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THERMAL STRESSES IN COMPOSITE STEEL-CONCRETE BRIDGES DURING CONSTRUCTION STAGE

In the paper the analysis of stresses in concrete deck of steel-concrete composite bridges due to cumulative impacts of cement heat of hydration and ambient temperature have been presented. Examples of analyses of thermal conditions and thermal stresses in construction stage of composite road bridge have been shown.

Keyword: steel-concrete composite bridges, thermal conditions, thermal stresses

Genesis of the problem

One of the most characteristic features of the steel-concrete composite bridges is a two-stage process of their erection. In the first phase (Phase I) acting loads are transferred by steel girders appropriately to the static scheme compatible with the construction method. After the erection of the bridge deck (Phase II) loads are transferred by steel-concrete composite cross section.

The period between phases I and II is a period of curing of concrete slab. In this period a series of chemical and physical processes start and run, among which the process of emission of cement heat of hydration is very important. Due to the emission of cement heat of hydration in erected composite cross-sections an additional thermal load occurs. During simultaneous impact of the cement heat of hydration and the ambient temperature in erected composite cross-sections arises the temperature difference caused by large differences in thermal inertia of steel and concrete. The temperature differences induce an additional stresses in concrete slab and steel beams.

The complexity of the stresses analysis in described stage consist in the necessity of taking into account the changes in mechanical properties and strength of the curing concrete (e.g. modulus of elasticity, compressive and tensile strength). The strength characteristics of the composite cross-section (equivalent cross-section) depend on the time. The cross-sectional area, the moment of inertia and the position of the center of gravity of the cross-section depends on changing in time modulus of elasticity of the curing concrete.

For the purpose of further analysis it is necessary to determine the amount of the heat of hydration which can emit cement used in the concrete and the ambient temperature at which the composite section is erected. These issues will be presented on the examples of two composite road bridges within the A4 motorway over Ship Canal Gliwice in Southern Poland.

The case study

In two composite bridges D(Mo)10, D(Mo)11 at the A4 motorway, within a short time after the concreting, significant cracks in concrete deck has occurred. The cracks were parallel, perpendicular and diagonal to the objects axis. The crack width was up to 1 mm and the depth of most of ¼ to ½ of the deck thickness. Some of the cracks passed throughout the deck. The number of the cracks was up to 358 in the area of 100 m^2 in the object D(Mo)010, and 120 in the area of 100 m^2 in the object D(Mo)011.

The deck in the object D(Mo)010, with design thickness $d = 220$ mm (230 mm the real thickness), was concreted on 09/03/2002 at the ambient temperatures: at 7^{00} a.m. – 13 ºC, at 1^{00} p.m. – 27 ºC, at 9^{00} p.m. – 20 ºC. The deck of the object D(Mo)011 was concreted on 16/09/2002 at the much lower ambient temperatures: at 7^{00} a.m. – 2 ºC, at 1^{00} p.m. – 14 ºC, at 9^{00} p.m. – 12 ºC.

The temperatures were confirmed by the Institute of Meteorology and Water Management (IMGW) branch in Katowice. In the nearby meteorological station Zakrzów and Katowice, they were: 03/09/2002 $T_{\max} = 26,2$ ºC, $T_{\min} = 9,7$ ºC, $T_{mean} = 17,4$ ºC, 16/09/2002 $T_{\max} = 15,2$ ºC, $T_{\min} = 8,7$ ºC, $T_{mean} = 10,8$ ºC.

The night temperature drops after the decks concreting amounted respectively for:

– D(Mo)010 $\Delta T_{out} = T_{\max} - T_{\min} = 16,5$ ºC,

– D(Mo)011 $\Delta T_{out} = 6,5$ ºC.

The number of the cracks formed at the observed temperature drops indicated the thermal reason of the cracks.

The deck plate with the thickness of $d = 0,23$ m is characterized by a massive factor m equal

$$m = \frac{F}{V} = \frac{2}{d} = \frac{2}{0,23} = 8,7 \tag{1}$$

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where F – the outer surface of the element by which the heat transfer with the environment takes place [m^2], V – volume of the element, [m^3], d – thickness of the plate.

With the $m = 8,7$ m^{-1} the deck belongs to the medium-thick (medium-massive) concrete elements ($2 \le m \le 15$) in which, for the concrete class C25/30 and higher, the self-heating due to the cement heat of hydration can reach $\Delta t_{self} = 3...35$ ºC [1], depending on external conditions, the type of the cement and the water-cement ratio in the concrete. This value of the self-heating arises during the so-called thermal shock, i.e. in the first 50…80 hours (2–3 days) of curing of concrete.

In concrete mixture in both structures the cement Warta CEM I NA MSR 42,5 has been used. This cement has a relatively high heat of hydration. In isothermal conditions after 72 hours the cement Warta releases heat of hydration of $Q_{72} = 246$ kJ/kg (58,7 kcal/kg) [1]. In the first 24 hours of curing of concrete the cement Warta releases $Q_{24} = 166$ kJ/kg (39,6 kcal/kg).

In case of bridge D(Mo)010 constructed at $T_{mean} = 17,4$ ºC the rate of emission of the heat of hydration was lower than at 20 ºC in the calorimeter.

Assuming that the heat Q_{24} self-heating the bridge slab in the first 24 hours of curing of concrete, according to E. Rastrup function the self-heating of the slab would amount:

$$\kappa_1(17,4) = 2^{\frac{17,4-20}{10}} \cdot \kappa(20) = 0,84 \quad (2)$$

$$\Delta t_{adiab} = \kappa_1 \frac{C \cdot Q_{24}}{c_c \cdot \gamma_c} = 0,84 \cdot \frac{365 \cdot 39,6}{0,23 \cdot 2300} \approx 23,0\,, \quad (3)$$

where: C – cement content in concrete [kg/m^3], Q_{24} – amount of the heat of hydration emitted in the calorimeter after 24 hours [kcal/kg], c_c – specific heat of concrete [kcal/(kg·ºC)], γ_c – density of concrete [kg/m^3]

Because of the simultaneous heat dissipation by the external surfaces the temperature of the self-heating of the deck Δt_{sh} will be lower:

$$\Delta t_{sh} = \gamma \cdot \Delta t_{adiab} = 0,5 \cdot 23,0 = 11,5\,, \quad (4)$$

where: $\gamma = 0,5$ is accepted on the basis of experiences for massive factor $m = 8,7$ m^{-1} [1].

In case of the initial temperature of the concrete mix adopted approximately for the bridge D(Mo)010 as equal $T_{mean} = 17,4$ ºC (average ambient temperature) the temperature of the concrete slab, after 24 hours of curing, will be:

$$t_c^{mean} = T_{mean} + \Delta t_{sh} = 17,4 + 11,5 \approx 29,0 \quad (5)$$

At night the ambient temperature in the case of the D(Mo)010 bridge declined to $T_{\min} = 9,7$ ºC. Steel beams quickly cooled down to ambient temperature. Therefore in the composite section could arise the temperature difference equal:

$$\Delta t = t_c^{mean} - T_{\min} = 29,0 - 9,7 = 19,3\,. \quad (6)$$

Using the same procedure for estimation of the heat of hydration and the self-heating of concrete after 72 hour the temperature difference will achieve:

$$\Delta t_{adiab} = \kappa_1 \frac{C \cdot Q_{72}}{c_c \cdot \gamma_c} = 0,84 \cdot \frac{365 \cdot 58,7}{0,23 \cdot 2300} \approx 34,0\,; \quad (7)$$

$$\Delta t_{sh} = 0,5 \cdot 34,0 = 17,0\,; \quad (8)$$

$$t_c^{mean} = 17,4 + 17,0 \approx 34,4\,; \quad (9)$$

$$\Delta t = 34,4 - 9,7 = 24,7\,. \quad (10)$$

According to the above estimates in the D(Mo)010 bridge during the night temperature drop the temperature differences between the concrete slab and steel girders $\Delta t = 20...25$ ºC could occur.

In case of the D(Mo)011 bridge obtained results are as follows:

– after 24 hours of curing of concrete:

$$\kappa_1(10,8) = 2^{\frac{10,8-20}{10}} \cdot \kappa(20) = 0,53 \quad (11)$$

$$\Delta t_{adiab} = \frac{0,53}{0,84} \cdot 23,0 \approx 14,5 \quad (12)$$

$$\Delta t_{sh} = 0,5 \cdot 14,5 = 7,2 \quad (13)$$

$$t_c^{mean} = 10,8 + 7,2 \approx 18,0 \quad (14)$$

$$\Delta t = 18,0 - 8,7 = 9,3 \quad (15)$$

– after 72 hours of curing of concrete:

$$\Delta t_{adiab} = \frac{0,53}{0,84} \cdot 34,4 \approx 21,7\,; \quad (16)$$

$$\Delta t_{sh} = 0,5 \cdot 21,7 = 10,8\,; \quad (17)$$

$$t_c^{mean} = 10,8 + 10,8 \approx 21,6\,; \quad (18)$$

$$\Delta t = 21,6 - 8,7 = 12,9\,. \quad (19)$$

In the bridge D(Mo)011 the temperature difference between concrete slab and steel girder could achieve $\Delta t = 9,0...13,0$ ºC.

Additionally, the cement used in both bridges have a much better characteristics than it appear from the information provided by the manufacturer. Therefore in both bridges a greater heat of hydration and self-heating of concrete has occurred. Concrete strengths after 7 and 28 days reached the values:

– D(Mo)010

$$f_{cd7} = 43{,}7 \text{ MPa}, \ f_{cd28} = 59{,}8 \text{ MPa};$$

– D(Mo)011

$$f_{cd7} = 42{,}3 \text{ MPa}, \ f_{cd28} = 55{,}8 \text{ MPa}.$$

Concrete class required in both bridges (C25/30) was achieved already after 7 days. After 28 days concrete class was much higher than required and reached C35/45. Undoubtedly it was an additional factor which increased the destructive thermal cracks of the decks in both bridges because of the large amount and higher rate of emission of heat of hydration.

Analysis of the bridge D(Mo)010

The detailed calculations for the D(Mo)010 bridge were carried out. The geometry of the bridge, its cross-sections and static schema during construction stage are shown in Fig. 1 (dimensions in [cm]).

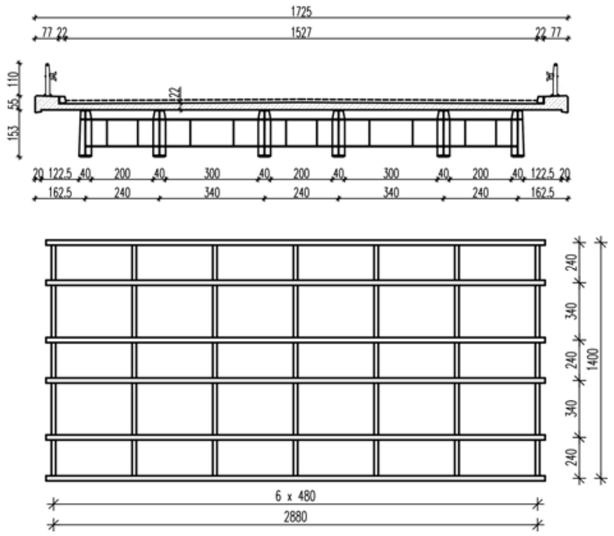

Fig. 1. Cross section and plan view of the D(Mo)010 bridge

The supports no. 2, 3, 4, 5 were temporary supports on which the precambering was realized.

As a result of the temperature difference between slab and steel girders in the structure a thermal bending moment M_T has occurred. Under the action of the M_T the structure had a tendency to lifting up and breaking away from the temporary supports.

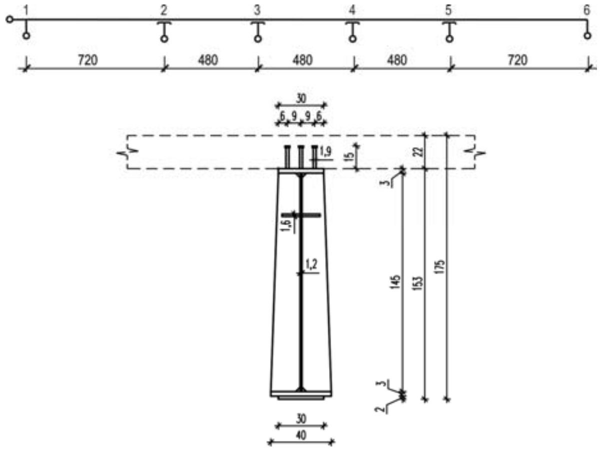

Fig. 2. Static schema during construction stage and beam cross-section of the D(Mo)010 bridge

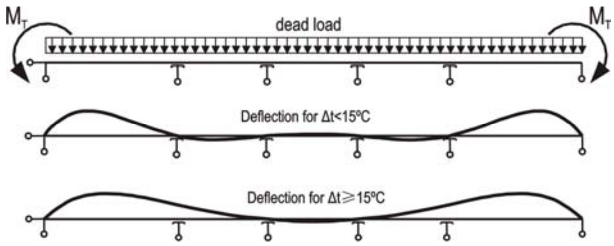

Fig. 3. The D(Mo)010 beams deflections due to the thermal bending moments for $\Delta t < 15$ ºC and $\Delta t \geq 15$ ºC

The calculations of internal forces and stresses during construction stage was carried out for $\Delta t = 10, 15$ and 20 ºC taking into account a time of concrete maturation $\tau_T = 1, 2, ..., 5$ days. Where τ_T the temperature is adjusted concrete age taking into account influence of elevated temperature of curing of concrete on faster maturation of concrete:

$$\tau_T = \sum_{i=1}^{n} e^{-(4000/\left[273+T(\Delta\tau_{ni})\right]-13,65} \cdot \Delta\tau_{ni} \, , \tag{20}$$

where: $\Delta\tau_{ni}$ – is the number of days where a temperature T prevails, $T\left(\Delta\tau_{ni}\right)$ – is the temperature in ºC during the time period $\Delta\tau_{ni}$ [2].

For example, for the first three days of maturation of concrete, in which the temperature of the concrete (mean value) were as shown in Fig. 4, the equivalent time is

$$\begin{aligned} \tau_T &= e^{-\left(4000/[273+23,2]-13,65\right)} \cdot 1 + \\ &+ e^{-\left(4000/[273+30,4]-13,65\right)} \cdot 1 + \\ &+ e^{-\left(4000/[273+33,1]-13,65\right)} \cdot 1 = \\ &= 1,2 + 1,6 + 1,8 = 4,6 \end{aligned} \tag{21}$$

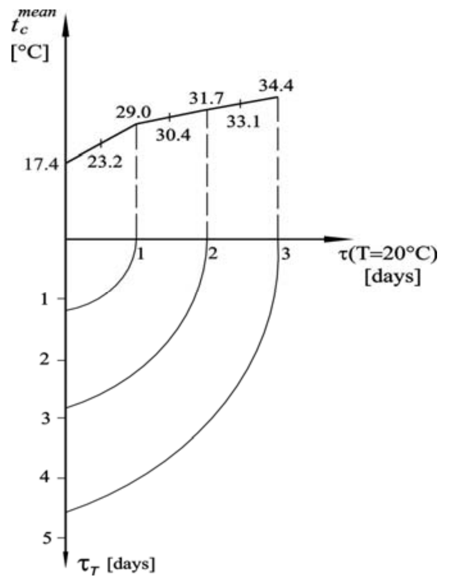

Fig. 4. The temperature of the concrete maturation t_c^{mean} and the temperature adjusted concrete age τ_T

Stress and deformation characteristics for concrete were determined in accordance with EC2 [2] formulas (Tab. 1):

$$E_{cm}\left(\tau_T\right)=\left[\frac{f_{cm}\left(\tau_T\right)}{f_{cm}}\right]^{0,3}\cdot E_{cm}\,; \qquad (22)$$

$$f_{cm}\left(\tau_T\right)=\beta_{cc}\left(\tau_T\right)\cdot f_{cm}\,; \qquad (23)$$

$$f_{ctm}\left(\tau_T\right)=\left[\beta_{cc}\left(\tau_T\right)\right]^{\alpha}\cdot f_{ctm}\,; \qquad (24)$$

$$\beta_{cc}\left(\tau_T\right)=\exp\left\{s\left[1-\left(\frac{28}{\tau_T}\right)^{1/2}\right]\right\}, \qquad (25)$$

where: E_{cm} – is the secant modulus of elasticity of concrete, f_{cm} – is the mean compressive strength at 28 days, f_{ctm} – is the mean value of axial tensile strength of concrete at 28 days, $\beta_{cc}\left(\tau_T\right)$ – is a coefficient which depends on the age of the concrete τ_T, τ_T – age of the concrete in days, $\alpha=1{,}0$ for $\tau_T<28$, s – is a coefficient which depends on the type of cement: $s=0{,}20$ for rapid hardening high strength cements (R) (CEM 42,5R, CEM 52,5), $s=0{,}25$ for normal and rapid hardening cements (N) (CEM 32,5R, CEM 42,5), $s=0{,}38$ for slow hardening cements (S) (CEM 32,5) [2].

After determining the internal forces caused by cumulative action of thermal bending moment and dead loads (using the nonlinear static analysis) the stresses in the phase I_a in top fibers of concrete slab, in critical support sections no. 2, 3, 4, 5 have been calculated (Tab. 2).

Table 1

The secant modulus of elasticity of concrete $E_{cm}\left(\tau_T\right)$ and the mean value of axial tensile strength of concrete $f_{ctm}\left(\tau_T\right)$ in each days of maturation

Age of concrete $\tau_{T'}$ [days]	$E_{cm}\left(\tau_T\right)$ [GPa]	$f_{ctm}\left(\tau_T\right)$ [MPa]
1	27,681	1,094
2	29,143	1,612
3	30,052	1,914
4	30,688	2,121
5	31,166	2,274

Table 2

Exemplary results of stresses in top fibers of the concrete slab for $\tau_{T'}=2$ days and $\Delta t=10, 15, 20$ ºC

Δt [ºC]	$\sigma_{ct}^{top}\left(\tau_T\right)$ [MPa]
10	1,42
15	2,34
20	2,95

Then the value of ratio of the $\sigma_{ct}^{top}\left(\tau_T\right)$ and $f_{ctm}\left(\tau_T\right)$ showing the elastic effort of top fibers of the cross-section was calculated $K\left(\tau_T\right)$ (Fig. 5 – solid line):

$$K\left(\tau_T\right)=\frac{\sigma_{ct}^{top}\left(\tau_T\right)}{f_{ctm}\left(\tau_T\right)}. \qquad (26)$$

The critical range of the stresses is for $K\left(\tau_T\right)\geq 1{,}0$. In Fig. 5 can be seen that for $\Delta t=10$ ºC concrete slab during the thermal shock ($\tau_T=2...3$ days) is not exposed to cracking. For $\Delta t=15$ ºC dangerous period is $\tau_T=1...7$ days. The top of the concrete slab can crack. For $\Delta t\geq 20$ ºC the slab can crack during the 28 days of maturation period.

In first step calculations were performed assuming an elastic work of the concrete until the scratch (index $K\left(\tau_T\right)$). The value of index $K\left(\tau_T\right)$ will be lower due to the elastic-plastic nature of concrete work.

$$K'\left(\tau_T\right)=\frac{K\left(\tau_T\right)}{\lambda}=\frac{K\left(\tau_T\right)}{1.7}=0{,}588K\left(\tau_T\right), \qquad (27)$$

where: λ – is the coefficient corresponding to the border deformation of the concrete equal $\varepsilon_{ct1} = 1,0 \cdot 10^{-4}$.

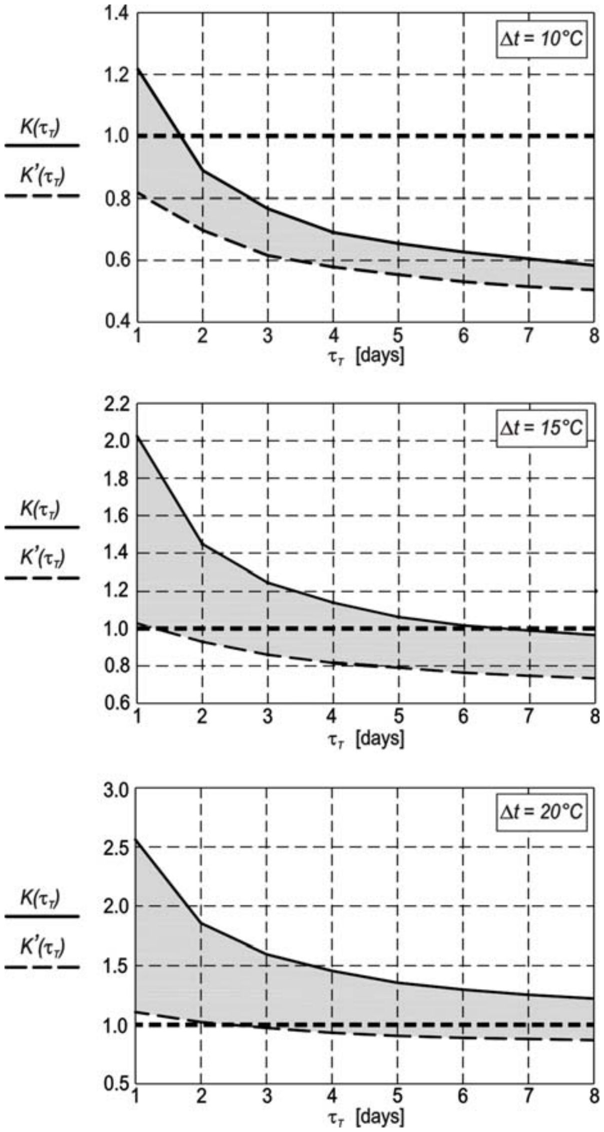

Fig. 5. Values of indexes of elastic $K(\tau_T)$ and elastic-plastic $K'(\tau_T)$ effort of the top fibers in the deck slab for $\Delta t = 10, 15, 20$ ºC

In the early stage of concrete maturity ε_{ct1} can be reduced up to $0{,}4 \cdot 10^{-4}$ and during thermal shock is equal $\varepsilon_{ct1} = (0,7...0,9) \cdot 10^{-4}$ [3, 4]. It means that in period of concrete maturity considered in this article (2–3 days) value $\lambda = 1,7$ should be considered as a top border of reduction coefficient.

In Fig. 5 the dashed lines indicates the elastic-plastic index of slab effort $K'(\tau_T)$ and the region above 1,0 is a critical period of work of the concrete slab.

Values of $K'(\tau_T)$ presented in Fig. 5 indicates that during the temperature difference $\Delta t = 10$ ºC concrete slab is not exposed to thermal cracking, at $\Delta t = 15$ ºC can be damaged by cracking in the most strained regions for $\tau_T < 1,5$ days, and for $\Delta t = 20$ ºC the critical period of the construction work relates to $\tau_T < 2,5$ days and covers the period of the thermal shock. This means that in case of the $\Delta t \geq 15$ ºC in the first 2–3 days of curing of concrete the thermal cracks can occur due to the temperature differences caused by the cumulative impacts of daily temperature variations and self-heating of concrete by the heat of hydration of cement.

In the analyses the impact of creep and shrinkage in early age concrete and susceptibility of the shear connectors have not been taken into account.

Conclusions

In the paper the possibility of cracking of the upper surface of the deck in composite steel-concrete bridges due to temperature differences between concrete slab and steel girders, resulting from daily temperature variations and self-heating of concrete by the heat of hydration in the first days after concreting, has been proved and described.

In characterized composite bridge the temperature difference (mean value) between the concrete slab and steel girders of $\Delta t \geq 15$ ºC was sufficient reason for cracking of the deck slab at critical cross sections over the supports and temperature difference $\Delta t \geq 20$ ºC was a reason of slab cracking on the entire length of the deck, also in span.

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ТЕРМІЧНІ НАПРУГИ У СТАЛЕЗАЛІЗОБЕТОННИХ МОСТАХ У СТАДІЇ БУДІВНИЦТВА

В доповіді обговорюється вплив дії тепла гідратації цементу і температури навколишнього середовища на напруження бетонної плити проїзної частини в сталебетонних мостах. Представлено приклади аналізу термічних умов, а також аналізу напруження перерізу викликане термічним впливом в стадії будівництва моста.

Ключові слова: сталебетонний міст, термічні умови, термічне напруження

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ТЕРМИЧЕСКИЕ НАПРЯЖЕНИЯ В СТАЛЕЖЕЛЕЗОБЕТОННЫХ МОСТАХ В СТАДИИ СТРОИТЕЛЬСТВА

В докладе обсуждается влияние воздействия тепла гидратации цемента и температуры окружающей среды на натяжение бетонной плиты проезжей части в сталебетонных мостах. Представлено примеры анализа термических условий, а также анализа натяжения сечения вызванного термическим влиянием в стадии строительства моста.

Ключевые слова: сталежелезобетонные мосты, термические условия, термическое натяжение