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## Stress-strain state of composite reinforced concrete slab elements under fire activity

## Напряженно-деформированное состояние фрагмента сталежелезобетонного перекрытия в условиях огневого воздействия

**V.A. Rybakov,**

**I.A. Ananeva,**

*Peter the Great St. Petersburg Polytechnic University, St. Petersburg, Russia*

**A.O. Rodicheva,**

*"LenzhilNIIProekt", Co., Ltd, St. Petersburg, Russia*

**O.T. Ogidan,**

*Peter the Great St. Petersburg Polytechnic University, St. Petersburg, Russia*

**Канд. техн. наук, доцент В.А. Рыбаков,**

**студент И.А. Ананьева,**

*Санкт-Петербургский политехнический университет Петра Великого,*

*г. Санкт-Петербург, Россия*

**инженер А.О. Родичева,**

*ЛенжилНИИпроект, г. Санкт-Петербург, Россия*

**студент О.Т. Огидан,**

*Санкт-Петербургский политехнический университет Петра Великого,*

*г. Санкт-Петербург, Россия*

**Key words:** reinforced concrete structures; composite reinforced concrete structures; profiled sheet; rebar; floor; fire resistance; limit of fire resistance, temperature; ultimate moment

**Ключевые слова:** железобетонные конструкции; температура; сталежелезобетонные конструкции; профилированный лист; арматура; перекрытие; огнестойкость; предел огнестойкости; предельный момент

**Abstract.** One of the most topical issues (in construction) has become providing fire protection facilities for reinforced concrete. The Eurocode-4 and other recommendations have been applied for more than 10 years in European countries. In Russia we have only one method of analysis - STO 36554501-006-2006. This work is devoted to compare all these algorithms. 3 methods of analysis of composite reinforced concrete slab elements under fire activity are described. 3 analytical models of structures with different calculation parameters are considered for determination of corresponding dependences and creating the algorithm of calculation. Partial coefficients were identified for analysis of composite reinforced concrete slab elements according European Standards for application in Russian Federation.

**Аннотация.** Одним из наиболее актуальных вопросов является обеспечение огнестойкости и огнесохранности железобетонных конструкций. В Российской Федерации для расчета сталежелезобетонной конструкции на огневое воздействие существует СТО 36554501-006-2006. В европейских странах уже много лет действует технический кодекс Eurocode-4; институты, занимающиеся конструкциями, издают ряд рекомендаций по расчету. В работе описаны 3 методики расчета сталежелезобетонного перекрытия на огнестойкость, проведено их сравнение. Рассмотрены 3 модели перекрытий с различными параметрами для выявления соответствующих зависимостей и создания алгоритма расчета. Выявлены частные коэффициенты для расчета на огневое воздействие по европейским нормам на территории Российской Федерации.

## *Introduction*

Fire is one of the most horrible disasters, which brings enormous losses and destruction, claiming many lives. [1–4] In the history of mankind, there have been fires that destroyed entire cities (Rome (70, B.C.), London (1666), Moscow (1812), Hamburg (1842), San Francisco (1906)). And these days as indicated by statistics, the number of fires not only does not decrease despite the technical progress but sometimes increases because of them. [5–11]

People since ancient times have searched for solutions to the fires buildings preservation problems, whereas many building materials were highly flammable. The search was crowned with success, when it was discovered

Reinforced concrete was launched by Joseph Monier (1861). In 1938, a progressive method of reinforced concrete strength analysis on destruction stages was obtained by scientists such as A.A. Gvozdev, Y.V. Stolyarov, V.I. Murashev and others. Since 1979, the use of profiled sheet as a retained formwork for construction of monolithic reinforced concrete floor slabs on steel beams started.

Authors R.I. Rabinovich, A.A. Bogdanov, M.G. Karpovskiy, in their researches performed in Promzdaniy Central Research Institute of USSR's Gosstroy and the Donetsk Promstroyniiproekt together with the Concrete and Reinforced Concrete Research Institute, experimentally studied the behaviour of composite slabs in an integrated manner, taking into account reinforced concrete and steel beams of floors.

Any structures must be protected against fire exposure. Therefore, there are requirements about the observance of fire safety. [12–14]

Fire-resistance is a global fire-technical characteristics, regulated with the construction codes and rules such as in Russia (the Federal law of the Russian Federation of 22 July 2008 N 123-FZ "Technical regulations on fire safety requirements"), and in the European Union (Directive 89/106/EC, EN 1991-1-2:2002. Eurocode 1. Impact on Structures. Part 1-2. The General Impact. Exposure to Determine fire Resistance, EN 13501-2:2007+A1:2009. Part 2. Classification Using Test Data on the Combustion Reaction When the Fire Resistance Test, Excluding Ventilation). [15–18, 20–22].

Calculation of steel-concrete composite slabs on profiled sheet as a whole is difficult because it does not have fully-fledged techniques regulated with building codes

In Russia, there is only one method of reinforced concrete structures calculation according to the Guidance to STO 36554501-006-2006 by Milovanov [15].

In European countries, Eurocode 4 "Design of Composite Steel and Concrete Structures. Part 1-2. General Rules. Structural Fire Designs" is used for analyses of fire resistance. [24] There are as well various recommendations, including "The Fire Resistance of Composite Floors with Steel Decking" [25], issued by The Steel Construction Institute.

There is a problem of harmonization of European and Russian standarts establishing General requirements for methods of test for fire resistance of building structures. There is a necessity for a global systematization of computational and experimental methods for fire resistance assessing of building structures and harmonize them with the current regulatory framework in Russian Federation in the fire safety field. [19, 23]

When using the three selected methods were considered their advantages and disadvantages. In the Manual by Milovanov the advantage is presence isofields and isolines, but there are disadvantages that the methodology considered a limited number of beams and narrow examples. There are difficulties in extrapolating the solution of problems. In other codes this is not a problem, but they are not applicable in Russia.

In the paper to determine the possibility of European codes harmonization on the territory of Russia three above-mentioned methods were used: according to the Guidance [15] and two European Standards [24, 25]. These techniques are applicable on the territory of the corresponding countries and therefore can be applied in solving design and construction problems.

The aim of this research was to assess the possibility of using European norms for the analysis of fire resistance of composite reinforced concrete floors and their adaptation in the Russian Federation.

The following objectives were set and addressed in order to achieve the goals:

1. Analysis of the ultimate equilibrium of three fragments of composite reinforced concrete joist floors of differing range of parameters, as described; selection of geometrical and physical characteristics of the structure under a specified load;
2. Analysis of the ultimate equilibrium of the above fragments of floors as described in methods proposed in Eurocode-4 [24]; selection of partial coefficients to maximize the correspondence of methods to each other;
3. Analysis of the ultimate equilibrium of the above fragments of floors as described in the methods proposed in the recommendations of the "The Fire Resistance of Composite Floors with Steel Decking" [25], issued by The Steel Construction Institute.

### *Methods*

For ultimate equilibrium analysis of reinforced concrete slabs according to different design standards there were taken 3 different buildings with different load, the class of functional fire hazard and type of premises.

The limits of fire resistance were taken in accordance with the technical regulations based on the fire resistance of the building:

1. Residential 5-storey building, height – 18m, floor area – 1150 m<sup>2</sup>. It can be concluded from the tables that the grade of fire resistance – I, structural fire hazard class – C1. It follows that the limit of fire resistance of floors for such a building should not be less than REI60.
2. Public office building – 6-storey building, height – 20.5 m, floor area – 2950 m<sup>2</sup>. It can be concluded from the tables that the grade of fire resistance – I and limit of fire resistance of intermediate floors for such a building should not be than REI60.
3. Industrial building – 2-storey building, height – 9.6m, fire danger rating – B, grade of fire resistance – I. Limit of fire resistance of floors for such a building should not be less than REI60.

According to Russian Set of Rules SP 20.13330.2016 (Loads and Impacts), which outlines the normative values of evenly distributed loads depending on the type of room, and loads from floor and equipment, we can say that usually the load on the floor may be the minimum of 1.96 kN/m<sup>2</sup> (if we are not dealing with industrial buildings). In industrial buildings sometimes we have to deal with even more load of 19.6 kN/m<sup>2</sup>, but in this work for the range of loads was taken from 3.63 m (load in corridors, staircases and hallways of residential and public buildings) to 17.75 kN/m<sup>2</sup> (conditional load for industrial buildings in the case when there is no data about the hardware).

One of the important parameters for composite reinforced concrete floor is the profiled sheet, adopted from Russian State Standard GOST 24045-94. [26–30]. Selected profiled sheets are shown in table 1.

Concrete of classes B15 to B30 are usually used when designing slabs. Since the class of concrete has little effect on the method for calculating the structure, class B25 is adopted for each model.

The following frequently used rebar diameters were taken for this research:

- bottom reinforcement (bars) – with a diameter of 8 to 12 mm.
- upper reinforcement (mesh) – reinforcement bar class – Br-1 (3 mm in diameter) and A400 (from 6 to 10 mm in diameter). Mesh spacing respectively: 3 x 50 x 50, 6 x 100 x 100, 10 x 200 x 200.

Continuous reinforcement is laid, as a more technologically advanced option. In the same way the rebar continuity provides continuity for the structure. [31–34].

Thus, the summary table for all three studied models can be provided as follows:

**Table 1. Summary table of the models**

	Model 1	Model 2	Model 3
Support and moment diagram			
Functional purpose	Residential	Administrative	Industrial
Type of room	garret	gym	Hydromechanical transmission repair workshop
Limit of fire resistance	REI60	REI60	REI60
Acting load	3.63 kN/m <sup>2</sup>	8.04 kN/m <sup>2</sup>	17.75 kN/m <sup>2</sup>
Profiled sheet	N57-750-0.9 	N57-750-0.9 	N114-600-0.9 
Concrete grade	B25	B25	B25
Method of reinforcement s and rebars	Upper: BP-1, 3mm mesh, spacing 50x50 Lower: A-III 8mm	Upper: A-III, 6mm mesh, spacing 100x100 Lower: A-III 10mm	Upper: A-III, 10mm mesh, spacing 200x200 Lower: A-III 12mm
Total floor thickness	107mm	150mm	214mm

#### *Analysis of fire resistance according to STO 36554501-006-2006*

The behaviour of reinforced concrete structures under and after standard fire were considered in the "Manual..." of STO, the stress-strain state of structures when briefly exposed to fire before the onset of ultimate fire resistance to loss of carrying capacity were analysed.

The order of analysis of the floors is as follows:

1. Evaluation of fire resistance of the building
2. Evaluation of the bake-out temperature of the section
3. Evaluation of the service factors of the rebars
4. Static analysis of the structure

$$M_u = R_{snt} \cdot A_s \cdot (h_0 - 0.5x) + R_{sct} \cdot A'_s \cdot (0.5x - a') \quad (1)$$

where  $a'$  – compressive reinforcement cover, m;

$R_{snt}$  – tensile strength of the reinforcing steel bars under heating, MPa;

$R_{sct}$  – compressive strength of the longitudinal reinforcing steel bars under heating, Mpa;

5. Conclusion of bearing capacity of the section: comparison of the external and ultimate moments.

$$M_0 < M_u \quad (2)$$

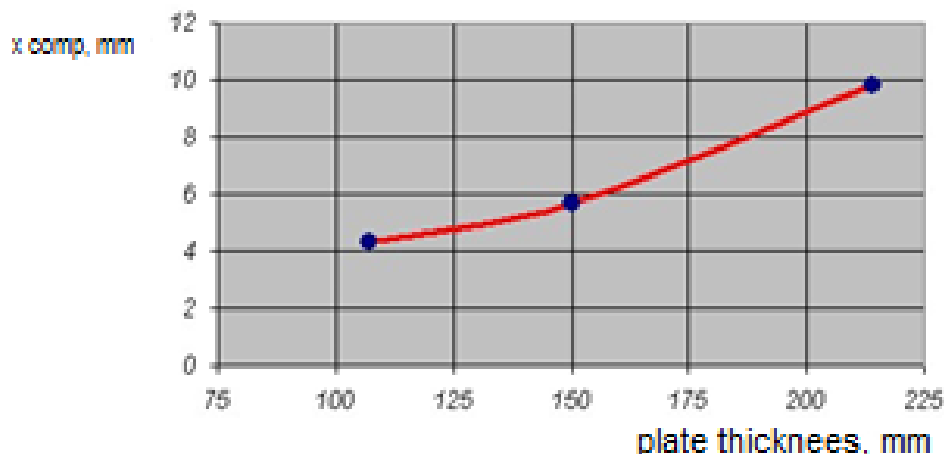
Thus, as a result of the analysis on all models the following data were gotten:

**Table 2. Summary results of the calculation according to STO**

	<b>N57-750-0.9</b>	<b>N57-750-0.9</b>	<b>N114-600-0.9</b>
Ø upper reinforcement	3 mm, 50x50	6 mm, 100x100	10 mm, 200x200
Ø lower reinforcement	8 mm	10 mm	12 mm
$h_{prof}$ (prof. height)	57 mm	75 mm	114 mm
$h'_c$ (comp. zone height)	50 mm	75 mm	100 mm
$h_{given}$ (given height)	85.9 mm	120.9 mm	169.6 mm
$q$ (distributed load)	3.7 KN/m <sup>2</sup>	8.2 KN/m <sup>2</sup>	18.1KN/m <sup>2</sup>
$M_0$ (bending moment)	4.1 KN·m	9.2 KN·m	20.4 KN·m
$A_s$ (cross-section area of the tensile zone)	50.3 mm <sup>2</sup>	78.5 mm <sup>2</sup>	113.1 mm <sup>2</sup>
$A'_s$ (cross-section area of the comp. zone)	21.3 mm <sup>2</sup>	56.6 mm <sup>2</sup>	78.5 mm <sup>2</sup>
$b_{rib}$ (rib width)	118.25 mm	114.75 mm	122 mm
$h_0$ (eff. depth of section)	77 mm	120 mm	179 mm
$h_t$ (eff. height under heating)	71.1mm	113mm	179mm
$x_{comp}$ (height of comp. zone of concrete)	4.3mm	5.7mm	9.8mm
$M_u$ (ultimate bending moment)	4.2 KN·m	9.6 KN·m	29.8KN·m
$M_0 < M_u$	<b>4.1KN·m&lt;4.2KN·m</b>	<b>9.2KN·m&lt;9.6KN·m</b>	<b>20.4KN·m&lt;29.8KN·m</b>

Based on the data from the table, the following conclusions were made:

1. Compressive zone of concrete increases with increasing thickness of the slab in a nonlinear relationship.



**Figure 1. Height of the compressive zone of concrete to slab thickness graph**

2. Effective height under heating increases as well with increasing slab thickness.

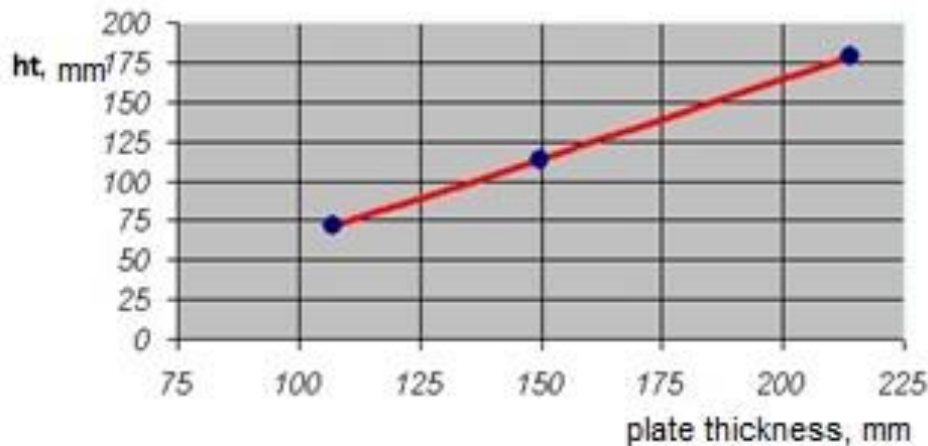


Figure 2. Effective height under heating to slab thickness graph

*Fire resistance analysis according to Eurocode-4 “Design of Composite Steel and Concrete Structures – Part 1-2: General Rules – Structural Fire Design”*

Fire prevention section of Eurocodes (part 2) determines the characteristics of fire resistance when designing structures that must implement the required functions (bearing and/or enclosure) during the set duration of the regulated impact of the fire at the specified load.

The order of analysis of the floor is as follows:

1. The choice of the design characteristics of materials.
2. Analysis of the structure with a choice of coefficients

$$\eta_{fi} = \frac{G_k + \psi_{fi} Q_{k,1}}{\gamma_G G_k + \gamma_{Q,1} Q_{k,1}} \quad (3)$$

where  $\gamma_G$  – partial factor of variable impact;

$\gamma_{Q,1}$  – partial factor of variable impact 1.

3. Evaluation of the ultimate temperature

$$\theta_{lim} = d_0 + d_1 \cdot N_s + d_2 \cdot \frac{A}{L_r} + d_3 \cdot \Phi + d_4 \cdot \frac{1}{l_3} \quad (4)$$

where  $\theta_{lim}$  – limiting temperature [°C];

$N_s$  – axial force in the upper reinforcement [N];

$d_i$  – coefficients, shown in table D.4;

$\Phi$  – projection coefficient of top flange.

4. The analysis of bending moments

$$M = z \cdot N_p \quad (5)$$

where  $z$  – rib-position indicator [mm<sup>-0,5</sup>];

$N_p$  – tensile force, [kN].

5. Conclusion of bearing capacity of the section: comparison of the external and ultimate moments.

$$M_0 < M_u \quad (6)$$

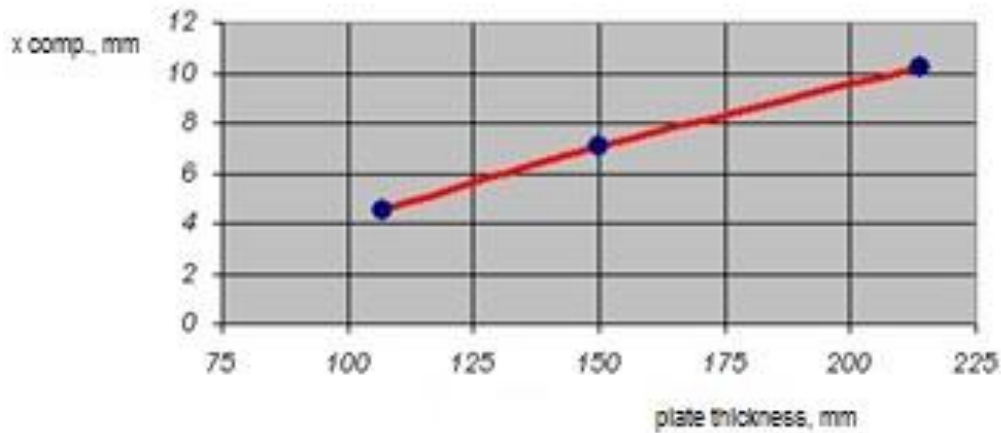
Thus, as a result of the analysis on all models, the following data were gotten:

**Table 3. Summary results of the analysis according to Eurocode-4**

	<b>N57-750-0.9</b>	<b>N57-750-0.9</b>	<b>N114-600-0.9</b>
$\varnothing$ upper reinforcement	3 mm, 50x50	6 mm, 100x100	10 mm, 200x200
$\varnothing$ lower reinforcement	8 mm	10 mm	12 mm
$h_{\text{prof}}$ (prof. height)	57 mm	75 mm	114 mm
$E_d$ (calc. impact)	5.1 KN/m <sup>2</sup>	11.67 KN/m <sup>2</sup>	26.24 KN/m <sup>2</sup>
$h_{\text{eff}}$ (eff. height)	85.9 mm	120.9 mm	169.6 mm
$G_c$ (stand. value of const. impact)	3.0 KN/m <sup>2</sup>	4.2 KN/m <sup>2</sup>	6.1 KN/m <sup>2</sup>
$Q_c$ (stand. value of main impact)	0.7 KN/m <sup>2</sup>	4.0 KN/m <sup>2</sup>	12.0 KN/m <sup>2</sup>
$M_0$ (bending moment)	3.97 KN·m	8.89 KN·m	19.01 KN·m
$A_s$ (cross-section area of the tensile zone)	50.3 mm <sup>2</sup>	78.5 mm <sup>2</sup>	113.1 mm <sup>2</sup>
$A'_s$ (cross-section area of the comp. zone)	21.3 mm <sup>2</sup>	56.6 mm <sup>2</sup>	78.5 mm <sup>2</sup>
$\eta_{fi}$ (coeff. of reduction)	0.66	0.6	0.64
$\psi$ (combination factor)	0.5	0.7	0.9
$\frac{A}{L}$ (given rib thickness)	30.96 mm	34.5 mm	41.6 mm
$\theta_{top}$ (top flange temp.)	635.8 C	598.3 C	514.1 C
$\theta_{mid}$ (mid flange temp.)	748.7 C	726.8 C	677.3 C
$\theta_{bot}$ (bottom flange temp.)	847.5 C	850.8 C	845.3 C
$\theta_{rod}$ (rod temp.)	368.7 C	380.9 C	342.4 C
$x_{\text{comp}}$ (height of comp. zone of concrete)	4.53 mm	7.1 mm	10.2 mm
$M_u$ (ult. moment)	4.13 KN·m	9.24 KN·m	24.85 KN·m
$M_0 < M_u$	<b>3.97KN·m&lt;4.13KN·m</b>	<b>8.89KN·m&lt;9.24KN·m</b>	<b>19.01KN·m&lt;24.85KN·m</b>

Based on the data from the table, the following graphs were plotted:

1. Compressive zone of concrete increases with increasing slab thickness in a curvilinear relationship, the numerical values are close to the values obtained in the analysis according to STO



**Figure 3. Height of the compressive zone of concrete to slab thickness graph**

*Fire resistance analysis according to the recommendations of “The Fire Resistance of Composite Floors with Steel Decking”, issued by The Steel Construction Institute*

The order of analysis of the floor is as follows:

1. The choice of the design characteristics of materials.
2. Analysis of the bending moments for continuous structures,

$$M_0 = \frac{L^2 \cdot (\gamma_{fd} \cdot W_d + \gamma_{fi} \cdot W_i)}{8} \quad (7)$$

where  $L$  – span, [m];

$\gamma_{fd}, \gamma_{fi}$  – load safety factor;

3. Conclusion of the compliance of limit of fire resistance and the need for additional reinforcement.

Thus, as a result of the analysis on all models, the following data were gotten:

**Table 4. Summary results of the analysis according to Eurocode-4**

	N57-750-0.9	N57-750-0.9	N114-600-0.9
Ø upper reinforcement	3 mm, 50x50	6 mm, 100x100	10 mm, 200x200
Ø lower reinforcement	8 mm	10 mm	12 mm
$h_{\text{PROF}}$ (prof. height)	57 mm	75 mm	114 mm
$W_d$ (floor load)	3.0 KN/m <sup>2</sup>	4.2 KN /m <sup>2</sup>	6.1 KN /m <sup>2</sup>
$W_i$ (live load)	0.7 KN /m <sup>2</sup>	4.0 KN /m <sup>2</sup>	12.0 KN/m <sup>2</sup>
$M_0$ ( bending moment)	4.1 KN·m	9.2 KN·m	20.1 KN·m
$d_{\text{COMP}}$ ( comp. zone of concrete)	4.9 mm	7.8 mm	11.2 mm
$M_U$ ( ultimate bending moment)	4.2 KN ·m	10.4 KN·m	25.5 KN·m
$M_0 < M_U$	<b>4.1KN·m&lt;4.2KN·m</b>	<b>9.2KN·m&lt;10.4KN·m</b>	<b>20.1KN·m&lt;25.5KN·m</b>

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Based on the data from all models, the following conclusions can be made:

1. The curve of the compressive zone height to the thickness of the floor is a straight-line, when the thickness of the slab increases, the thickness of the compressive zone of the slab increases respectively

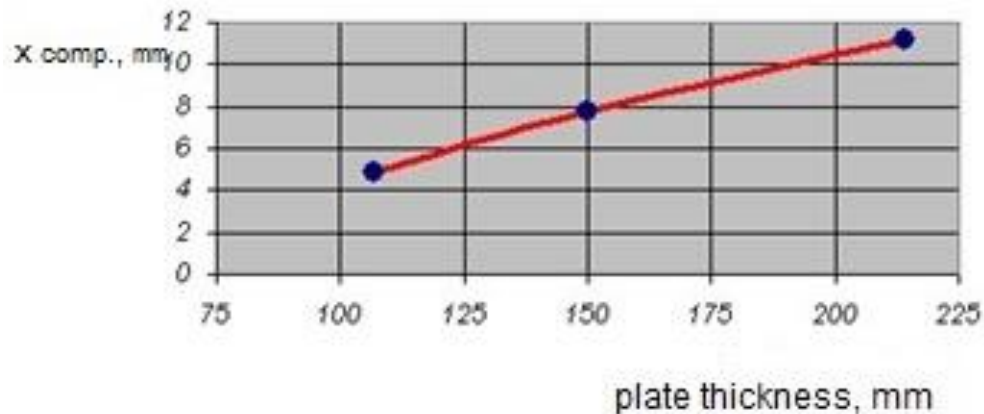


Figure 4. Height of compressive zone of concrete to slab thickness graph

### Results and Discussion

To prove the reliability of the results obtained in the calculation for the three above-mentioned methods, the model was calculated by the finite element method (FEM)

The calculation was carried out in the SCAD Office software (version 11.3). For calculation, the third model of the slab with the profiled sheet H114-600-0,9 for the industrial building was adopted. The thickness of this slab is 214 mm.

When calculating by the FEM, the profiled sheet was not modeled. It was taken into account that the stresses of the profiled sheet reach the yield point of steel in conditions of fire action.

Model:

Number of nodes: 252 630;

Number of elements: 224 800;

The size of grid is 15mm;

Boundary conditions - fixing by the X, Y, Z-axes at the beginning, middle and end of the slab.

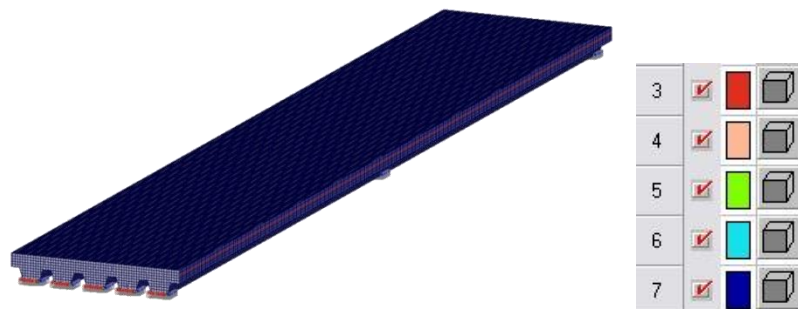
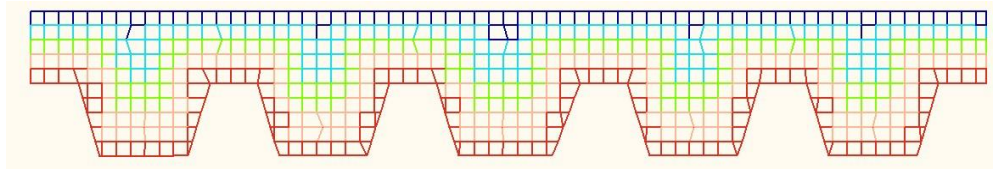


Figure 5. Model of the slab fragment for calculation according to FEM  
3 types of elements:

- bars;
- 6-node isoparametric finite elements;
- 8-node isoparametric finite elements.



**Figure 6. The partitioning into finite elements in cross section of the slab**

Fire is a consequence of designed emergency case, according to 1.5.2.5 EN 1990. Possibility of deformational analysis according to Russian and foreign normative documents is considered.

Value of variable force for designed emergency case must be installed pursuant to EN 1990, according to appendix 4.2.1 of EN 1991-1-2. However, as it is stated in A.1.4. appendix, limit state is normed by second group of limit states that does not include any emergency cases such as fire. Emergency load is considered only in bearing capacity calculation (appendix A.1.3). Thus, structural normative documents do not regulate analysis of deformations in fire case. Also, Russian Set of Rule SP.20.13330.2011 and [15] do not consider fire impact deformation analysis.

However, limit state takes place if deformation value reaches 5% of span value, according to Russian State Standard GOST 30247.1.

$$f_{max} = \frac{l}{20} = \frac{3000}{20} = 150\text{mm} \tag{8}$$

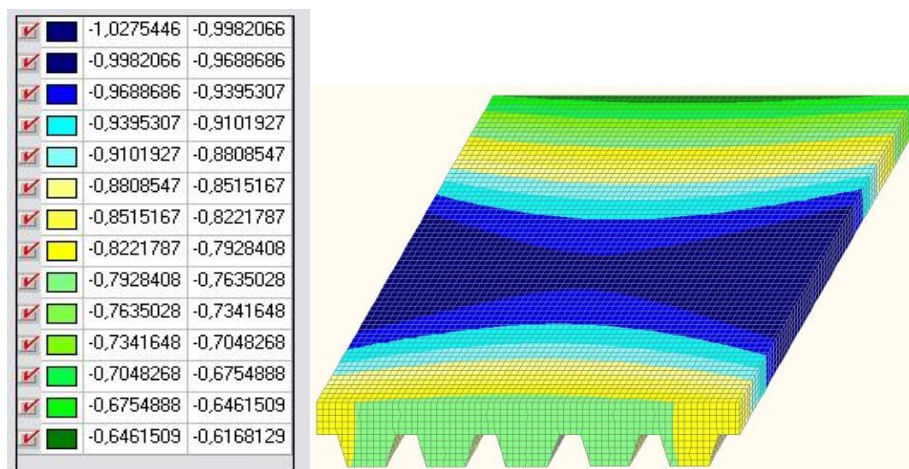
$$f_{slab} = 1.03\text{mm} \tag{9}$$

$$f_{slab} < f_{max} \tag{10}$$

where,  $f_{max}$  – ultimate slab information [mm]

$f_{slab}$  – slab deformation, [mm];

$l$  – length of plate, [mm];



**Figure 7. The displacements isofields results by Z-axis of the half-plate (mm)**

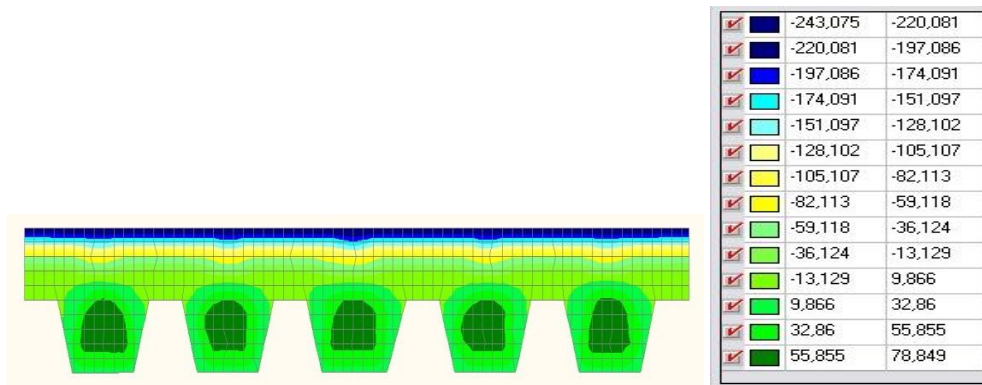
As all the deflections of the slab are much less than the ultimate, it can be concluded that the steel-reinforced concrete slab should be calculated according to the first group of ultimate fire resistance states only (in strength), because in the calculation of the second group of limiting states the rigidity conditions are satisfied automatically.

The condition for the strength of the slab can be expressed as follows:

$$\sigma_{max} < R_{bnt} \tag{11}$$

where  $\sigma_{max}$  – maximum normal stress [MPa];

$R_{bnt}$  – characteristic concrete resistance to compression under fire activity, [MPa]



**Figure 8. The cross-section normal stresses isofields of the middle span cross-section of the plate (10-2 MPa)**

According to this strength calculation, it is possible to draw the conclusions:

- When finite elements method calculating in the elastic stage, the neutral line is sharply shifted;
- The elasto-plastic stage of the concrete operation in the fire activity occurs much earlier than under normal use conditions;
- It is not advisable to calculate the steel reinforced concrete slab for the fire action in the elastic stage.

Also the following strength calculating condition must be satisfied:

$$\sigma_{max} = N_{max} / A_s < R_{snt} = R_{sn} * \gamma_{st} \tag{12}$$

where  $N_{max}$  – maximum tensile force in the reinforcement bar, [kg];

$A_s$  – reinforcement bar area, [mm<sup>2</sup>];

$R_{snt}$  – characteristic tensile resistance of the reinforcement bar under fire activity, [MPa];

$R_{sn}$  – characteristic tensile resistance of the reinforcement bar, [МПа];

$\gamma_{st}$  – condition load effect factor of the reinforcement bar

Thus, the strength calculation results are shown in Table 5.

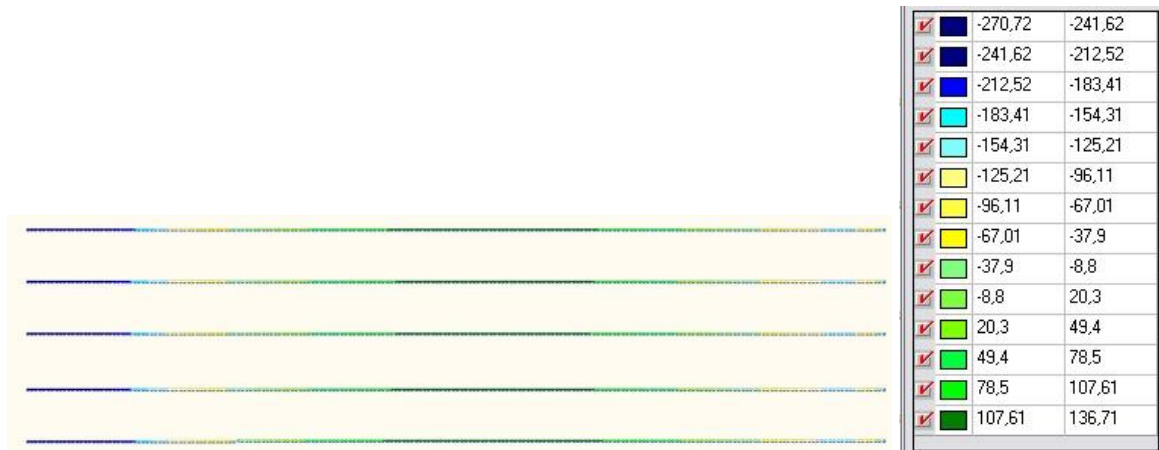
**Table 5. Results of strength calculating of the slab**

reinforcement	Nmax, daN	As, mm2	σmax	Rsn, MPa	γst	Rsnt, MPa	Conclusion
top	20	28.3	7.1	400	1.0	400	$\sigma_{max} < R_{snt}$
bottom	271	78.5	34.5		0.63	252	$\sigma_{max} < R_{snt}$

When calculating, the upper and lower reinforcement were considered for compliance with the strength condition.



**Figure 9. Top reinforcement bar axial force isofields (daN)**



**Figure 10. Bottom reinforcement bar axial force isofields (daN)**

As can be seen from figures 8, 9, 10 and table 5 stresses in concrete and reinforcement within the allowable. Thus, the slab retains its bearing capacity during fire exposure for 60 minutes.

## Conclusions

In the course of this work, the following conclusions were made:

1. It was shown that the compressive zone of concrete is located above the rebars, which means that the floor is a heavily reinforced structure at the time of fire;
2. It was shown that the analysis according to Eurocode-4 corresponds with the analysis according to Milovanov's manual to STO which is confirmed by close results of analysis of external and ultimate moments when exposed to fire;
3. It was shown that the recommendations of the Steel Construction Institute are identical to the fire prevention section of the Eurocode-4 which is confirmed by the same approach of analysis of the positive bending moments and the same results when analyzing the negative moments;
4. It is suggested to use the following partial coefficients of variable impact for the analysis of fire resistance according to Eurocode in the Russian Federation: for dead loads – 1.35, for live loads – 1.5;
5. Reliability of the calculation results for the three methods (Manual for STO, Eurocode, "Recommendations...") is confirmed by the results obtained by the finite element method.

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Vladimir Rybakov,  
+7(964)3312915; fishermanoff@mail.ru

Irina Ananeva,  
+7(931)9714391; irina.ananeva94@yandex.ru

Anna Rodicheva,  
+7(951)6542356; vse\_zdes@mail.ru

Olamipe Ogidan,  
+7(911)0967745; ogidano@gmail.com

Владимир Александрович Рыбаков,  
+7(964)3312915; эл. почта: fishermanoff@mail.ru

Ирина Александровна Ананьева,  
+7(931)9714391;  
эл. почта: irina.ananeva94@yandex.ru

Анна Олеговна Родичева,  
+7(951)6542356; эл. почта: vse\_zdes@mail.ru

Оламипе Тимоти Огидан,  
+7(911)0967745; эл. почта: ogidano@gmail.com

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