SCIENTIFIC
AND DIDACTIC EQUIPMENT

Computational assessment of reinforced concrete slab load bearing capacity in a span in a fire situation

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ABSTRACT:

The resistance of reinforced concrete slabs, with a thickness of not less than 15 cm, in a fire situation, has been calculated in the document. A standard fire curve was adopted. The calculations have been performed using the simplified Isotherm 500 method. Equilibrium curves that facilitate determination of the fire resistance of the analysed slabs, have been established. The results have been tabulated for use by designers.

Obliczeniowa ocena nośności płyt żelbetowych w strefie przęsłowej w warunkach działania pożaru

Słowa kluczowe: płyta, żelbet, nośność, pożar

STRESZCZENIE:

W pracy obliczono nośność pożarową płyt żelbetowych o grubościach nie mniejszych niż 15 cm. Przyjęto standardową krzywą pożarową. Obliczenia wykonano przy pomocy metody Izoterma 500. Wyznaczono krzywe równowagi pomocne do wyznaczenia nośności ogniowej analizowanych płyt. Jako pomoc dla projektantów wyniki zebrano w formie tabelarycznej.

INTRODUCTION

In professional practice, a designer is required to ensure appropriate fire resistance of a structure in a fire situation. The following methods are proposed to meet the fire resistance requirements for EC2 designed buildings [6]:
– advanced computational models,
– simplified computational models for separate structural elements, such as the Isotherm 500 method or the zone method,
– the tabular method.
The tabular method is the most common. According to it, fire resistance is ensured by maintaining set minimum dimensions for the cross-section and the distance of the main reinforcement's centre of gravity from the slab face.
Isotherm 500 [4, 6] allows assessment of the fire resistance of any structural element, which is impossible with tabular restrictions compliant with EC2. The study verified numerous parameters affecting the load-bearing capacity of a structure in a fire situation, such as:
– the distance of the main reinforcement's centre of gravity from the slab face - *a*,
– reinforcement ratio -α,
– calculated load reduction factor - η_{fi}.
The objective of the study is to analyse the calculated results, compare them with tabular data specified as safe according to EC2 and create more precise load-bearing capacity tables. This will allow more economical results from the perspective of an engineer and investor to be obtained.

ASSUMPTIONS OF THE ISOTHERM 500 METHOD

An analysis based on the isotherm 500°C method was used to calculate the resistance.
The assumptions of this method are as follows [6]:
– reduction of the steel yield point depending on the calculated temperature in the reinforcement's centre of gravity,
– reduction of the concrete compression zone as determined by the isotherm 500°C range. It has been assumed that concrete in the reduced zone demonstrates compressive strength as per EC2 under normal conditions,
– fire according to the standard fire curve.

Additional restrictions placed by the authors:
– The analysis pertained only to sections reinforced with $f_{yk} = 500\ MPa$ steel, with appropriate ductility allowing deformations of not less than 2% to be obtained at temperatures exceeding $\theta \geq 200^{o}C$;
– Concretes with compressive strength up to $f_{yk} \leq 50\ MPa$ were analysed;
– Rectangular cross-sections with a thickness of $h \geq 15\ cm$ have been assumed;
– It has been assumed that floor layers provide protection to slabs against fire from above. Screed significantly increases an element's passive protection against the undesirable effects of fire;
– In a fire situation, destruction of the slab due to steel yielding to tensile stress or concrete crushing has been allowed,
– Design load under fire conditions has been reduced using the η_{fi} factor as per the EC2 [6].
A significant increase in steel deformation caused by its high temperature reduces the limit range of the compressed zone. Therefore, in the case of high reinforcement ratios, the cross-section can be destroyed in a fire situation by exhausting the concrete's load-bearing capacity. The authors allow for such a situation, as omitting it would result in underestimation of the load-bearing capacity of the structure, which under normal circumstances is destroyed by steel yielding to tensile stress, i.e. indicating it is properly designed.

TEMPERATURE IN THE REINFORCEMENT

To calculate the load-bearing capacity, it is necessary to determine the temperature at the centre of gravity of the designed slab reinforcement. Numerical simulations of heated reinforced concrete slabs from document [2] were used to obtain the temperature values in the tensioned reinforcement for slabs with a minimum thickness of 15 cm. The temperature in the reinforcement of the reinforced concrete cross-section is dependent on the distance of the main reinforcement's centre of gravity from the slab face, which is illustrated below (Fig. 1).

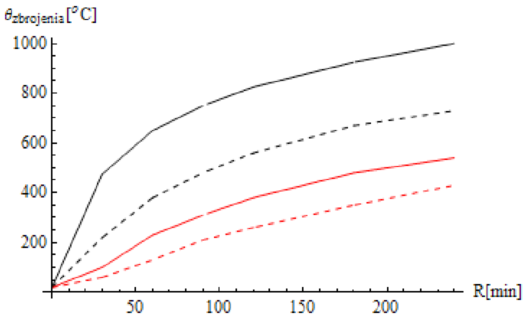

Figure 1 Dependence of temperature in the reinforcement's centre of gravity on the duration of the fire for different distances from the heated face of the slab, see [2]

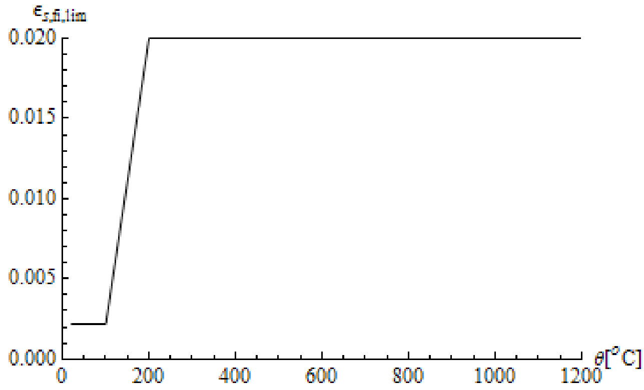

Figure 3 Maximum plastic deformation of steel in a fire situation

STEEL DEFORMATION

According to study [1], assuming steel deformation limits in accordance with EC2 and omitting thermal deformations may lead to the fire resistance of reinforced concrete elements to be overestimated. Therefore, temperature-induced deformations were added to limit values for steel deformations: $\mathcal{E}_{s,fi,\theta,\lim} = \mathcal{E}_{s,\theta} + \mathcal{E}_{s,fi,\lim}$.

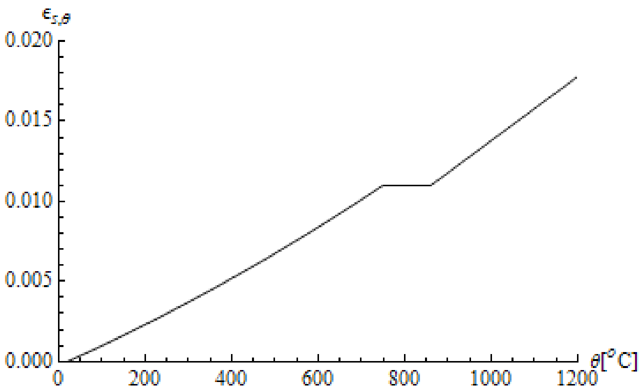

Figure 2 Thermal deformations of steel in fire conditions

The total thermal elongation for ordinary reinforcing steel according to EC2 is presented in Figure 2. As the temperature increases, the maximum plastic deformations resulting from forces in the reinforcing steel change. For temperatures below 100°C, the maximum plastic deformation is 0.2%. At temperatures exceeding 200°C, the maximum plastic deformation is 2%. Linear interpolation was adopted for intermediate situations (Fig. 3).

FACTORS REDUCING THE STRENGTH OF MATERIALS

Concrete

According to the assumptions of the Isotherm 500 method, the reduction of concrete load capacity is achieved by reducing the concrete cross-section in places where it reaches a temperature exceeding 500°C. For an area with a temperature $\theta < 500^{o}C$ it has been assumed that $f_{cd,fi} = f_{ck}$, while for areas where $\theta \geq 500^{o}C$ it has been assumed that $f_{cd,fi} = 0$.

Steel

The decrease in the strength of the reinforcement was taken into account by using a temperature-dependent coefficient reducing the characteristic yield strength of steel $f_{yd,fi} = k_{s\theta} f_{yk}$. According to EC2 recommendations, the value of the $k_{s,\theta}$ factor has been adopted in accordance with Table 1 [6]. As mentioned earlier in the paper, cases in which the cross-section's capacity is exceeded due to concrete crushing will also be analysed. In such a design situation, the deformations in steel do not reach the limit deformations, and therefore the stresses in the steel are lower than the plastic yield point of the tested reinforcement. The following diagrams were used to determine the design value of stresses in the reinforcement when the load capacity is exhausted, as shown in Figure 4 [1].

Table 1 $k_{s\theta}$ factor depending on the temperature

T[°C]	20	100	200	300	400	500	600	700	800
$k_{s\theta}$	1.00	1.00	1.00	1.00	1.00	0.78	0.47	0.23	0.11

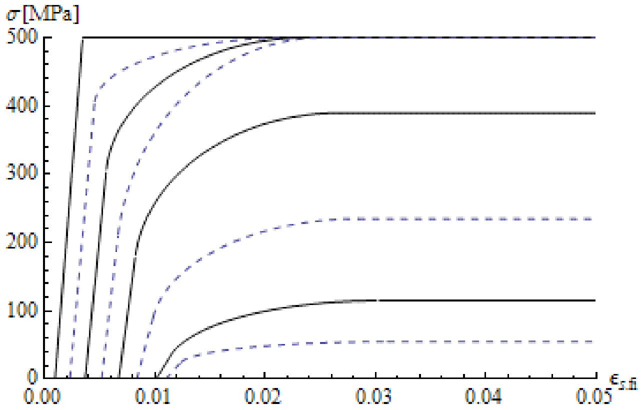

Figure 4 Dependence for different temperatures of reinforcing steel (Solid lines from the top – 100/300/500/700°C, lines dashed from the top – 200/400/600/800°C) [1]

It should be noted that Table 1 defining the reduction coefficient $k_{s\theta}$ and dependencies $\sigma(\varepsilon)$ can be used in the case of cold-rolled steels, with appropriate ductility, which makes it possible to achieve deformations of not less than 2% at temperatures exceeding $\theta \geq 200^{o}C$.

CROSS-SECTION REDUCTION

In study [2], once a numerical analysis of the most frequently used reinforced concrete bent elements has been conducted, arbitrary values a_z – the location of Isotherm 500 in reinforced concrete slabs – were recommended. In effect, in case of slabs with a thickness of $h \geq 15\ cm$ the location of isotherm 500 is actually independent of the tested element's thickness (Fig. 5).

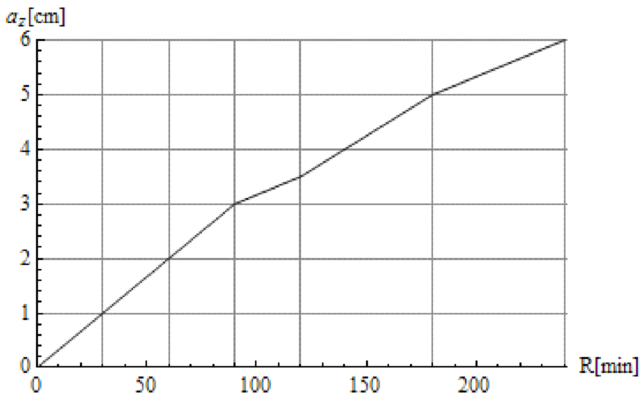

Figure 5 Location of the Isotherm 500; dependence $a_z(t)$ [2]

RESISTANCE OF THE SLABS

Computational analysis was conducted for the most common floor slabs with a thickness greater than 15 cm. Values *a* = 2 cm, 3 cm, 4 cm, 5 cm, where *a* is the distance between the reinforcement's centre of gravity from the slab face, have been assumed.

COMPUTATIONAL LOAD REDUCTION FACTOR

A fire load is counted among exceptional loads on a structure. When determining the strain on a structure exposed to fire, the ratio of the design load under fire conditions to the design load under normal conditions n_{fi}.

EC2 [6] recommends that $\eta_{fi} = 0.7$ be adopted when the value of the coefficient is unknown or difficult to determine. This is a conservative, safe estimate. Adopting smaller values for reduction factors may increase the fire resistance class of the designed building. Smaller values were also included in the final tables [Tab. 2].

Table 2 Fire resistances

Fire resistance for a = 2 cm						
n_f	**maximum α**					
	R30	R60	R90	R120	R180	R240
0.7	0.5	0.44	0	0	0	0
0.6	0.5	0.5	0	0	0	0
0.5	0.5	0.5	0.006	0	0	0
0.4	0.5	0.5	0.5	0	0	0
0.3	0.5	0.5	0.5	0.122	0	0

Fire resistance for a = 3 cm						
n_f	**maximum α**					
	R30	R60	R90	R120	R180	R240
0.7	0.5	0.5	0.5	0.092	0	0
0.6	0.5	0.5	0.5	0.494	0	0
0.5	0.5	0.5	0.5	0.5	0	0
0.4	0.5	0.5	0.5	0.5	0.336	0
0.3	0.5	0.5	0.5	0.5	0.5	0

Fire resistance for a = 4 cm						
n_f	**maximum α**					
	R30	R60	R90	R120	R180	R240
0.7	0.5	0.5	0.5	0.5	0	0
0.6	0.5	0.5	0.5	0.5	0.401	0
0.5	0.5	0.5	0.5	0.5	0.5	0
0.4	0.5	0.5	0.5	0.5	0.5	0.5
0.3	0.5	0.5	0.5	0.5	0.5	0.5

Fire resistance for a = 5 cm						
n_f	maximum α					
	R30	R60	R90	R120	R180	R240
0.7	0.5	0.5	0.5	0.5	0.5	0.34
0.6	0.5	0.5	0.5	0.5	0.5	0.5
0.5	0.5	0.5	0.5	0.5	0.5	0.5
0.4	0.5	0.5	0.5	0.5	0.5	0.5
0.3	0.5	0.5	0.5	0.5	0.5	0.5

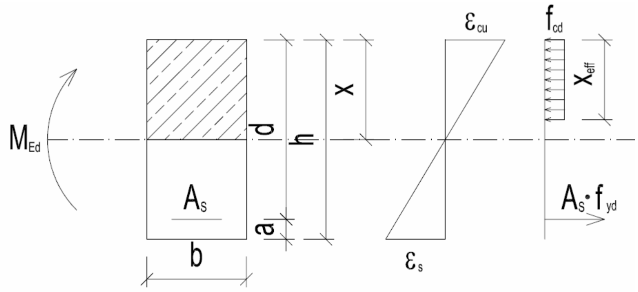

Figure 6 Forces in a section for a normal situation

LOAD-BEARING CAPACITY OF THE SLAB IN A NORMAL SITUATION

The ultimate bending strength was calculated according to the standard [5], using rectangle approximation on the diagram of stresses in the concrete's compression zone. The equations for the balance of forces and moments (Fig. 6) are as follows [3]:

$$bx_{eff} f_{cd} = A_s f_{yd} \tag{4}$$

$$M_{Rd} = x_{eff} f_{cd} (d - \frac{x_{eff}}{2}) \tag{5}$$

After switching to dimensionless values, we obtain:

$$m_{Rd} = \alpha (1 - \frac{\alpha}{2}) \tag{5}$$

where:

$m_{Rd} = \dfrac{M_{Rd}}{f_{cd} bd^2}$ – dimensionless resistance of the cross-section to bending,

$\alpha = \xi_{eff} = \dfrac{x_{eff}}{d} = \dfrac{A_s f_{yd}}{bdf_{cd}}$ – reinforcement ratio [3].

In subsequent considerations it has been assumed that $m_{Ed} = m_{Rd}$. Reducing the force by the η_{fi} factor results in a dimensionless load in a fire situation of $m_{Edfi} = \eta_{fi} m_{Ed}$.

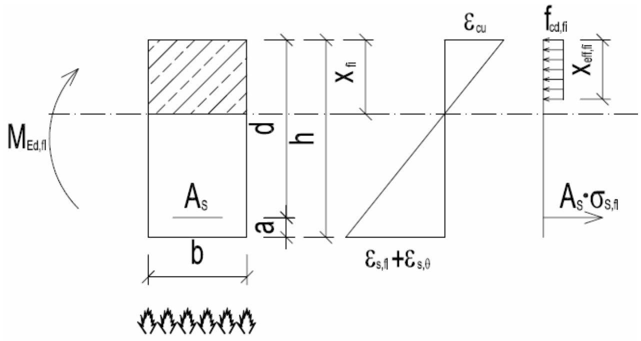

Figure 7 Forces in a section for a fire situation

RESISTANCE OF THE SLAB IN A FIRE SITUATION

Similarly, for a fire situation (Fig. 7):

$$bx_{eff} f_{cd,fi} = A_s \sigma_{s,fi}\,; \quad \sigma_{s,fi} = pf_{yd,fi}\,; \tag{7}$$

$$f_{yd,fi} = k_{s,\theta} f_{yk}$$

$$M_{Rd,fi} = x_{eff,fi} f_{ck} (d - \frac{x_{eff,fi}}{2}) \tag{8}$$

After switching to dimensionless values, we obtain:

$$m_{Rd,fi} = \alpha \gamma_s pk_{s,\theta} (1 - \frac{1}{2} \frac{\gamma_s}{\gamma_c} pk_{s,\theta}) \tag{9}$$

where:

$m_{Rd,fi} = \dfrac{M_{Rd,fi}}{f_{cd} bd^2}$ – dimensionless resistance of the cross-section to bending in a fire situation,

$\sigma_{s,fi}$ – stresses in the reinforcement in a fire situation,

p – reinforcement ratio use in a fire situation, $p \in \langle 0{,}1 \rangle$,

γ_c – concrete partial factor under normal conditions (according to EC2 = 1.4),

γ_s – steel partial factor under normal conditions (according to EC2 = 1.15).

The following set of equations was solved to determine *p*:

$$\begin{cases} \varepsilon_{s,fi} + \varepsilon_{s,\theta} = \dfrac{\varepsilon_{cu}(1-\xi)}{\xi} \\ \sigma_{s,fi} = pf_{yd,fi} \\ \xi_{eff,fi} = \alpha \dfrac{\gamma_s}{\gamma_c} pk_{s,\theta} \\ \sigma_{s,fi} = f(\varepsilon_{s,fi} + \varepsilon_{s,\theta}) \end{cases} \tag{10}$$

ANALYSIS RESULTS

Solving the system of equations (10) has resulted in resistance diagrams (Figs. 8-11) for the distance of the centre of gravity to the slab face *a* = 2 cm, 3 cm, 4 cm and 5 cm. The horizontal axis indicates dimensionless reinforcement ratios, while the vertical axis denotes the dimensionless resistance of the cross-section. If the fire resistance diagram (R30, R60, R90, R120, R180, R240) is above the load diagram, this indicates that a particular cross-section has at least this level of fire resistance.

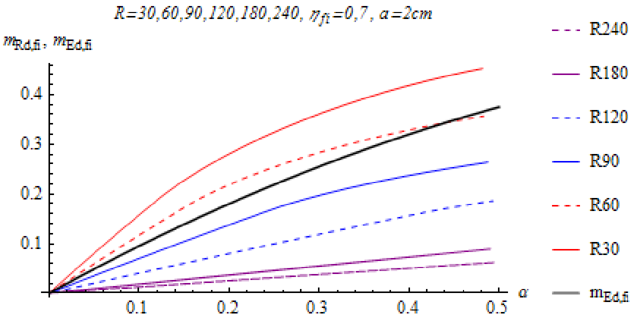

Figure 8 Fire resistance for a = 2 cm

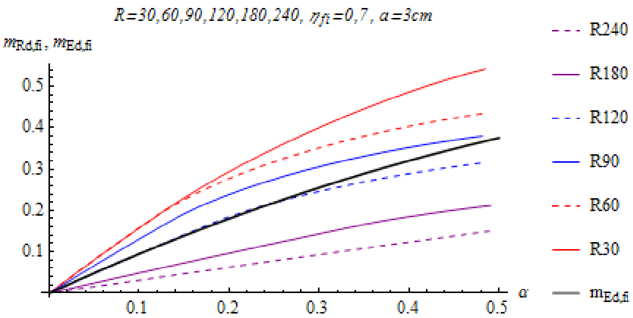

Figure 9 Fire resistance for a = 3 cm

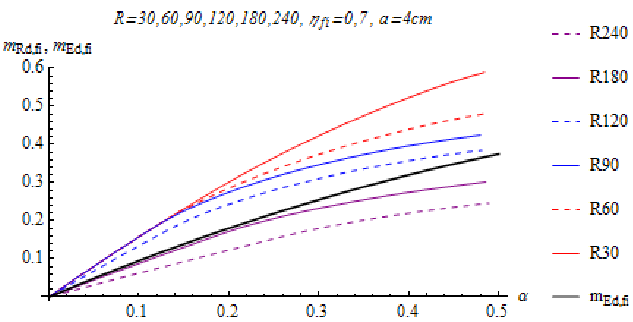

Figure 10 Fire resistance for a = 4 cm

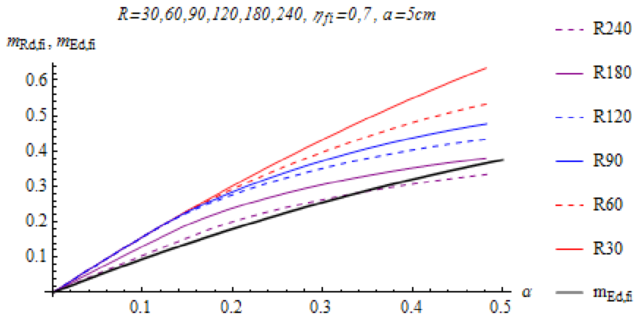

Figure 11 Fire resistance for a = 5 cm

The charts show that fully strained sections with lower reinforcement ratios belong to a higher fire resistance class. In order to facilitate the use of the charts, tables (Tab. 2) showing the maximum slab reinforcement ratios for a particular fire resistance class have been prepared. Given that the assumed $\eta_{fi} = 0.7$ is very restrictive, the load capacity has also been distinguished for different values of this factor.
For cross-sections not strained to 100%, the slab fire resistance can be calculated using the formula $M_{Rd,fi} = m_{Rd,fi} f_{cd} b d^2$ ($m_{Rd,fi}$ (as read from the graphs) and compared to the load in a fire situation equal to $M_{Ed,fi} = \eta_{fi} M_{Ed}$. This allows the fire resistance class of a slab to be raised by increasing the reinforcement ratio without modifying the element's dimensions.

CONCLUSIONS

The study shows that fire resistance calculated according to the Isotherm 500 method largely corresponds to the tabular method given in EC2 [6]. More detailed tables with more important parameters have been developed. They allow the reinforcement ratio – α, load reduction factor – n_{fi} and the distance of the reinforcement's centre of gravity from the heated face of the slab – *a*, to be considered.
In further work, the authors plan to examine the support zone, i.e. punching shear and load-bearing capacity when bending. Other elements of the reinforced concrete structure, such as beams and pillars, should also be examined using the same method.

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