17-osios jaunųjų mokslininkų konferencijos „Mokslas – Lietuvos ateitis“ teminės konferencijos
**TRANSPORTO INŽINERIJA IR VADYBA**,
vykusios 2014 m. gegužės 8 d. Vilniuje, straipsnių rinkinys

Proceedings of the 17th Conference for Junior Researchers ‘Science – Future of Lithuania’
**TRANSPORT ENGINEERING AND MANAGEMENT**, 8 May 2014, Vilnius, Lithuania

Сборник статей 17-й конференции молодых ученых «Наука – будущее Литвы»
**ИНЖЕНЕРИЯ ТРАНСПОРТА И ОРГАНИЗАЦИЯ ПЕРЕВОЗОК**, 8 мая 2014 г., Вильнюс, Литва

# INFLUENCE OF SUPPORT CONDITIONS ON THE STRENGTH OF STEEL-CONCRETE BEAM UNDER THERMAL FORCE IMPACT

**Marina Verevicheva, Glib Vatulia, Andrii Ignatenko**

*Building Mechanics and Hydraulics Department, Ukrainian State Academy of Railway Transport, Feuerbach sq. 7, 61050 Kharkiv, Ukraine*

*E-mail: vatulya@kart.edu.ua*

**Abstract.** The authors proposed the methodology of carrying capacity determination of steel-concrete beams with external steel sheet reinforcement under thermal and power impacts. It’s assumed standard temperature range of fire in the room and instantaneous load. The strength calculations were provided in order to determine bending stress and beams deflection due to support conditions of the beam. The mineral wool coat Rockwool series Conlit SL150 were used as the beam fire protection. Described researches were done taking into account the basic conditions of beams loading and bearing as well as non-linear “stress-strain” dependencies of concrete and external steel reinforcement.

**Keywords:** Steel-concrete beam; structure fire resistance; fire-resistance rating; fire standard temperature conditions; limiting state; carrying capacity; beam deflection.

## Introduction

Widespread implementation of structures with the external reinforcement in practice is hampered by lack of existing standards in design methodologies for strength and carrying capacity calculating under power and thermal impacts. The steel-concrete beams with external steel sheet reinforcement were examined in the article under instantaneous load and standard temperature range of fire. Joint work of concrete layer and the steel sheet reinforcement was ensured by loop anchors.

## Strength calculation methodology

The algorithm for strength calculations of steel-concrete slabs with $1 \times h$ section size at temperature influence (Fig. 1) is considered in (Chikhladze *et al.* 2000; Milovanov 1986).

It’s assumed that fire is located under the slab, cause and fire scenario are not considered. As strength criterion it is considered the occurrence of ultimate bending strength in compressed zone of concrete or yield strength in steel sheet.

The equilibrium equations for examined plates (Fig. 2) in conditions of uniaxial deformation could be written as:

$$\frac{\mathrm{d}^2}{\mathrm{d}x^2}\left(D_1 \frac{\mathrm{d}^2 w}{\mathrm{d}x^2} - M_T\right) = q \,. \tag{1}$$

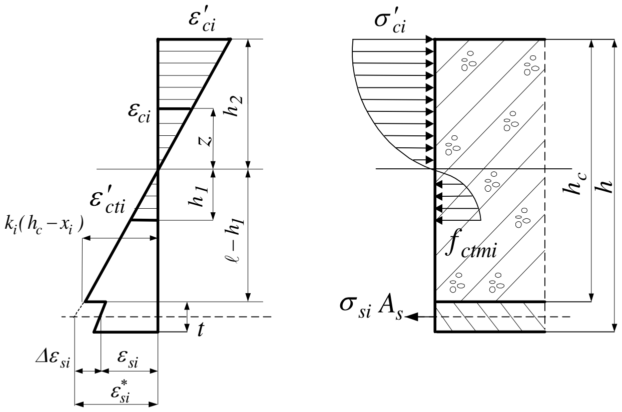


**Fig. 1.** The deformations in cross section of steel-concrete element

The temperature bending moment $M_T$ and rigidity $D_1$ were obtained in assumption that slab width ($b$) is equal to 1 *m*. For slabs calculation with different size let us change the load intensity $q$ on $q_0 = q / b$.

Deflection function for two hinged supported beam could be written as:

$$w(x) = -\frac{q_0}{24D_1}x(L-x)\left(L^2 + Lx - x^2\right) - \frac{M_T}{2D_1}x(L-x)\,. \tag{2}$$

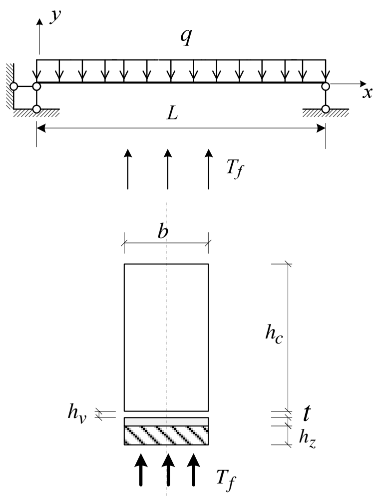


**Fig. 2.** Slab analytical model

The maximum bowing (maximum value of bending moment) take place at the slab middle $x = L/2$:

$$k_* = \max w''(x) = w''(L/2) = \frac{q_0 L^2}{8D_1} + \frac{M_T}{D_1}\,. \tag{3}$$

It is necessary to mention that rigidity coefficient $D_1$ and temperature bending moment $M_T$ are not determined yet, since it is unknown position of neutral surface (dimension $h_1$).

In case of uncracked condition $h_2 = h_c - h_1$, $\ell = 0$, thus we obtain the following equation for the unknown dimension $h_1$:

$$\int_{-h_1}^{h_c - h_1} \frac{E_c}{1-\nu_c^2}\, y\mathrm{d}y + \int_{-(h_1+t)}^{-h_1} \frac{E_s}{1-\nu_s^2}\, y\mathrm{d}y = 0\,.$$

According to (Chikhladze *et al.* 2000), we have the necessary equation for $h_1$:

$$h_1 = \frac{\varepsilon \int_o^{h_0} \beta_c(T) y\mathrm{d}y - \beta_s(T_f)\frac{t^2}{2}}{\varepsilon\beta_c(T_{\mathrm{cold}})h_c + \beta_s(T_f)h_s}\,, \tag{4}$$

where $\varepsilon \approx 0{,}05$, coefficients $\beta_c(T)$ and $\beta_s(T)$ characterize dependence of concrete and steel modulus of elasticity on temperature.

For compression concrete side strength conditions could be written as follows:

$$\sigma_c(y) = \frac{E_c(20°\mathrm{C})}{1-\nu_c^2}\, y w''\left(\frac{L}{2}\right) < f_{cm}(T) = f_{cm}(20°\mathrm{C})\gamma_c(T)\,,$$

$$\frac{E_c(20°\mathrm{C})}{1-\nu_c^2} \cdot \frac{q_0 L^2 + 8M_T}{8D_1}\beta_c(T)|h_2| < f_{cm}(20°\mathrm{C})\gamma_c(T)\,, \tag{5}$$

$$\frac{E_s(20°\mathrm{C})}{1-\nu_s^2} \cdot \frac{q_0 L^2 + 8M_T}{8D_1}\beta_s(T_f)h_1 < \sigma_T(20°\mathrm{C})\gamma_s(T)\,, \tag{6}$$

where coefficients $\gamma_c(T)$ and $\gamma_s(T)$ characterize dependence of concrete working compression stress $f_{cm}$ and steel yield stress on temperature.

Let us consider the beams with other supporting conditions (Fig. 3).

If we examine the beam with restrained and hinged ends (Fig. 3a) the maximum bowing (maximum value of bending moment) takes place in cross section where $x = 0$. Desired value will be equal to those, which we obtained during the previous calculation of hinged supported beam. Due to this the strength conditions are equal to (5) and (6).

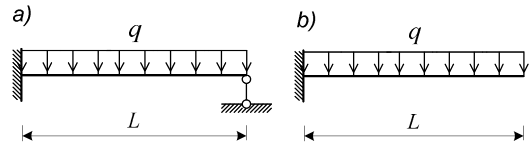


**Fig. 3.** The beams with different supporting conditions:
a) – beam with restrained and hinged ends; b) – cantilever beam

In arbitrary cross section the beam deflection can be calculated as follows:

$$w(x) = \left(-\frac{q_0 L^2}{8} + M_T\right)\frac{x^2}{2D_1} + \frac{5}{8D_1}q_0 L\frac{x^3}{6} - q_0\frac{x^4}{24D_1}\,. \tag{7}$$

We can say that maximum value of bending moment take place in section where $x = 0{,}578L$.

If we examine the cantilever beam (Fig. 3b) the maximum value of bending moment takes place in cross section where $x = L$

$$w(x) = \left(-\frac{q_0 L^2}{2} + M_T\right)\frac{x^2}{2D_1} + \frac{q_0 L}{D_1}\frac{x^3}{6} - q_0\frac{x^4}{24D_1}\,. \tag{8}$$

The maximum bowing in section where $x = 0$ is equal to:

$$\max w''(x) = -\frac{q_0 L^2}{2D_1} + \frac{M_T}{D_1}\,. \tag{9}$$

In this case the strength conditions could be written as follows:

$$\frac{E_c(20°\mathrm{C})}{1-\nu_c^2} \cdot \frac{q_0 L^2 + 2M_T}{2D_1}\beta_c(T)|h_2| < f_{cm}(20°\mathrm{C})\gamma_c(T)\,, \tag{10}$$

$$\frac{E_s(20°\mathrm{C})}{1-\nu_s^2}\cdot\frac{q_0L^2+2M_T}{2D_1}\beta_s(T_f)h_1<\sigma_T(20°\mathrm{C})\gamma_s(T).\quad(11)$$

The design model of steel-concrete beam assumed during the computational investigation is shown at Fig. 4. The beams are made from concrete with strength quality C25/30, the consistency which is equal to 2300 kg/m$^3$ and normal humidity equal to 3%, which is correspond to concrete hardening during the 28 days. The bearing element (steel sheet) is made from structural steel of St.3 grade. Loop anchors were made from reinforcing-bar steel of A240 grade and bar diameter equal to 4 mm. In order to ensure the demanded values of fire-resistance rating of considered beams (DBN V.1.2-7-2008), the mineral wool coat Rockwool series Conlit SL150 were used as the beam fire protection during the calculations. Thickness of fire-retardant coat is 25 mm. The thermal-physic characteristics of fire-retardant coat were taken in accordance with (EN 1992-1-2:2004).

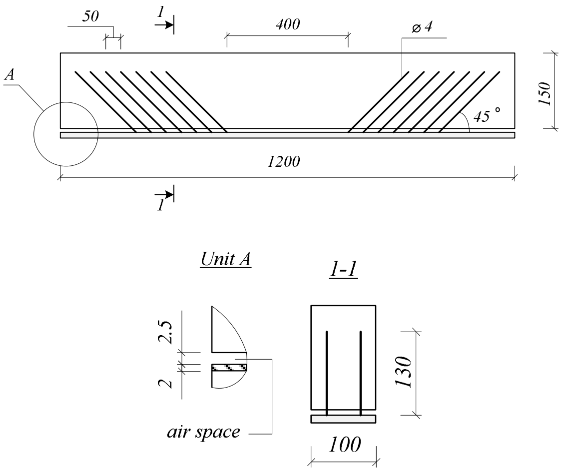


**Fig. 4.** The design model of examined beams with external steel sheet reinforcement

The standard fire temperature conditions were assumed during the computational investigation. The temperature-height graph for described beams after two hours of fire impact, are shown at Fig. 5.

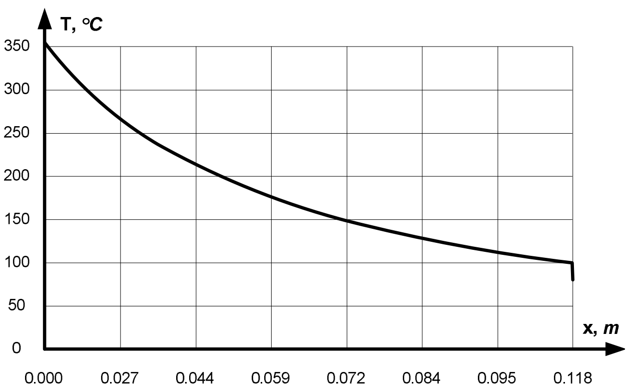


**Fig. 5.** The temperature distribution at 120th minute of fire impact

Analyzing the calculation results it is possible to say that carrying capacity of two hinged supported beam under only power load is $q_{\max}=43\,kPa$; $0.3q_{\max}=12.9$ kPa. In accordance to equations (5) and (6) the residual strength equal to 7 % from $\gamma f_{cm}$ could be ensured after two hours of fire impact. The stresses in steel sheet are less than yield stress in 2–2.5 times. The dependence of maximum beam deflection value from the fire impact period is shown at Fig. 6a.

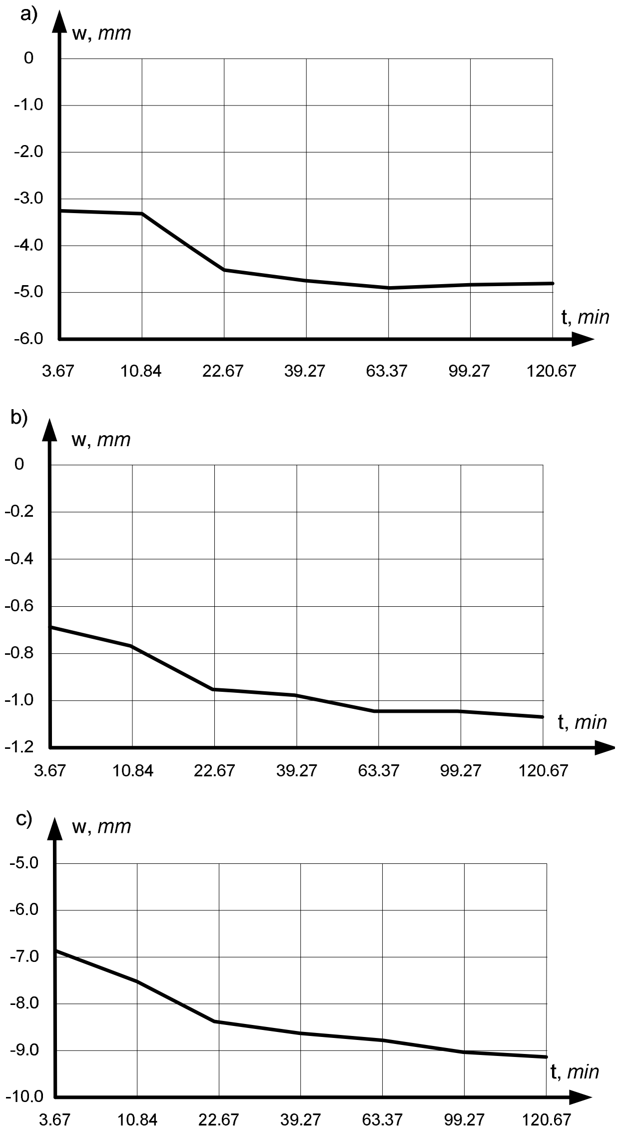


**Fig. 6.** The deflection-temperature diagrams: a) – two hinged supported beam; b) – beam with restrained and hinged ends; c) – cantilever beam

The validity of obtained results were proved in comparison with the experimental results of steel-concrete beams destruction under only the mechanical load (Kovalov 2008) and the results of finite-element modeling in software package ANSYS (Vatulia *et al.* 2014).

We can mentioned, that the maximum bending moment $M_{\max}$ for beam with restrained and hinged ends take place in built-in support and is equal to correspondent moment value for two hinged supported beam. Thus, the fire resistance calculation (strength calculation on first cluster of limiting state) is equal to previous calculation. The beam destruction is not occurred during two hours of fire impact. The dependence of maximum beam deflection value from the fire impact period is shown at Fig.6b.

Analyzing the calculation results it is possible to say that carrying capacity of cantilever beam under only power load is $q_{\max} = 12.3\, kPa$ ; $0.3q_{\max} = 3.69$ kPa . The residual strength equal to 2.3 % from $\gamma f_{cm}$ could be ensured after two hours of fire impact. The dependence of maximum beam deflection value from the fire impact period is shown at Fig.6c.

## Conclusions

Based on verification results we could ensure that proposed calculation methodology can be used during the strength evaluation of steel-concrete beams with external sheet reinforcement under thermal and power impact taking into account different support conditions. It's shown that utilizing the mineral wool fire-retardant coat Rockwool series Conlit SL150 with thickness equal to 25 mm helps to warranty demanded beams fire resistance during 120 minutes.

## References

Chikhladze, E. D.; Zhakin, A. I.; Verevicheva, M. A. 2000. *Fire Resistance of Concrete and Steel-Concrete Structures*. Kharkiv. Kharkivskaya gosudarstvennaya academiya zheleznodorozhnogo transporta, 40, 97 c. (in Russian).

DBN V.1.2-7-2008 *Main Requirements for Structures and Buildings. Fire Safety*, Ukrarhbudinform, Kiev 52 p. (in Ukrainian).

EN 1992-1-2:2004 *Eurocode 2: Design of concrete structures.* Part 1-2: General rules - Structural fire design, Brussels.

Kovalov, M. A. 2008. *Stress-Strained and Limit State of Steel-Concrete Beams under the Short-Term Static Loading*: Dissertacija na soiskanije stepeni kandidata technicheskih nauk (05.23.01 – Stroitelnye kostruktsii, zdaniya i sooruzheniya) Kharkiv: Ukrainskaya gosudarstvennaya academiya zheleznodorozhnogo transporta. 184 c. (in Russian).

Milovanov, A. F. 1986. *Fire Resistance of Reinforce Concrete Structures,* Storjizdat, Moscow, 224p. (in Russian).

Vatulia, G. L.; Orel, Ye. F.; Ignatenko, A. V. 2014. *Evaluation of Carrying Capacity of Steel-Concrete Beams with External Sheet Reinforcement under the Fire Impact*. Kharkiv: Ukrainskaya gosudarstvennaya academiya zheleznodorozhnogo transporta. 144. 154–160 pp. (in Russian).