Climate conditions and their impact on road pavement design

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ABSTRACT: The basis for determining the temperature characteristics of the pavement were the temperature measurements of multi-layer structures. For the design of pavements structures in the calculations we consider the deformation of materials for equivalent temperatures derived from actual measured temperatures. The strain and stresses in concrete slabs are important / significant temperature gradients - the temperature difference at the top and bottom surfaces. In the paper are examples of calculations and impact of standard and changing weather conditions on the dimensions of the structures and pavements lifetime.

KEY WORDS: Pavement design, stress, temperature, climate condition.

1 INTRUDUCTION

The territory of Slovakia is in Central Europe at the interface of the parts of the continent that are influenced by continental and by oceanic climates. This is reflected in the variability of weather and climate characteristics. In addition, meteorologists recognize that climate has changed in Slovakia over the past years. Since the XIX century, Slovakia has experienced a growth in temperature (annual average) of about 1.5 °C, precipitation changes, decreased relative humidity and changes in solar radiation (Lapin, 2008). Data from the Hydro-meteorological Institute and measurements presented in the study of (Poliaček et al, 1975) shows that a relatively rapid warming was seen around since 1985. Data on air temperature after 1987 reveals that there were only weakly to strongly temperature-above-average years in this period, with two particularly above average years (2000 and 2007). According to the most widely used climate change scenarios a continuous increase in air temperature throughout the 21st century is indicated. According to the SRES B1 scenario (Lapin, 2008) a further warming by 1.6 °C is expected by 2100.

Some pavement structural design problems and solutions with respect to their temperature regime and climatic change conditions are described in the paper in sections dealing with asphalt pavements and pavements with cement concrete slabs.

2 TEMPERATURE REGIME OF ASPHALT PAVEMENTS

The most important characteristics of the asphalt pavement temperature are considered to be the average annual temperature of the asphalt layer, the average daily temperature, and their average values in each of the seasons. The bases for finding the characteristics of road pavements in Slovakia are data from long-term (repeated) measurements. Pavements were constructed only for this purpose (even in 1971) at four locations in Slovakia at varying altitudes from 100 to 1000 m above sea level. At each of these locations measurements of air temperature were taken. In Figure 1 we can see the daily temperature within an asphalt pavement at one of the sites (at 100 m.s.l.) during a summer day. The sequence of structure is given in the paper.

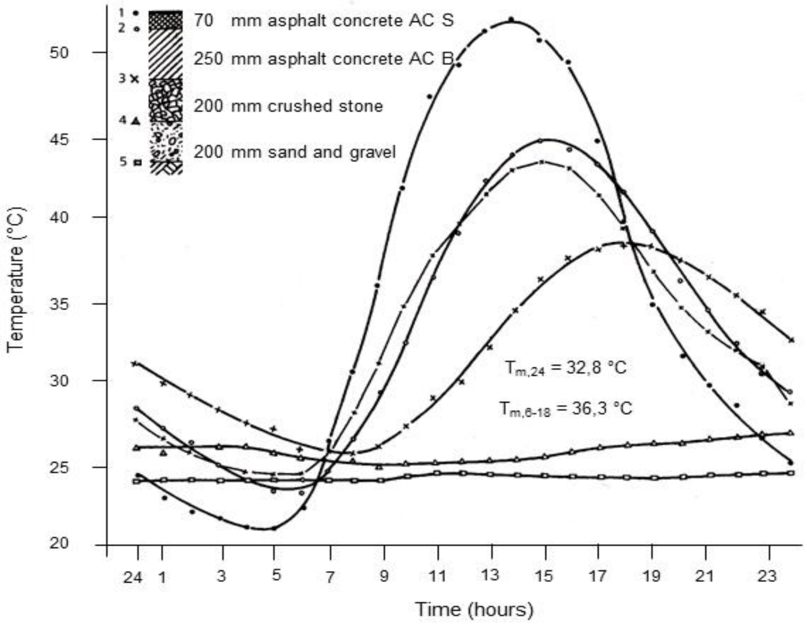

Figure1: Temperature changes in an asphalt pavement during a summer day

Figure 2 shows the changes of temperature in the layers of the same pavement over the year - being the average from daily measurements.

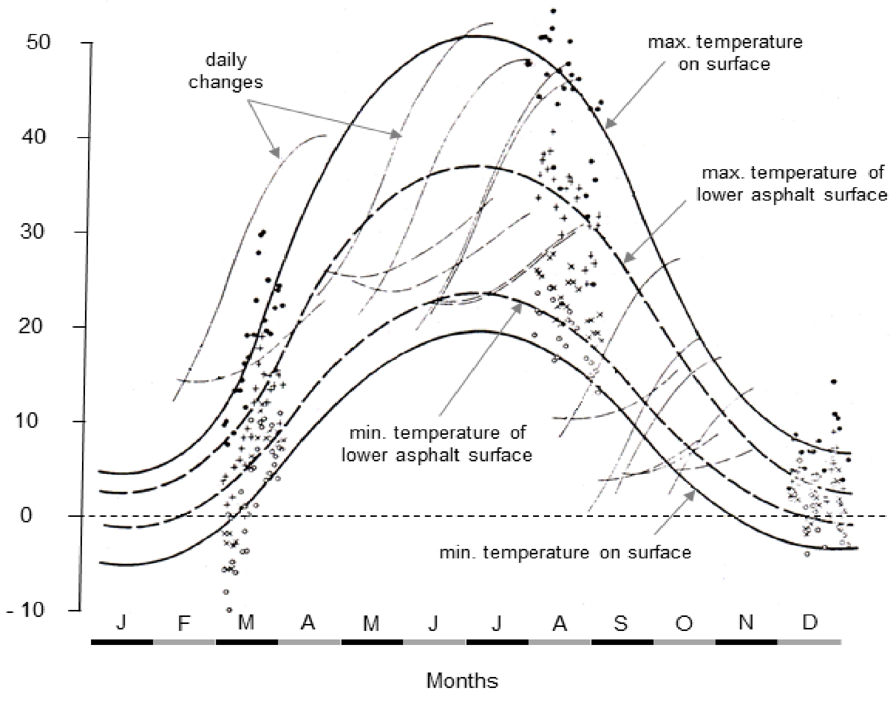

Figure 2: Temperature changes in asphalt pavement during the year

One piece of information gained from the temperature measurements was that the average annual temperature on the surface of asphalt pavements, at the bottom layers of the asphalt, and also on the subgrade are related to the average annual air temperature at the site (Poliaček & Gschwendt, 1990). This relationship is approximately linear and is shown in Figure 3.

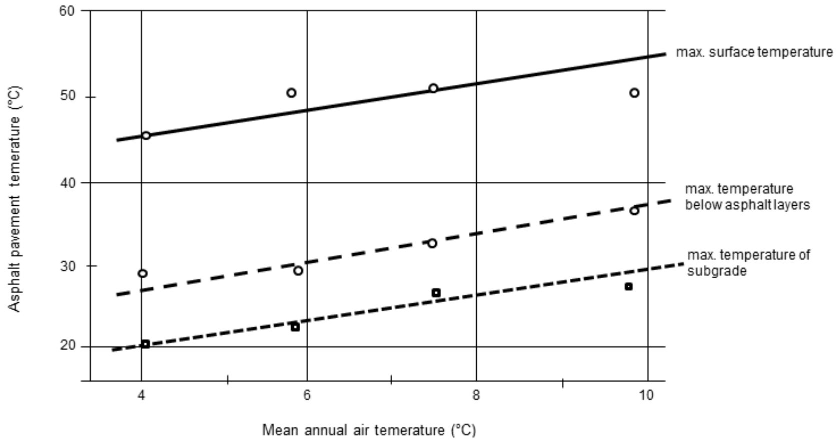

Figure 3: Asphalt pavement temperatures and mean annual air temperature for an asphalt pavement at 100 m.s.l. in Slovakia

In determining the representative temperature of asphalt layers for pavement design calculations, the temperatures during the day time [06 - 18 hours] were used, when five sixths of the traffic load is carried by the road. By a special study (Poliaček et al, 1975) we estimated that the damage to the pavement asphalt layers over a 24 hour period is the same as at a temperature equivalent during day time between 06 and 18 hours. The average equivalent temperature of asphalt layers from measurements at two locations and at different seasons are listed in Table 1.

Table 1: Average and equivalent temperatures of asphalt layers

Place altitude MAAT °C	Period	Temperature of 220 mm asphalt layers	
		24 hour average	equivalent 06 -18 hours
Bratislava	spring	11.3 °C	13.0 °C
130 m. s. l.	summer	26.6 °C	27.0 °C
9.8 °C	winter	- 0.8 °C	0.2 °C
Poprad	spring	10.2 °C	11.8 °C
700 m. s. l.	summer	25.0 °C	27.4 °C
5.8 °C	winter	- 1.8 °C	- 0.8 °C

In the calculations, we take the average asphalt pavement layer equivalent temperatures (for surfacing, binder and base layers) as shown in Table 2. We call them the "design values".

Table 2: Design values of temperatures of asphalt layers

Equivalent temperatures of asphalt layers according to the thickness		The portion of the year
from 150 to 200 mm	from 200 to 250 mm	
0 °C	0 °C	0.2 - winter
10 °C	11 °C	0.5 - spring, autumn
25 °C	27 °C	0.3 - summer

To assess the effect of temperature on the stress distribution in the asphalt pavement we used multi-layered elastic half-space calculations. Figure 4 shows the distribution of stresses in asphalt pavements layers which had their modulus mixes altered with temperature that varied from 0 °C (average in winter) to 27 °C (average in summer). Figure 4 seems to show that the radial stresses is more or less constant in the CBGM, but that the tensile stresses are much higher at the bottom of the asphalt in winter (0 °C).

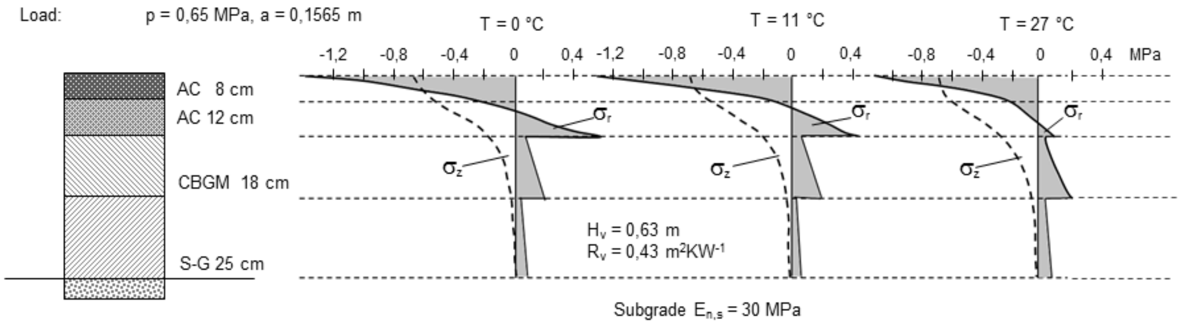

Figure 4: Stress distribution in asphalt pavement layers (σ_z and σ_r for equivalent temperatures)

To assess the influence of temperature change on the asphalt layers in standard and changing climatic conditions an exemplar asphalt pavement structure was selected:

SMA 11	40 mm
AC 16 L, PmB	60 mm
AC 16 P	80 mm
CBGM C5/6	150 mm
ŠD 31.5, G_C	200 mm

Subgrade bearing capacity	E_p = 60 MPa

To calculate the radial stress in the layers for a standard axle load (2P = 100 kN, tire pressure p = 0.60 MPa), we used a multi-layered elastic half-space solution (computer Slovak programme with the name LAYMED). When considering the life of such structures the radial stress will be critical in the cement bounded material CBGM. The computed radial stress, σ_r, was compared with the tensile strength of the CBGM material:

- for the regime with equivalent temperatures of 0 °C, 10 °C, 25 °C:

0.04214 MPa 0.2839 MPa 0.3266 MPa

- for the regime with equivalent temperatures of 0 °C, 11 °C, 27 °C:

0.04214 MPa 0.2861 MPa 0.3323 MPa

The coefficient of fatigue strength utilization, *SV*, of the material (CBGM) was calculated using the equation:

$$SV = \sum q_i \frac{\sigma_R}{S_N R} \qquad (-)$$

where q_i is the relative length of the winter, mid-season and summer periods (0.2, 0.5 and 0.3 of a year),

S_N - fatigue expressed by the equation 1 - 0.095 log N (N is number of repeated loading),

R - tensile strength of material (for the three temperature conditions).

For climate conditions in Slovakia with periods of 0 °C → 0.2 (20% of the year), 10 °C → 0.5 (for 50% of the year) and 25 °C → 0.3 (for 30% of the year) the coefficient of fatigue strength utilization was as follow: ***SV* = 0.8573**

When we increased the temperature to 11 °C during spring and autumn, and 27 °C in summer factor, then:

***SV* = 0.8670**

If the summer period is assumed to be 10 days longer (duration of 0.33 of the year) and an equivalent temperature of +25 °C, the coefficient was recalculated as:

***SV* = 0.8617**

and if summer had an equivalent temperature of + 27 °C the coefficient was:

***SV* = 0.8718**

On the basis of the results of this calculation we may consider equivalent summer temperature +27 °C. When temperature changes from 25 °C to 27 °C, coefficient SV changes from 0,857 to 0,867, which has s greater influence on the life of structure, than extension the summer period from 104 to 114 days with equivalent temperatures (experimental measurements).

An important climatic condition for Slovakia is the frost action on the pavement in winter. Therefore, one of the criteria for assessing the design of pavement structures is to protect against penetration of frost into the subgrade. The frost penetration is calculated by the empirically derived equation:

$$h_{pr} = c \sqrt{I_m} \qquad \text{(m)}$$

where I_m is the frost index of the territory °C, day ,

c - coefficient dependent on thermal conductivity of asphalt pavement, with an empirically derived value of $c = 0.050$.

An extended (and refined) relationship for frost penetration in which we consider the thermal resistance of pavement R_V and thermal conductivity of the frozen soil in the subgrade λ_Z has the form:

$$h_{pr} = H_V + \left(\frac{h_{pr}}{\lambda_m} - R_V \right) \times \lambda_Z \qquad \text{(m)}$$

where H_V is the thickness of pavement, m,

h_{pr} – reduced frost penetration for λ_m, which is $0.178 \times I_m^{0.3}$,

λ_m – 1.75 W.m^{-1}.K^{-1},

R_V – thermal resistance of the pavement, m^2.K.W^{-1},

λ_Z – thermal conductivity of the frozen soil, W.m^{-1}.K^{-1} .

Criteria for the protection of pavement were derived from the results of measurements of the frost penetration into, and heave of the pavement surface. The frost heave of a pavement, Δz, depends on the thickness of the frozen subgrade soil layer $h_{z,pr}$ (m) according to empirically estimated relationship:

$$\Delta z = 80 h_{z,pr}^2 \quad \text{(in mm)}$$

Thermal resistance, R_V, of a pavement must be such that in a given area, characterized by the frost index, I_m, the frost heave, Δz, must be limited. Its value was determined to be 20, 25 or 45 mm, depending on the category of road and traffic load. Taking the smallest value, Δz = 20 mm, then the permissible thickness of the frozen soil in the subgrade is:

$$h_{z,pr} = \sqrt{\frac{20}{80}} = 0{,}50 \quad \text{(m)}$$

Criteria for frost heave of concrete pavements can be more demanding, and also have to be corrected due to the stiffness of the cement concrete structure.

The design methods specified in the technical recommendation (used in Slovakia) for design of cement concrete and asphalt pavements establish criteria to protect pavements against damage by use of the relationship:

$$R_V \geq R_{V,\,potr} \qquad (\mathrm{m^2KW^{-1}})$$

where Rv_{potr} is the required thermal resistance of the pavement, with values presented in Table 3. There are dependent of the frost index, I_m, the water regime in the subgrade, the frost susceptibility of soil and the category of the road (A, B, C, D). The arrangement (organization) of the table with Rv_{potr} is as follows:

Table 3: The required thermal resistance of the pavement (x, y, z)

Frost Index (°C, day)	Subgrade moisture regime	Frost susceptibility of soil								
		Road category			Road category			Road category		
		A B	C	D	A B	C	D	A B	C	D
I_m	diffusive	x_1	x_2	x_3	x_4	x_5	x_6	x_7	x_8	x_9
	pendular	y_1	y_2	y_3	y_4	y_5	y_6	y_7	y_8	y_9
	capillary	z_1	z_2	z_3	z_4	z_5	z_6	z_7	z_8	z_9

According to the basic empirical relationship for frost penetration, the required thermal resistance depends on the frost index. The question then arises whether to modify the criterion for protection of pavement so as to take into account the effect climate change and the frost index. Here is a sample analysis for one place:

- for the region of Bratislava (130 m. s. l.) the average air temperature records for 50 years from the winter of 1951/52 leads to a calculated design frost index, with a frequency of n = 0.1, $I_{m,n}$, of 300 °C days,
- the mean frost index for 20 years from the winter of 1951/52 was 157.15 °C days, with the maximum value of I_m in this period being 402.4 °C days,
- the mean frost index for the last 20 years from the winter of 1989/90 was only 94.025 °C days, with the maximum value of I_m in this period being 290.4 °C days.

The frost penetration for the design frost index value, $I_{m,n}$, is 866 mm, with the maximum value of I_m = 402.4 °C days giving a penetration of 1003 mm. For the average I_m over the last 20 years, the frost penetration, h_{pr}, is 852 mm. One could say that for reduction the criteria, respectively design frost index value there are not reasons in present time. The problem is connected to the fact that, during winter, there are the frequent changes in temperature and repeated cycles of thaw and freezing of the soil, which has an adverse effect on the subgrade bearing capacity and on the load-bearing capacity of a pavement.

3 THE TEMPERATURE REGIME OF CEMENT CONCRETE PAVEMENTS

The experience from practice and knowledge about the behavior of cement concrete (CC) pavements confirms that there is a very significant effect of temperature and temperature regime on the deformation and stresses in concrete pavements. The dimensions of slabs, their thickness and length are designed according to an empirically observed relationship between air temperatures in the area (region) and the temperature in the CC slabs. The temperature regime characteristics of cement concrete pavements are:

- the average annual air temperature $T_{m,v}$ (°C),
- amplitude of average daily air temperature in the annual cycle A_r (°C),
- frost index I_m (°C days), the design value is taken consider with different frequency.

We assume that the mean temperature in the middle of cement concrete pavement structure is equal to $T_{m,v}$, to the average annual air temperature.

A very important characteristic of the CC slab is the temperature difference between the top and bottom surfaces: ΔT. It tends to cause deformation of slabs in the form of curling. Depending on the weight of the slab (its dimensions) and friction between the slab and the surface of base layer this can lead to stresses as large as those caused by the vehicle loading. By measuring the temperature and the temperature regime in the CC slabs during the day (for example, see Figure 5) and throughout the annual cycle, we obtain values from which is derived the relationships to the average annual air temperature $T_{m,v}$:

- average daily temperature amplitude in the CC slab throughout the annual cycle

$$A_r = 17.82 + 1.2\, T_{m,v} - 0.39\, h_B$$

where h_B is the thickness of the CC slab, mm

- positive temperature difference (design value)

$$+\Delta T_h = 12.440 - 0.6\, T_{m,v} + 0.028\, h_B,$$

- negative temperature difference (design value)

$$-\Delta T_h = 6.214 - 0.3\, T_{m,v} + 0.013\, h_B,$$

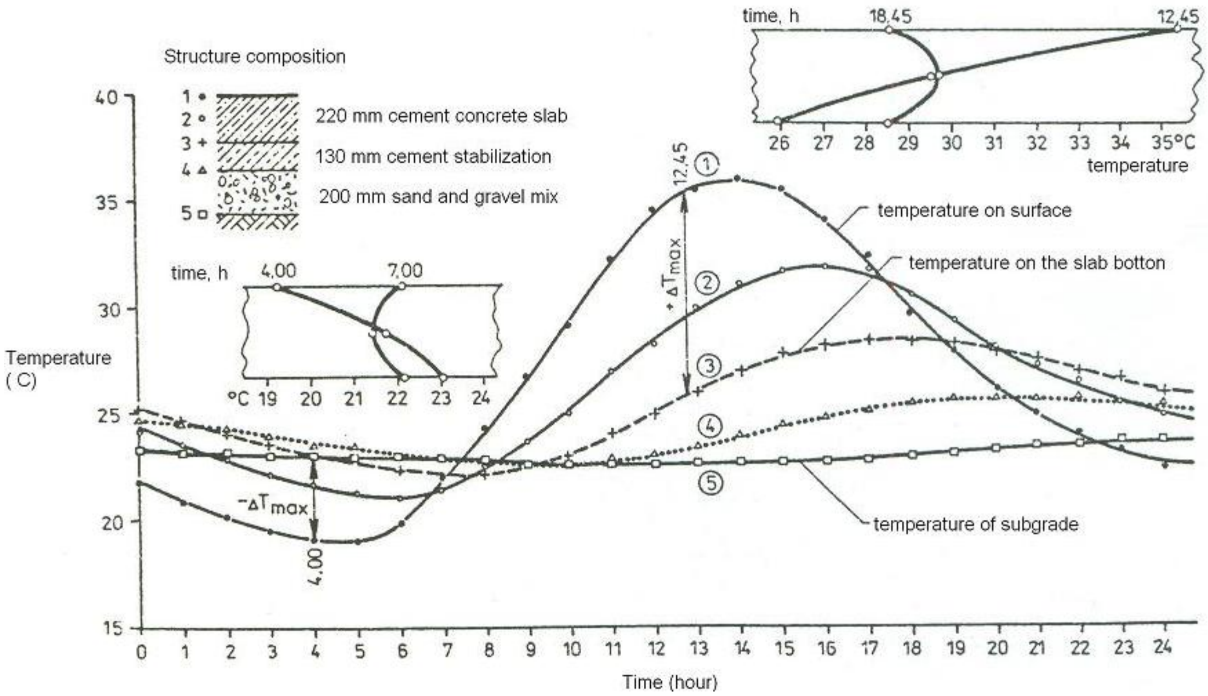

Figure 5: Temperature changes in cement concrete pavement during a (summer) day.

For the Bratislava region, with an average annual air temperature of $T_{m,v}$ = 9.8 °C and CC slab thickness 250 mm, the values of these characteristics were calculated as follows:

$$A_r = 19.83\ °C$$
$$+\Delta T_h = 13.560\ °C$$
$$-\Delta T_h = 6.524\ °C$$

The magnitude of displacement of 5m long slabs would then be:

$$\Delta l = \Delta T.\alpha_T.l, \quad \text{where } \alpha_T = 0.000012$$
$$\Delta l = 1.2\ \text{mm}$$

If the CC slab is long and cannot expand, stresses are induced in it due to friction

$$\sigma_N = \Delta T.\alpha_T.E$$

Where the elastic modulus E is 30 000 MPa, the compressive stress is, thus, 7.2 MPa. In technical recommendation is reported that the maximum value of compressive stress for this conditions is approximate:

$$\max.\ \sigma_N = 0{,}35\ \Delta T \quad \text{(MPa)}.$$

In real conditions, slab lengths are limited and displacement (expansion) depends on contact friction with the base layer and on the density of the concrete ρ = 2400 kg/m^3. The degree of freedom to expand and the induced stress affects the need for reinforcement of joints by dowels, but the compressive stress in slabs with the contraction joints will be not crucial for slab dimensions. For the assessment of the dimensions of slabs due to static load, and of the service life of slabbed pavements, we need to know the temperature difference between the top and bottom surfaces of the slab, ΔT. One of the solutions to the calculation of compressive and tensile stresses caused by temperature difference, ΔT, is by Westergaard's method. In the geometric center of the slab (s) the stress will be:

$$\sigma_{\Delta T,sx} = \frac{\alpha_T \Delta T E_{B,T}}{2\left(1-\mu^2\right)}\left(C_x + \mu C_y\right)$$

where α_T is the coefficient of thermal expansion of concrete = 0.000012,

$E_{B,T}$ - modulus of elasticity of the concrete at temperature, T, usually (0.6 to 0.7) E_B,

μ - Poisson's ratio of the concrete,

C_x, C_y - coefficients determined for the ratios L_x/l_T and L_y/l_T, where the radius of relative plate stiffness l_T is given by:

$$\sqrt[4]{\frac{E_{B,T}.h_B^3}{12\left(1-\mu^2\right)k}}$$

k is the base and subgrade modulus of reaction [MN.m^{-3}]

When we consider temperature difference $+\Delta T$ according to the empirical relationship given earlier, we find that the dependence of ΔT on the average air temperature $T_{m,v}$ does not make sense. The true relationship may be based on a statistical evaluation of multi-year measurements. For a design value of ΔT we take 0.048 °C/mm and a maximum value ΔT_{max} = 0.054 °C/mm. As in the derivation of equivalent temperature of asphalt layers, we want to know and consider the effective temperature difference ΔT during the period of significant traffic loading from 06 to 18 hours. This difference ΔT we called the "reduced" temperature difference and it can be calculated from the equation:

$$+\Delta T_r = \eta.\beta.\Delta T_h$$

where η is a coefficient that depends on the distribution of traffic loading between 06 and18 hours and on the temperature profile,
β - coefficient expressing the frequency of positive difference of the temperature in the CC slab.

Tables defining the realationship between $+\Delta T_r$ and ΔT_h were prepared on this basis, but to simplify the calculations, the relationship can be used:

$$+\Delta T_r = 0.35\ \Delta T_h$$

Experiments (Poliaček et al, 1975) found that the value of radial stress induced by temperature differences between the top and bottom surface of the CC slab is also dependent on the support characteristics of the underlying system or subgrade. A physical experiment was prepared with a 210 mm thick CC slabs, each measuring 3.5 x 4 m, being placed on different base layers: a layer of cement-bound aggregate, an asphalt concrete, a crushed stone, and on gravel. The stiffness of the base layer was assessed by the modulus of reaction to be between 80 and 200 $MN.m^{-3}$. In addition, the slab was supported by an air bag (pillow) when the modulus, k_{ef}, is considered to be zero. When heating the surface of the slab and creating the temperature difference ΔT, the warping height y_w in the middle of plate was measured. Figure 6 shows the dependence of the warping height on the stiffness of the base at a constant temperature gradient across the slab's thickness.

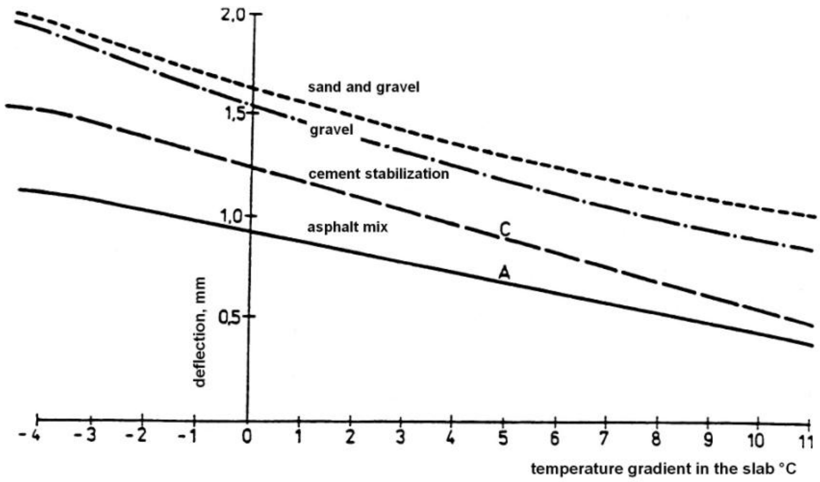

Figure 6: Warping height of the curled slab on different base supports

The largest warping height of the slab was on the base with $k_{ef} = 0$, when deformations are "free" and then the slab experiences very little stress. As the stiffness of the base is increased, the value of warping height, y_w, becomes smaller, which means that the stresses due to the temperature difference are higher.

The effect of base stiffness and thickness of the slabs on the value of the radial stress in slabs due to the temperature difference, ΔT, can be analyzed with respect to stress, $\sigma_{T,sx}$, as proposed by Westergaard. In the formula for the radius of relative stiffness, in addition to the modulus of elasticity of concrete, $E_{B,T}$, and the thickness of the slab, h_B, there is the modulus of reaction of base, k, [$MN.m^{-3}$].

Calculating $\sigma_{T,sx}$, for example with a slab 3.75 x 3.75 m in area and 220, 250 or 280 mm thick on a base with k = 80, 100, 150 or 200 $MN.m^{-3}$ at different temperature gradients we find that:

- a thin slab (220 mm) on a 100 MN.m^{-3} stiffness base gives $\sigma_{T,sx}$ as follows
  ΔT = 10 °C 1.515 MPa,
  ΔT = 20 °C 3.0320 MPa,
- and when the slab is 280 mm thick on a base of k =100 MN.m^{-3}, $\sigma_{T,sx}$ is
  ΔT = 10 °C 0.984 MPa,
  ΔT = 20 °C 1.968 MPa.

The stress $\sigma_{T,sx}$ in the thicker slab is smaller and the effect of temperature gradient is significant.

In terms of the design of slabs (the shape) we can aid decisions by use of a graph, on which the relationship between $\sigma_{T,sx}$ for square 3.75 x 3.75 m and a rectangular slabs 3.75 x 5.5 m in area and 250 mm thickness at different temperature gradients is plotted (Figure 7). Smaller values of $\sigma_{T,sx}$ are observed in the middle of square slabs.

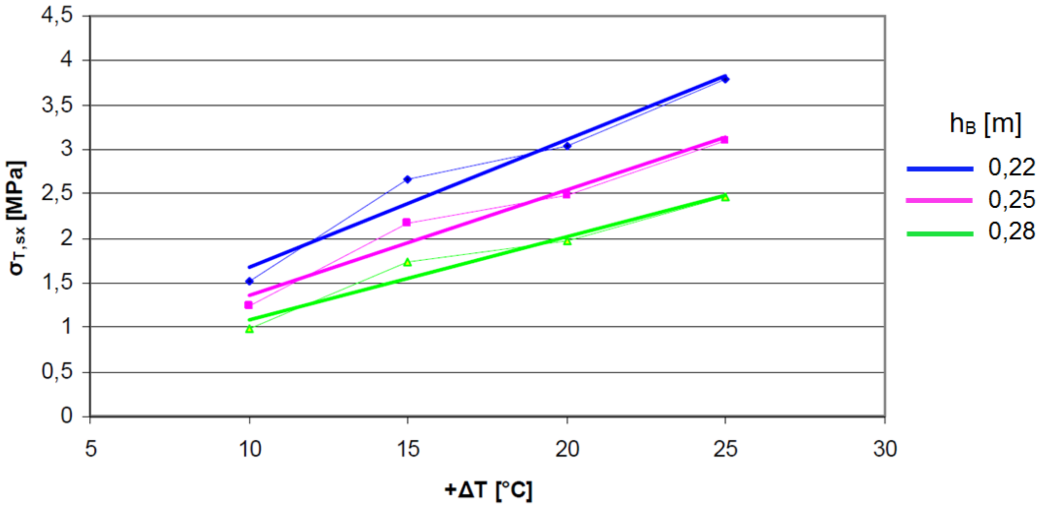

Figure 7: Radial stresses in a CC slab 3.75 x 3.75 m induced by temperature gradient ΔT

The adverse effect of temperature gradient on the CC slab stress can be reduced when pavements are designed for tunnels (Zuzulová, 2007). As an example of the calculations of the stresses caused by an axle carrying axle load 2P = 100 kN and the temperature gradient in the tunnel which was 10 °C - 2.2 °C = 7.8 °C (structure in tunnel K HD) compared to that in a rural area which was 35 °C - 6.6 °C = 28.4 °C (structure K EX). To calculate the stress of a slab 220 mm thick a finite element method (FEM) was used. In Tables 4 and 5 σ_r is an overview of the computed load stresses for the two temperature gradients. The results of calculations of the stresses induced by the temperature gradient alone are plotted on Figure 8 and Figure 9.

Table 4: The calculated stresses in the CC slab – structure K EX

Structure	Position of the load	Stresses (MPa) from		
		load 100 kN	load 115 kN	temperature
K EX	center	0.781	0.881	1.753
	longitud. edge	1.237	1.409	1.468
	transver. edge	1.272	1.446	1.404

Temperature: at the surface +35 °C and temperature gradient 0.03 °C per mm.

Table 5: The calculated stresses in the CC slab – structure K HD

Structure	Position of the load	Stresses (MPa) from		
		load 100 kN	load 115 kN	temperature
K HD	center	0.757	0.854	0.588
	longitud. edge	1.232	1.337	0.482
	transver. edge	1.170	1.389	0.499

Temperature: at the surface +10 °C and temperature gradient 0.01 °C per mm.

For the given conditions, the calculations stresses were made on the K EX and K HD structures. In the case of the K EX structure, the length of slabs is 5.5 m and the width is 3.75 m. Structure K HD has a pavement divided into slabs 7.0 m long and 3.53 m wide. The stresses calculated due to the temperature effect inside the tunnel (K HD) are smaller than in the case of the rural pavement (K EX). This difference is due to the smaller temperature gradient and due to the fact that the structure (K HD) is designed for the middle of the tunnel. As opposed to the point of maximum load-induced stress, the temperature-induced maximum stresses are in the center of the slab.

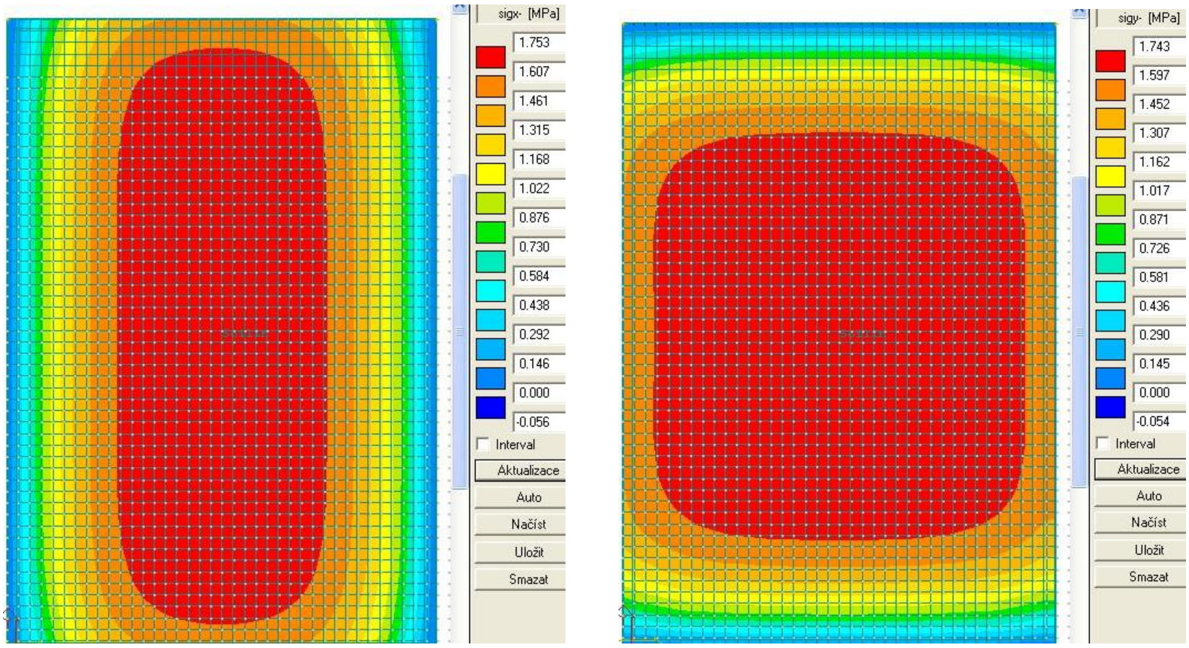

Figure 8: The stress distribution (σ_x, σ_y) in concrete slabs (K EX) due to temperature effects (surface temp. = +35 °C; bottom temp. = +28.4 °C)

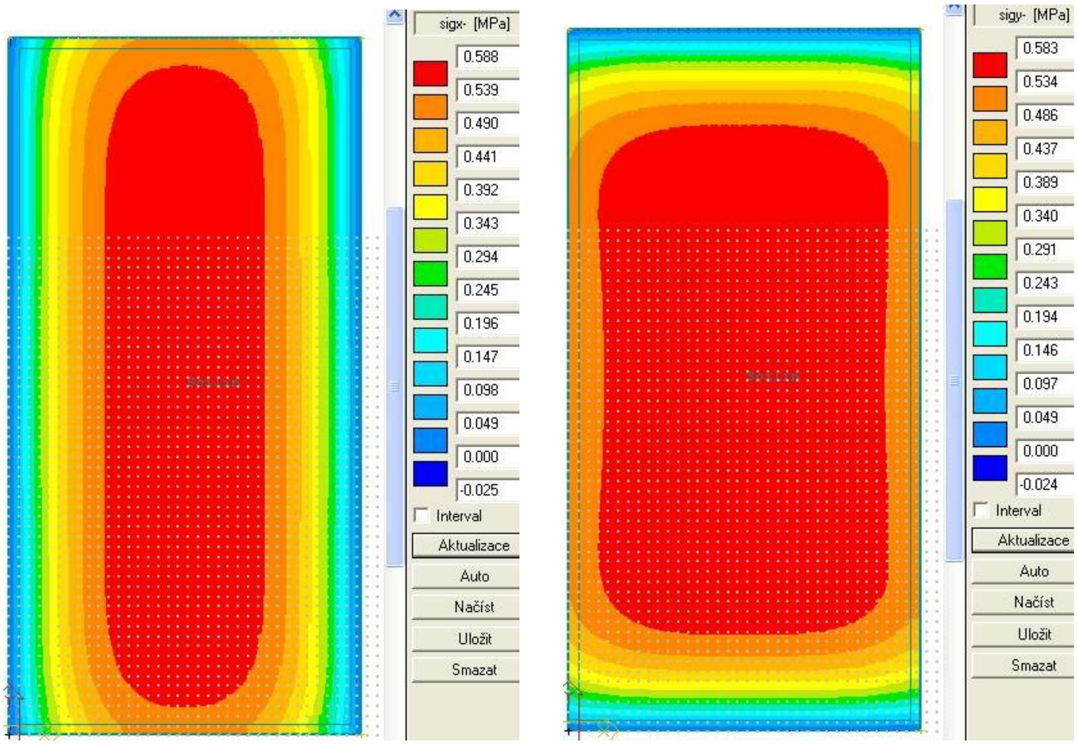

Figure 9: The stress distribution (σ_x, σ_y) in concrete slabs (K HD) due to temperature effects (surface temp. = +10 ºC; bottom temp. = +7.8 ºC)

4 CONCLUSION

Long-term measurements of temperature and thermal characteristics of the asphalt pavement in places with different altitudes were very important as a basis for the formulation of the input data (terms) and criteria in the road pavement design method. The design methods utilize the equivalent temperature of the asphalt layers in the winter, spring and autumn and summer, as well as the duration of each period. Comparative calculations with a view to evaluation of the consequences of changes in climatic conditions have shown that the greater impact on the dimensioning of the pavement structure has the effect of the anticipated temperature increase from a mean of 25 °C to 27 °C than extension of summer period.

Experiments with cement concrete slabs showed that, due to the distortion caused by the temperature gradient across the slab, it is better to allow deformation of the base of slab, as this is associated with a smaller radial stress. Calculation models of cement concrete slabs showed that the temperature-induced stresses are smaller in square panels as compared to rectangular slabs (with an aspect ratio up 1:2).

The results of experiments and model calculations give a basis for improvement of pavement design methods and the design principles (rules) and allow the likely effects of climate change to be incorporated into the design process.

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