slab and the base layer, shoulder type, slab geometry, subgrade properties, axle type, weight, and position and the effect of voids under pavements. The MEPDG also considers the effect of the built-in temperature gradient or construction temperature gradient. Eisenmann and Leykauf (1990) found out that constructing concrete pavements in hot days induces a large positive temperature gradient during the hardening, that is the cause for early upward curling. Yu et al. (1998) studied the effects of the built-in temperature curling on the critical wheel load stress. Beckemeyer et al. (2002) demonstrated the influence of the base layer type on the magnitude of built-in curling. Later Rao and Roesler (2005) introduced the effective built-in temperature difference, combining the effects of temperature, shrinkage, and creep.

Hiller and Roesler (2010) proposed a nonlinear area method called NOLA that captures the effect of temperature nonlinearity. Lately, a strain-based equivalent temperature gradient $\Delta T_{strain}$ was proposed by Gao et al. (2017). They found that $\Delta T_{strain}$ is the smallest among $\Delta T$ and $\Delta T_{eq}$ because of the added effect of moisture gradient which causes upward warping equivalent to a negative temperature gradient. It was recommended to use $\Delta T'$ for design purposes because of the conservative, calculated curling that a linear temperature gradient provides.

The curling stresses can also be calculated using a numerical method, although they are time-consuming, costly and require an understanding of the complexity of the problem.

Furthermore, comparative studies (Eberhardsteiner et al. 2016; Caliendo and Parisi 2010) of the calculated curling stresses with Westergaard–Bradbury’s model, Eisenman’s model and a 3D FE model (ABAQUS/Standard 2014) have shown a good agreement between the Eisenmann’s and the FE solutions at the slab edge (Table 1). In consequence of these results and for simplicity, the revised Eisenmann–Houben model was implemented in the new Austrian mechanistic pavement design method for rigid pavements.

3. Simulation of temperature distribution

As aforementioned, the FDM can be employed to solve the differential equation and predict pavement temperature distribution. The following section presents a numerical solution of the Fourier heat equation with a Finite Difference Method and describes the input parameters and demonstrates that the model is capable of reproducing measured temperature data.

3.1. Finite difference method (FDM)

The Fourier equation (1) for the one-dimensional case describes a transient heat transfer or the change in temperature $T$ [K], in a solid environment, without any internal heat source as a function of time $t$ [s] and depth $x$ (location coordinate) [m].

$$\frac{\partial T}{\partial t} = \alpha \cdot \frac{\partial^2 T}{\partial x^2}$$  (1)

Where the thermal diffusivity $\alpha$ describes the rate of thermal

![Figure 1. Components of the nonlinear temperature-induced deformation (Thomlinson 1940; Eisenmann 1979).](Image)
Percentage

| Location | Bradbury | Eisenmann | Eisenmann-Houben | FE-Abaqus |
|----------|----------|-----------|------------------|-----------|
| Slab edge | 3.45     | 1.31      | 1.34             | 1.47      |
| Percentage difference | 157%     | -2%       | -                | 10%       |

Table 1. Maximum tensile stresses according to different models (Eberhardsteiner et al. 2016).

energy spreading through the material. By this means, the thermal diffusivity can be calculated as a quotient of thermal conductivity and heat capacity. The thermal conductivity $\lambda$ [W·m$^{-1}$·K$^{-1}$] describes the ability of a stratum with density $\rho$ [kg·m$^{-3}$] to conduct and the specific heat capacity $c$ [J·kg$^{-1}$·K$^{-1}$] describes the ability to store thermal energy.

$$\alpha = \frac{\lambda}{c \cdot \rho}$$

(2)

For numerical evaluation, the exact solution is approximated by finite time intervals $\Delta t$ and thickness intervals $\Delta x$. Hereby, the temperature change $\Delta T_f$ occurring during the time interval $\Delta t$ can be written as

$$\Delta T_f = \alpha \cdot \frac{\Delta t}{\Delta x^2} \cdot \Delta^2 T_x$$

(3)

where $\Delta T_f$ describes the change of temperature by time, $\Delta T_x$ the change by position and $\Delta^2 T_x$ the difference between two successive differences. Now, the indices $k$ for the time step and $n$ for the position step are introduced, as is shown in Figure 2.

With this in mind, the differences between two nodes can be written as

$$\Delta T_f = T_{n,k+1} - T_{n,k}$$

(4)

$$\Delta T_x = T_{n+1,k} - T_{n,k}$$

(5)

$$\Delta^2 T_x = (T_{n+1,k} - T_{n,k}) - (T_{n,k} - T_{n-1,k})$$

$$= T_{n+1,k} - 2 \cdot T_{n,k} + T_{n-1,k}$$

(6)

Inserting Equations (4) and (6) in Equation (3) leads to the explicit FTCS (Forward Time Centered Space) FDM in

$$T_{n-1,k-1}^{l} \quad T_{n-1,k}^{l} \quad T_{n-1,k+1}^{l}$$

$$T_{n,k-1}^{l} \quad T_{n,k}^{l} \quad T_{n,k+1}^{l}$$

$$T_{n+1,k-1}^{l} \quad T_{n+1,k}^{l} \quad T_{n+1,k+1}^{l}$$

Equation (7).

$$T_{n,k+1} = \alpha \cdot \frac{\Delta t}{\Delta x^2} \cdot (T_{n+1,k} - 2 \cdot T_{n,k} + T_{n-1,k}) + T_{n,k}$$

(7)

The explicit FTCS scheme provides, as in Krebs and Böinger (1981) and Dempsey et al. (1985) shown, reliable and straightforward results. The scheme is numerically stable when Equation (8) is satisfied.

$$\alpha \cdot \frac{\Delta t}{\Delta x^2} \leq \frac{1}{2}$$

(8)

The explicit scheme needs to be adapted in order to solve the equation with a mixed condition, where the surrounding temperature is given while the surface temperature is unknown. Under the assumption that the heat flow only depends on the heat transfer between the surrounding air and the surface of the body, the heat balance equation can be written as

$$\lambda \cdot \frac{dT}{dx} = h \cdot (T_A - T_S).$$

(9)

Where the left-hand side term represents the heat conduction and the right-hand side, the heat transfer between the air and the surface. The heat transfer describes the amount of heat exchanged per area and time between the air and the ground surface. The heat transfer is formulated as a function of the wind speed $u$ [m·s$^{-1}$]. Krebs and Böinger (1981) have derived an empirical formulation (11) from measurements on the surface.

$$h(t) = c_1 + c_2 \cdot u(t)^{c_3}$$

(10)

$$h(t) = 10 \cdot (0.174 + 0.941 \cdot u(t)^{0.366})$$

(11)

Where $u(t)$ [m·s$^{-1}$] represents the wind speed in 10 m height and $c_1$, $c_2$ and $c_3$ are empirically determined constants. Now the temperatures at position $n = 1$ can be written as

$$T_{1,k+1} = \alpha \cdot \frac{\Delta t}{\Delta x^2} \cdot \left[ T_{2,k} - 2 \cdot T_{1,k} - \frac{T_A}{\lambda} \cdot \left( \frac{\lambda}{h_k} - \frac{\Delta x}{2} \right) + T_{A,k} \right] + T_{1,k}$$

(12)

and the temperatures at the surface $n = 0$ as

$$T_{S,k+1} = -\frac{T_{A,k+1} - T_{1,k+1}}{\lambda} \cdot \frac{\lambda}{h_{k+1}} + T_{L,k+1}. $$

(13)

For the temperatures at positions $n > 1$, the application remains unchanged using Equation (7). To describe the influence of the radiation balance, the reduced heat balance Equation (9) needs to be supplemented by the solar radiation...
where 0 ≤ k ≤ ω and S = 0 ≤ n ≤ v. For numerical computation and implementation the input data are divided into two groups (data frames), the time-dependent climate-data, which defines the time step Δt, and the location-dependent layer-data. With the wind speed u the heat transfer coefficient for every time step k h₀ = hₜ [W·m⁻²·K⁻¹] is given by Equation (11). Further, this leads to the virtual air temperature Tₐ,k⁺ for every time step k with Equation (15). With the layer-data of every layer i the thermal diffusivity αᵢ is given by Equation (2). This leads, together with the time step size Δt from the climate-data input, to a stable layer thickness Δxᵢ. For accurate simulations each layer i needs to be divided into j sub-layers with Δxᵢ > Δxᵢ, where Δxᵢ is the thickness of the sub-layer. By setting the sublayer thickness, the position of the interfaces is defined and the initial temperature profile can be chosen. For long-term simulations the failure of the chosen initial temperature will be neglected. Now both data frames are preset for the pre-described simulation.

3.3. Input data

Necessary input parameters to compute the temperature profiles are: (i) air temperature (ambient temperature), (ii) wind speed, (iii) solar radiation, (iv) vapor pressure, (v) sub-grade temperature, (vi) initial temperature profile, (vii) surface and can be estimated using Equation (20).

\[ Q_{\text{ns}} = (1 - \bar{\alpha}) \cdot Q_{\text{solar}} \]  

(20)

Where \( \bar{\alpha} \) is the albedo of the surface, representing the percentage of reflected solar radiation and \( Q_{\text{solar}} \) [W·m⁻²] is the global solar radiation (measured by pyranometer). The long-wave radiation \( Q_{\text{nl}} \) (Equation (21)) or the net thermal radiation is described by the outgoing long-wave radiation \( Q_{\text{L}} \) [W·m⁻²] which follows the Stefan–Boltzmann law and by the atmospheric downward long-wave radiation \( Q_{\text{A}} \) [W·m⁻²] (Solaimanian and Kennedy 1993)

\[ Q_{\text{nl}} = Q_{R} - Q_{\text{A}} \]  

(21)

\[ Q_{\text{A}} = \varepsilon \sigma T_{\text{A}}^4 - \varepsilon_a \sigma T_{\text{A}}^4 \]  

(22)

with the emission coefficient \( \varepsilon \), the Stefan–Boltzmann constant \( \sigma = 5.67 \times 10^{-8} \) [W·m⁻²·K⁻⁴], the surface temperature \( T_{\text{A}} \) [K], a factor accounting for pavement surface absorptivity for long-wave radiation \( \varepsilon_a \) and the air temperature in 2 m height \( T_{\text{A}} \) [K]. According to Geiger (1959) and Linke and Möller (1974) the factor \( \varepsilon_a \) can be calculated using the following equation

\[ \varepsilon_a = a - b \cdot 10^{-c \cdot e} \]  

(23)

where \( e \) is the vapor pressure [h·Pa] and \( a \) is a constant equals 0.79, \( b \) is a constant equals 0.174 and \( c \) is a constant equals 0.095 [h·Pa⁻¹] (constants according to Linke and Möller 1974).

3.2. Implementation

Figure 4 shows the flowchart of the FTCS scheme with a mixed boundary condition, with 0 ≤ k ≤ ω and S = 0 ≤ n ≤ v. For numerical computation and implementation the input data are divided into two groups (data frames), the time-dependent climate-data, which defines the time step Δt, and the location-dependent layer-data. With the wind speed u the heat transfer coefficient for every time step k h₀ = hₜ [W·m⁻²·K⁻¹] is given by Equation (11). Further, this leads to the virtual air temperature Tₐ,k⁺ for every time step k with Equation (15). With the layer-data of every layer i the thermal diffusivity αᵢ is given by Equation (2). This leads, together with the time step size Δt from the climate-data input, to a stable layer thickness Δxᵢ. For accurate simulations each layer i needs to be divided into j sub-layers with Δxᵢ > Δxᵢ, where Δxᵢ is the thickness of the sub-layer. By setting the sublayer thickness, the position of the interfaces is defined and the initial temperature profile can be chosen. For long-term simulations the failure of the chosen initial temperature will be neglected. Now both data frames are preset for the pre-described simulation.
geometry, (viii) thermophysical material parameters and (ix) pavement surface properties.

Actual hourly data of air temperature, wind velocity, solar radiation and vapor pressure are needed to define the upper boundary condition when the surface temperature is unknown (Equation (17) and Equation (18)).

As aforementioned, in Eberhardsteiner et al. (2016) and Bayraktarova et al. (2017) it was found that the solar radiation has a great impact on the surface temperature. The proposed temperature prediction model uses measured solar radiation data, which is only rarely available in Austria. However, there are some empirical equations, that either estimate the solar radiation based on the solar constant and the location of the sun or use cosine functions. Some of them also consider the effect of cloud cover by using quantified parameters. These equations are summarised in Chen et al. (2019). Most of these

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**Figure 4.** Flowchart of the FTCS scheme with a mixed boundary condition, with $0 \leq k \leq \kappa$ and $S = 0 \leq n \leq v$.

| Geometry | Thermophysical Parameters | Pavement Surface Properties |
|----------|---------------------------|----------------------------|
| Input Climate Data: |
| - air temperature $T_{A,0} - T_{A,\kappa}$ [K] |
| - wind speed $u_0 - u_\kappa$ [m/s] |
| - radiation balance Eq. (19) $Q_0 - Q_\kappa$ [W/m²] |
| - heat transfer coeff. Eq. (11) $h_0 - h_\kappa$ [W/m²K] |
| - virtual air temperature Eq. (16) $T_{A,0} - T_{A,\kappa}$ [K] |
| - surface temperature Eq. (17) $T_{S,k-1}$ [K] |
| - temperature pos. 1 Eq. (18) $T_{1,k}$ [K] |
| - temperature pos. n Eq. (7) $T_{2,k} - T_{v-1,k}$ [K] |
| - temperature 2 m Eq. (7) $T_{v,k} = T_{2m}(date_k)$ [K] |
| $k = 1$ |
| $\Delta t$ [h] |
| $\Delta z_i$ [m] |
| $\alpha_i$ [m²/h] |
| $\rho_0 - \rho_i$ [kg] |

| Input Layer Data: |
| - thermal conductivity $\lambda_0 - \lambda_i$ [W/mK] |
| - specific heat capacity $c_0 - c_i$ [J/kgK] |
| - density $\rho_0 - \rho_i$ [kg] |

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$k < \kappa$

**false**

- surface temperature Eq. (17) $T_{S,\kappa}$ [K]
equations have been derived based on statistical analysis and are database- and region-dependent. For this reason their applicability in Austria has to be examined. The pavement surface properties – albedo and emissivity have to be cautiously determined, in order to calculate the radiation balance. A summary of typical values of these parameters is given in Chen et al. (2019).

The lower boundary condition in the presented method is the ground temperature at 2 m depth. Typical ground temperatures for different climatic periods in Austria are given by Wistuba (2003) with

\[
y(x) := 1.19 \cdot 10^{-3} \cdot x^5 - 4.02 \cdot 10^{-2} \cdot x^4 + 4.3 \cdot 10^{-1} \cdot x^3 - 1.42 \cdot x^2 - 1.55 \cdot 10^{-1} \cdot x + 1.02 \cdot 10^{-4}
\]  

(24)

where \(1 \leq x \leq 12\) (Months).

Measurements of the ground temperature at a depth of 2 m at two places with various sea levels, ground conditions and climatic region in Austria show the same annual temperature profile (Wistuba 2003). Another study (Qin and Hiller 2011) confirmed that the ground temperature is strongly affected by the heat flow from the interior of the earth and for this reason exhibits fewer variations.

The chosen initial temperature profiles are based on long-term ground-temperature measurements (from 1961 to 1990) at measuring station Kremsmünster/Upper Austria. Thereby, the average ground temperature at different depths was implemented (Table 2).

### 3.4. Validation of the model

Since the FDM is a simulation with approximate approaches, a validation was carried out comparing measured temperature profiles with simulated temperature profiles. Data was collected from two measuring stations in Austria: (i) in Wopfing on a test track and (ii) on the highway A2 near Baden at km 21. The test track in Wopfing consists of 130 mm thick concrete slabs, a 150 mm thick upper base layer and a 150 mm thick lower base layer. The measured hourly data from Wopfing includes air temperature, wind velocity, relative air humidity, global solar radiation, pavement surface temperature and temperature at a depth of 130 mm. The data acquisition lasted 10 months (from 25 December 2015 to 23 October 2016). Using this data, material properties from Table 3, an albedo of 0.45 and an emission coefficient of 0.88 the temperature distribution has been simulated with the proposed model. The values of the parameters albedo and emission coefficient have been adapted to give a good correspondence between measured and simulated surface temperature as in Figures 5 and 6 shown.

Differences up to 4 K arise when comparing both temperature plots. It is obvious from Figures 5 and 6 that the temperature prediction is more accurate for the summer than for the winter. This means that different sets of the parameters, defining the heat flux at the upper boundary, has to be used depending on the weather conditions (e.g. snow, freeze and thaw, the degree of water saturation). Hermansson (2004) studied extensively this effect and developed a model that changes automatically between summer and winter values for these parameters. The change is based on air temperature and solar radiation only. An evaluation of such weather data from multiple weather stations must be performed to define such limit values for Austria. Due to lack of sufficient weather data, this was not implemented in the proposed model.

In addition, a comparison of the temperature plots at 130 m depths (Figure 7) shows differences up to 3 K, which demonstrate the accuracy of the proposed temperature prediction model. It appears that the temperature at the top nodes on the pavement surface is more affected by the ambient temperature variation than the temperature at the bottom nodes. This trend was also confirmed by Ali and Urgessa (2012).

### Table 2. Initial temperature profiles for the six climate periods at various depths (Wistuba 2003).

| Period             | Date               | Temperature (°C) at depth (cm) |
|--------------------|--------------------|--------------------------------|
|                    |                    | 2    | 10   | 20   | 30   | 50   | 100  | 200  |
| 1                  | December 16–March 15 | 2.4  | 2.7  | 2.9  | 3.0  | 3.8  | 5.0  | 7.6  |
| 2                  | March 16–May 15    | 9.6  | 9.0  | 8.4  | 7.9  | 8.0  | 7.0  | 6.3  |
| 3                  | May 16–June 15     | 17.1 | 16.2 | 15.4 | 14.7 | 14.2 | 11.8 | 8.7  |
| 4                  | June 16–September 15 | 17.6 | 17.0 | 16.7 | 16.4 | 16.1 | 14.7 | 12.3 |
| 5                  | September 16–October 15 | 12.5 | 13.7 | 13.9 | 14.1 | 14.4 | 14.6 | 14.4 |
| 6                  | October 16–December 15 | 5.8  | 6.5  | 7.1  | 7.4  | 8.2  | 9.8  | 12.1 |

### Table 3. Material properties.

| Parameter                | Units     | Concrete slab | Asphalt layer | Base layer | Ground (gravel) |
|--------------------------|-----------|---------------|---------------|------------|-----------------|
| Density                  | kg m⁻³    | 2370          | 2120          | 1800       | 2100            |
| • Dry                    |           | 2370          | 2120          | 2300       | 2100            |
| • Water-saturated        |           | 0.78          | 0.92          | 0.72       | 0.94            |
| Specific heat capacity   | kJ kg⁻¹ K⁻¹ | 0.78          | 0.92          | 1.13       | 1.80            |
| Thermal conductivity     | W m⁻¹ K⁻¹ | 1.60          | 0.70          | 0.40       | 0.50            |
| • Dry                    |           | 1.60          | 0.70          | 1.40       | 2.50            |
| • Water-saturated        |           | 1.60          | 0.70          | 1.40       | 2.50            |
| Reference                |           | FGSV (1994)   | FGSV (1994)   | VDI (2001) | VDI (2001)      |
Furthermore, to verify the accuracy of the proposed method using only the surface temperature as an input, the temperature distribution within a concrete layer on the highway A2 near Baden at km 21 was measured every hour over a one-year period at 4 depths (at 50, 100, 150 and 200 mm distance from the surface) and on the surface. The pavement structure consists of a 250 mm concrete slab, 50 mm asphalt layer and 450 mm base layer.

Figure 8 shows a very good agreement between the measured and simulated values with a maximum discrepancy of 2 K. This agreement indicates an adequate accuracy of the proposed temperature prediction model by using actual surface temperatures.

3.5. Simulations and results

Hourly surface temperature data from 16 measuring stations distributed over the Austrian rigid highway pavement network obtained from 2011 to 2015 was utilised for the simulation of the temperature distribution and the establishment of representative temperature gradients. The data was collected and provided by the Austrian highway authority ASFINAG. This was done because there is no data on countrywide solar radiation and the simulation of temperature profiles in pavements with surface temperature is more accurate.

Within this section, the results from the simulations of the temperature distribution for dry and water-saturated unbound

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**Figure 5.** Comparison of measured and predicted surface temperatures in the summer, weather data from Wopfing.

**Figure 6.** Comparison of measured and predicted surface temperatures in the winter, weather data from Wopfing.
layers with surface temperature data from the 16 above-mentioned measuring stations are discussed.

The following Figure 9 shows typical temperature distributions at various times of the day in a concrete slab with a thickness of 250 mm, obtained as a result from the simulations using the proposed temperature prediction model. A strong inward curvature of the temperature profile indicates high-temperature gradients. As in Figure 9 compared, the temperature gradients are positive during day time and negative during night time. However, in Eberhardsteiner et al. (2016) and Bayraktarova et al. (2017) was concluded that the negative temperature gradients, which occur during the night time have an insignificant impact on the maximum curling stresses and, therefore, were not considered in pavement design.

Figure 10 shows another comparison between the resulting temperature gradients estimated with thermophysical material parameters of dry and water-saturated unbound layers. The resulting temperature gradients from water-saturated unbound layers tend to be higher during the warm periods and lower during the cold periods. This can be explained by the fact that water-saturated layers have greater thermal conductivities than dry layers. This trend can also be seen in Table 4, where the 95th percentile was estimated based on the resulting temperature gradients from measuring station Inzersdorf in eastern Austria for the six climate periods during four years (2011 to 2014). Due to the lack of soil moisture data for the evaluated stations, the final value of the temperature gradient for each period is generated.
as a mean value of the temperature gradients from dry and water-saturated layers.

As the differences in the temperature gradients for each year and station were insignificant, it was concluded to summarise the resulting gradients for each climate period with the mean value of each year of each measuring station. Table 5 shows the summarised temperature gradients for the six climate periods, which were statistically analysed with the 95th, 75th and 50th percentiles. Depending on the road category, a different percentile can be chosen to account a representative proportion for the calculation of curling stresses and for the design life of the pavement structure.

### 4. Impact of the temperature gradients on the design life of the rigid pavement

#### 4.1. Design example

To analyse the impact of the chosen evaluation percentile for the representative temperature gradients on the design life of

| Year | Unbound layers | Temperature gradients $\Delta T$ (K/mm) for different climate periods |
|------|----------------|---------------------------------------------------------------|
|      |                | 1  | 2          | 3          | 4          | 5          | 6          |
| 2014 | Dry            | 0.022 | 0.063 | 0.079 | 0.067 | 0.040 | 0.020 |
|      | Water-saturated | 0.021 | 0.068 | 0.090 | 0.075 | 0.041 | 0.020 |
| 2013 | Dry            | 0.024 | 0.063 | 0.070 | 0.066 | 0.005 | 0.020 |
|      | Water-saturated | 0.022 | 0.069 | 0.078 | 0.075 | 0.006 | 0.019 |
| 2012 | Dry            | 0.020 | 0.062 | 0.064 | 0.062 | 0.045 | 0.021 |
|      | Water-saturated | 0.019 | 0.068 | 0.071 | 0.072 | 0.046 | 0.019 |
| 2011 | Dry            | 0.029 | 0.058 | 0.071 | 0.060 | 0.047 | 0.021 |
|      | Water-saturated | 0.027 | 0.064 | 0.080 | 0.068 | 0.050 | 0.018 |
rigid pavement structures, a design example was performed according to the new Austrian MEPDR method (Eberhardtsteiner et al. 2016, 2018; FSV 2020). The obtained results are compared to those resulting from the Austrian standard design catalogue given in the national standard RVS 03.08.63 (FSV 2016). For the purpose of this investigation, a typical highway pavement structure (BE 1, LK 40) was analysed, consisting of 250 mm thick concrete layer with a slab size of 5 × 5 m and a bending tensile strength of 5.27 MPa, 50 mm asphalt layer and 450 mm unbound gravel layer (Table 6). The load transfer between individual slabs is realised by standard dowels with a diameter of 25 mm, length of 500 mm and dowel spacing of 300 mm. Various bearing capacities of the base and the subgrade layers were used depending on the seasonal period (P1, P2, P3 and P4). Traffic loads are estimated using representative distributions for the occurrence of heavy goods vehicle (HGV) types, gross vehicle weights and axles loads (Eberhardtsteiner and Blab 2017). To account for seasonal temperature variations, different percentiles of the representative temperature gradients (Table 6) were used.

Considering these input parameters, plate theory according to Kirchhoff (Holller et al. 2019) is applied to determine the resulting stresses due to traffic loads for each axle of each vehicle and each subgrade period. Then, the further developed Eisenmann’s model (Houben 2009) is used for the estimation of the curling stresses for the six climate periods.

In another step, the allowable number of cycles the pavement is able to resist, $N_{res}$, reads as

$$N_{res} = \frac{1}{C_{res}}$$

(28)

For the considered road section, the number of the expected load passages $N_{imp}$ (impact) was estimated using the following equation:

$$N_{imp} = AADTT \cdot V \cdot S \cdot 365 \cdot n \cdot z$$

(29)

and assuming an annual average of the overall daily heavy traffic (AADTT) of 2000 HGVs/24 h, a factor $V$ related to the distribution of vehicles to several lanes ($V = 1$), a factor $S$ considering the loading distribution of vehicle tracks within one lane ($S = 1$), a 30-years design life $n$, an annual traffic growth of 2% and a growth factor $z$

$$z = \frac{q^n - 1}{n \cdot (q - 1)}$$

(30)

where $q = 1 + p/100$, and $p$ denotes the annual traffic growth rate in %.

Within this design process (according to the serviceability limit state), the number of load cycles the pavement is able to resist $N_{res}$ (resistance), is related to the expected number of passages during the design life, $N_{imp}$ (impact), by Equation (31)

$$\frac{N_{imp}}{N_{res}} \leq 1.$$  

(31)

### 4.2. Analysis of the results

Figure 11 shows the impact of the chosen temperature gradient percentile on the design life. Since the temperature gradients evaluated with lower percentile are low (Table 5), the allowable number of cycles the pavement is able to resist at each percentile. The grey bars in Figure 11 show the relative change of $N_{res}$ assuming that 100% corresponds to 95th percentile. $N_{res}$ rises by up to 57% when applying temperature gradients evaluated with the 50th percentile.

The predicted design lives using the new MEPDR and temperature gradients evaluated with different percentiles are

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**Table 6.** Input parameters for the design example.

| Layer               | E-Modulus (MPa) | $\nu$ | Thickness (mm) |
|---------------------|-----------------|------|----------------|
| Concrete slab       | 30,000          | 0.17 | 250            |
| Asphalt             | 3500            | 0.40 | 50             |
| Base layer          |                 |      |                |
| P1                  | 678             | 169  | 242            |
| P2                  | 169             | 242  | 339            |
| P3                  | 140             | 140  | 450            |
| P4                  | 280             | 70   | 100            |

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**Table 5.** Temperature gradients for the six climate periods, evaluated with different percentiles.

| Percentile | 1  | 2  | 3  | 4  | 5  | 6  |
|------------|----|----|----|----|----|----|
|            | 16.12–15.03 | 16.03–15.05 | 16.05–15.06 | 16.06–15.09 | 16.09–15.10 | 16.10–15.12 |
| 95th       | 0.023 | 0.070 | 0.83 | 0.076 | 0.045 | 0.024 |
| 75th       | 0.010 | 0.042 | 0.051 | 0.024 | 0.011 | 0.009 |
| 50th       | 0.002 | 0.023 | 0.030 | 0.024 | 0.011 | 0.001 |

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compared with the design life obtained from the traditional design catalogue RVS 03.08.63 (FSV 2016) in Figure 12. Applying the MEPDR results in an increase of calculated technical lifetime up to 21 years (50th percentile of temperature gradients) compared to the traditional design catalogue. These results can be explained by the fact that the design life obtained from the design catalogue has been estimated with a constant temperature gradient during the whole year. Furthermore, the detailed characterisation of the climatic boundary conditions leads to a more accurate prediction of the design life.

### 5. Conclusion

The main purpose of the current study was to develop an easily adaptable method to account for the seasonal temperature variations in the design of rigid pavements. For this reason, a temperature prediction model was implemented in a tool, validated and used to derive realistic temperature gradients representing the climatic conditions in Austria. The temperature gradients are used as input parameter for the estimation of resulting curling stresses. The proposed procedure is practical and much simpler in comparison to the procedure implemented in the MEPDG. It uses the information readily available in Austria and can be easily adapted for other locations.

The analysis of simulated temperature profiles shows that the temperature gradients estimated with data from dry unbound layers tend to be higher during the cold periods and lower during the warm periods compared to water-saturated layers. This indicates the importance of incorporating soil moisture data in the simulation of temperature distribution in pavement structures. The results of statistical
analysis of the temperature gradients allow for choosing a different temperature gradient percentile depending on the road category and on the specific local climatic conditions. The main finding from the demonstrated design example is that the more detailed the characterisation of climatic boundary conditions is, the more accurate is the prediction of the design life.

Disclosure statement
No potential conflict of interest was reported by the author(s).

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