Durability of bridge asphaltic concrete pavements under temperature loads

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Abstract. This article presents the major results obtained during the research of the effect of temperature factors on the formation of cracks in asphalt concrete pavements of bridges. The main problem was divided into a number of parts: determination of temperature field in pavement; creation of an engineering design model for determining the stress-strain state caused by temperature deformation; determination of asphaltic concrete rheological characteristics; introduction of criteria for assessing the durability of asphaltic concrete pavement.

1. Introduction
The main objective of the studies is to determine the stress-strain state of the asphalt concrete pavement of bridges caused by the difference in the thermal expansion-compression coefficients of the hard base (reinforced concrete slab or orthotropic metal slab) and asphalt concrete pavements interconnected by a system of protective-adhesive and waterproofing layers, and their durability. As a basis for research, we took asphalt concrete pavement of an orthotropic plate of a metal bridge, with a single protective-adhesive layer made of a material similar in its properties to bitumen. In general, the design was considered as a multilayer plate.

2. Temperature field of a multilayer plate
We assumed that the temperature varies only in thickness. Initially, we attempted to obtain an analytical approximate solution using Fourier series. However, in the case of multilayer plates, this approach led to the fact that the solution did not have physical sense. Moreover, for single-layer plates, this method is perfectly suitable and was used to verify the results of determining the temperature field in ANSYS, which implements the finite element method (FEM) [1]. The verification showed a difference in the results of the analytical approach and the FEM for a single-layer plate of less than 4% and we decided to use ANSYS for a multi-layer plate.

We took the data on temperature changes for December, January and February as temperature boundary conditions (this period has the highest rates of temperature change) in 2006. The initial temperature conditions assumed a uniform temperature distribution. We used a flat 4-node PLANE55 element (Quad 4node 55), as in the test task. An example of a temperature field is shown in Figure 1.
Figure 1. The temperature field of the bridge canvas:
point 2 - free boundary of asphalt concrete; point 22 - the contact plane of asphalt concrete and the protective-adhesive layer; point 26 - the free surface of the metal flooring of the orthotropic plate.

Analyzing Figure 1, we can conclude that the stress-strain state is influenced by the temperature drop across the thickness of the asphalt concrete. We found that the stiffeners of the orthotropic plate do not significantly affect the temperature field of the multilayer plate. According to the calculation results in ANSYS, depending on the thickness of asphalt concrete, the maximum values of the temperature difference in thickness were: for asphalt concrete of 0.1 m thick - 5.5 °C; for asphalt concrete of 0.15 m thick - 7.5 °C; for asphalt concrete of 0.2 m thick - 8.9 °C.

3. Stress-strain state model of multilayer plate

We considered 3 calculation models describing the stress-strain state of a multilayer plate [2]:
- A model using the hypothesis of complete deformations uniform in height, with stresses determined by the following equation (uniform deformation hypothesis):
\[
\sigma_i(y) = E_i \cdot c - E_i \cdot \alpha_i \cdot \Delta T_i(y),
\]
(1)
- A model using the hypothesis of linearly varying heights of the entire multilayer deformation plate, with stresses determined by the following equation (inclined hypothesis):
\[
\sigma_i(y) = E_i \cdot (a \cdot x + b) - E_i \cdot \alpha_i \cdot \Delta T_i(y),
\]
(2)
- A model using the hypothesis of linearly varying deformations for each layer (the hypothesis of a broken line):
\[
\sigma_{zz}^{(i)}(z,y) = E_i \cdot \varepsilon_{zz}^{(i)}(z,y) = E_i \cdot \alpha_i \cdot \Delta T_i(y),
\]
(3)
\[
\varepsilon_{zz}^{(i)}(z,y) = \frac{\partial u^{(i)}(y,z)}{\partial z} = U_i(z) \cdot \frac{-y + y_{i+1}}{h_i} + U_{i+1}(z) \cdot \frac{y - y_i}{h_i},
\]
(4)
where:
\( i \) – number of layer;
\( \sigma_i(y) \) – stress in the i-th layer, MPa;
\( E_i \) – the elastic modulus of the material of the i-th layer, MPa;
\( \alpha_i \) – coefficient of linear thermal expansion-compression, \( \frac{1}{\text{°C}} \);
\( \Delta T_i(y) \) – function representing the temperature difference in the i-th layer, °C;
\( c \) – value of deformations uniform in height;
\( a, b \) – sought coefficients for the hypothesis of the inclined;
$U_i'(z)$ – displacements of the boundary of the layers $i$ and $i-1$, specified in the form of approximating functions satisfying the boundary conditions;  
$h_i = y_{i+1} - y_i$ – thickness of the $i$-th layer, m.

The hypotheses of inclined and broken lines are shown more clearly in Figure 2.

![Figure 2. Hypotheses of inclined and broken lines.](image)

The results obtained by the above hypotheses were compared with the results obtained in ANSYS. In this case, the selection criteria were the difference between the results obtained using the hypothesis and the FEM results and the simplicity of the equations obtained using the hypothesis. In addition, using FEM, we preliminary analyzed the factors affecting the choice of a hypothesis for the calculation model and made the following conclusions:

1. The temperature difference across the thickness of the asphalt concrete does not significantly affect its thermally stressed state. As a consequence, we further assumed that the temperature is distributed uniformly throughout the structure and is equal to the ambient temperature.
2. The normal stresses of asphalt in the transverse and longitudinal directions differ slightly.
3. Normal stresses of asphalt concrete increase from the edge of the plate to its center.
4. An increase in the stiffness of the base by a factor of 1000 does not lead to a significant increase in stresses. For this reason, we concluded that the models under development are applicable not only for metal bridges with an orthotropic plate, but also for reinforced concrete bridges, where the stiffness of the base, taking into account the walls of the main beams, can be higher.

As a result of the comparison, we concluded that the hypothesis of the inclined and the broken line hypothesis correspond to a numerical solution with a sufficient degree of accuracy. Basing on the simplicity criterion, we should choose the hypothesis of the inclined. A comparison of the hypothesis of the inclined with the uniform deformation hypothesis, shown in Table 1, allows us to conclude that the simplest uniform deformation hypothesis is best suited to determine the thermally stressed state.
### Table 1. A comparison of the hypothesis of the inclined with the uniform deformation hypothesis.

| Description                                                                 | Hypothesis of Uniform Deformations from the FEM | Hypothesis of the Inclined and the FEM |
|------------------------------------------------------------------------------|--------------------------------------------------|----------------------------------------|
| Difference in maximum stresses at various geometrical parameters of an orthotropic plate | 8.4 %                                            | 12 %                                   |
| The difference in maximum stresses with an increase in the elastic modulus of a metal of an orthotropic plate by 1000 times | 10 %                                             | 10 %                                   |
| The difference in maximum stresses with an increase in the elastic modulus of asphalt concrete by 10 times | 1.5 %                                            | 2.5 %                                   |
| The difference in maximum stresses with a decrease in the elastic modulus of asphalt concrete by 10 times | 4 %                                              | 4 %                                     |
| The difference in maximum stresses with an increase in the thickness of asphalt concrete by 0.05 m | 2 %                                              | 7 %                                     |
| The difference in maximum stresses with a decrease in the thickness of asphalt concrete by 0.03 m | 6 %                                              | 6 %                                     |

### 4. Building a creep model for asphalt concrete

To describe the creep of asphalt concrete, the equation of the stiffness theory was used in the following form [3, 4]:

\[
\dot{\varepsilon}_{cr}(t) = \frac{\sigma^{k(T, \sigma)}_c}{(c(T) + s^{4}(\sigma, T))^{\alpha(T)}}
\]

where:
- \(\dot{\varepsilon}_{cr}(t)\) – creep relative strain rate;
- \(\sigma\) – stress, MPa;
- \(t\) – time, sec;
- \(T\) – temperature, °C.
- \(k(T, \sigma), c(T), s^{4}(\sigma, T), \alpha\) – coefficients, which in the general case depend on temperature and stresses, which are selected from the condition of best approximation to the experimental data.

Since during the experiment the stresses remained constant, integrating expression (5) we obtain the expression of creep deformations in explicit form:

\[
\varepsilon_{cr}(t) = \frac{\sigma^{k(T, \sigma)}_c}{(c(T) + s^{4}(\sigma, T))^{\alpha(T)}} \cdot t \cdot s(\sigma, T),
\]

To obtain creep curves from fine-grained dense asphalt concrete grade B-I, we took standard samples with a diameter and height of 71.4 mm. Samples were inserted into a lever specially made for this, which was located under a canopy on the street (Figure 3), where under conditions of variable temperature equal to the ambient temperature, a compressive load of various sizes was applied to the samples. As a result, the following dependences were obtained [5], graphically shown in Figures 4, 5 (experimental curves are shown at a variable temperature, on average equal to the specified):

- Stresses 0-1.75 MPa:

\[
\varepsilon_{cr}(t) = 4 \cdot \left(350 + 310e^{-0.05T}\right) \cdot 1.75(-15.5-15.5e^{-0.04}) \cdot t \cdot s(\sigma, T)
\]

1) Stresses 1.75-3.5 MPa:
\[ \varepsilon_{cr}(t) = 4 \sqrt{\frac{350 + 310e^{-0.167T}}{1 + 0.171(T - 1.75)}} \cdot \sigma(15.5 - 15.5e^{-0.044}) \cdot \frac{t}{T} \] (8)

2) Stresses 3.5-5 MPa:

\[ \varepsilon_{cr}(t) = 4 \sqrt{\frac{350 + 310e^{-0.167T}}{1 + 0.171(T - 1.75)}} \cdot \sigma(15.5 - 15.5e^{-0.044}) \cdot \frac{t}{T} \] (9)

3) Stresses >5 MPa:

\[ \varepsilon_{cr}(t) = 4 \sqrt{\frac{350 + 310e^{-0.167T}}{1 + 0.171(T - 1.75)}} \cdot \sigma(15.5 - 15.5e^{-0.044}) \cdot \frac{t}{T} + \frac{0.034 + 0.14 \cdot 10^{-3}T + 8.43 \cdot 10^{-5}T^2 - 1.17 \cdot 10^{-7}T^3 - 2.31 \cdot 10^{-8}T^4}{(\sigma - 5)} \] (10)

Figure 3. Asphalt concrete test lever.

Figure 4. Creep curves at 5 MPa.

Figure 5. Creep curves at 1.75 MPa.
In order to determine asphalt concrete creep strains under tension, an asphalt concrete bend experiment was performed on beams (Figure 6) and a conversion coefficient was found that allows us to determine tensile creep strains, knowing the creep strains under compression [6]:

$$k \varepsilon_{cr}^p = \frac{1+15 \cdot 10^{17} \cdot 2.3 (\varepsilon_{cr}^p)^6}{1+15 \cdot 10^{17} (\varepsilon_{cr}^p)^6},$$  \hspace{1cm} (11)$$

Multiplying (7-10) by (11), we obtain creep strains under tension at any stress level and temperature.

**Figure 6.** Bending experiment on asphalt concrete beams.

5. **The criterion for assessing the durability of asphalt concrete bridge pavements under thermal effects**

Given the above, the full deformations will be expressed as follows:

$$\varepsilon = \varepsilon_{el} + \varepsilon_T + \varepsilon_{cr},$$  \hspace{1cm} (12)$$

Where :
- $\varepsilon_{el}$ - elastic deformations;
- $\varepsilon_T$ - thermal strains;
- $\varepsilon_{cr}$ - creep strains.

From equation (12), we can determine the thermally stressed state, taking into account creep. But since temperature stresses act for a long time, the durability of the pavement is defined as its long-term strength using the theory of linear damage accumulation, the possibility of using which is mentioned in [7]:

$$P = \sum_{i=1}^{n} \frac{d t_i}{\tau_i},$$  \hspace{1cm} (13)$$

where: $i$ – time interval number;
- $d t_i$ – the duration of the $i$-th time interval, sec;
- $\tau_i$ - durability in the $i$-th time interval, under the assumption that during the time $d t_i$ the stresses act constantly, sec;
- $n$ – total number of time intervals.

When $P$ value becomes equal to one, destruction occurs. The durability $\tau$ can be determined, for example, by Zhurkov or Bartenev equations. In the absence of a sufficient amount of experimental data, we used the following data:

- [8] presents data for fine-grained asphalt concrete B-I in the temperature range from -40 $^\circ$C to 0 $^\circ$C:

$$\tau = \tau_0 \exp \left( \frac{\mu c - Y}{RT} \right),$$  \hspace{1cm} (14)$$

where: $\tau$ – durability of the material, under the action of constant stress $\sigma$, sec;
- $T$ – temperature, K;
- $\mu = 8.31 \cdot 10^{-3}$ – universal gas constant, $k/\text{mol}$.

\[ \frac{1}{K} = \frac{k_f}{m K} \]
\[ \tau_0 = 10^{-13} \] – constant corresponding to the period of oscillation of kinetic units (\( \tau_0 = 10^{-12} \div 10^{-13} \) c);

\[ u_0 = 89,85 \] – effective activation energy of the destruction process, \( \frac{kJ}{mol} \);

\[ \gamma = 60,58 \cdot 10^{-1} \] – structural coefficient characterizing the sharpness of the decrease in the activation energy of fracture with increasing stress, \( \frac{kJ}{mol \cdot MPa} \).

- in [9], data is presented for sandy asphalt concrete type G in the temperature range from -15 °C to +50 °C

\[ \tau = 5,9 \cdot 10^{-2} \cdot \sigma^{-(1,8+0,2\sigma)} \exp \left( \frac{43,2}{RT} \right) \] (15)

In equation (15), stresses \( \sigma \) are taken in \( \frac{kg}{cm^2} \), and durability is obtained in seconds. The change in ambient temperature can be set in the following form:

\[ T = \sum_{i=1}^{2} a_i \sin \left( \frac{\pi t}{p_i} + \frac{\pi}{2} \right) \] (16)

where: \( a_1 = 30^\circ C, a_2 = 5^\circ C \) - the amplitude of the annual and daily average temperature fluctuations.

\( p_1 = 1 \text{ year}, p_2 = 1 \text{ day} \) - periods of annual and daily temperature fluctuations.

By setting the temperature load in accordance with (16), taking into account (12), (1), (7) - (11) and substituting the obtained values in (13), it is possible to obtain the time after which the process of destruction of asphalt concrete pavement begins. The result of applying this procedure showed that the data obtained in this way do not correspond to the service life observed in the practice of building asphalt concrete pavements. Figures 7 and 8 show the stress variation curve during the year and the damage accumulation curve for the year for laying asphalt concrete at +25 °C. We can see that the onset of destruction is projected in less than a year.

Most likely this is a consequence of the lack of experimental data on the durability of asphalt concrete and the possible phenomenon of “healing” of damage in asphalt concrete at positive temperatures. Therefore, we decided to introduce a hypothesis: damage accumulation occurs only below a certain limit \{X\}, called the damage accumulation limit.

Figure 7. Curve of stress variation (MPa) over time, taking into account creep.
Based on observational data on asphalt concrete pavements of bridges and roads, we can conclude that cracks caused by temperature changes do not appear earlier than the autumn-winter period, i.e. at temperatures below +10 °C. At \( X = +10 \) °C and when laying asphalt concrete at +30 °C, it turns out that \( P \) reaches only the value of 0.33 in a year, i.e. destruction will begin in 3 years (Figure 9, 10). Thus, at the moment, it is possible to predict the start time of the destruction of asphalt concrete bridge pavements only with certain assumptions that require verification.

**Figure 8.** Curve of accumulation of damage for the year.

**Figure 9.** The curve of stress variation (MPa) over time, taking into account creep at \( X = +10 \) °C.
6. Conclusion

1) Numerical experiments have established that the real rate of change in ambient temperature does not lead to large temperature differences in the layer of asphalt concrete (not more than 5 °C). This, in turn, does not lead to a significant difference in the thermally stressed state of asphalt concrete in comparison with the case of a uniform temperature distribution (not more than 6% of the stresses in the elastic setting).

2) Numerical studies with FEM have established that the thermal stress state of asphalt concrete of the bridge pavement corresponds to the stress state of a multilayer beam with a uniform distribution of total deformations along its thickness, which is typical for the operation of centrally stretched elements.

3) Numerical studies with the FEM found that even a multiple increase in the stiffness of the base of a multilayer plate leads to an increase in stresses in asphalt concrete of not more than 10% in an elastic setting.

4) New experimental data on the creep of asphalt concrete have been obtained. Based on the analysis of experimental data, we found dependencies that describe the creep of asphalt concrete at various stresses in the temperature range from -30 °C to +30 °C. These equations can be used to calculate taking into account the creep of roads, bridges and other structures with asphalt concrete pavement.

5) At present, there is insufficient experimental data to predict the beginning of the destruction of asphalt concrete bridge pavements. However, we can introduce simplifying hypotheses based on observational data on the destruction of asphalt concrete pavements.

6) To accurately predict the onset of failure of asphalt concrete pavements, it is necessary to obtain more accurate data on the creep of asphalt concrete under cyclic loads, its durability and the presence or absence of the phenomenon of “healing” of damage at positive temperatures.

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