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Research paper

Punching shear of RC slabs exposed to fire: assessment of load-bearing capacity based on European and American provisions. Part 1 – General code requirements

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Abstract: Slab-column connection is one of the most critical points in reinforced concrete structures. Punching shear capacity of this connection must be properly determined in both the persistent design situation and the accidental design situation of fire. European and American codes give simplified (tabulated) requirements for minimal slab thickness and minimal column cross-section width in accordance with the required fire resistance. In many cases, a more accurate prediction of fire resistance might be needed. It can be achieved when the ultimate limit states of a structure are checked in the case of fire conditions. This paper shows the comparison between European and American code requirements for punching shear capacity as the base for further calculations of flat RC slabs subjected to fire. The second part is going to present European and American requirements for determining the effects of loads relevant to the consideration of an accidental design situation of fire and comparing these effects with the punching shear capacity of the slab-column connection, which decreases with fire duration.

Keywords: fire resistance, isotherm 500°C method, punching shear, RC structure, slab-column connection

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1. Introduction

Punching shear capacity of a slab-column connection is usually one of the critical points when structural designing of modern reinforced concrete buildings is conducted. European [1] and American [2] codes give simplified (tabulated) requirements for minimal slab thickness and minimal column cross-section width in accordance with the required fire resistance. However, when special importance buildings or buildings with enormous endangering of human safety are designed, more accurate prediction of fire resistance might be needed in many cases. It can be achieved when ultimate limit states of a structure are checked in the case of fire conditions. Comprehensive requirements in this field are introduced in Eurocodes [1,3,4] with supplementary information available in various sources [5–11]. Information about punching shear of flat slabs exposed to fire temperature can be found in the literature [12–22].

When ultimate limit state of a particular critical point of a structure has to be checked a comparison between the design (factored) effect of actions (load) (E_d) and the design (factored) structure resistance (R_d) has to be made. Figure 1 [23] summarizes this comparison between aforementioned parameters in persistent design situation and in accidental design situation of fire. Due to different values of partial safety factors taken into calculation assumed initial design effect of actions (loads) in fire situation is lower than that in the persistent situation, and initial design value of structure resistance in fire situation is higher than that in the persistent situation. Therefore at the beginning of fire a considerable design reserve of load capacity remains in the structure. During fire the design value of structure resistance decreases due to high temperature influence and after critical fire duration the ultimate limit state of the structure occurs.

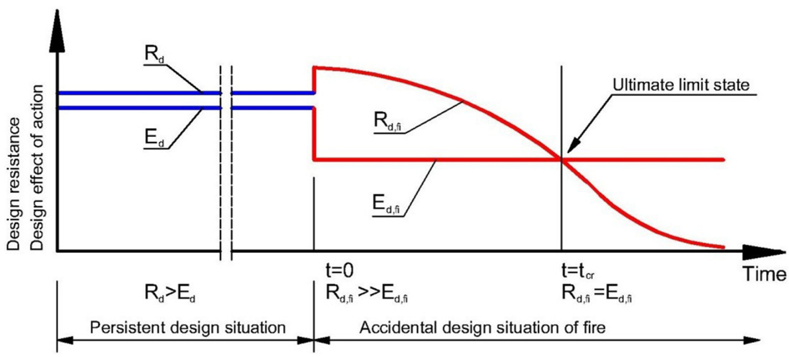


Fig. 1. Comparison between design effect of load (E_d) and the design structure resistance (R_d) for persistent design situation and accidental situation of fire [23]

This paper shows the comparison between the design (factored) effect of actions (loads) and the design (factored) punching shear capacity calculated for fire conditions with European [1,24] and American [2,25] code recommendations.

The first part provides information on calculating the punching shear capacity of a slab-column connection in accordance with European and American standards. Calculations of the load-bearing capacity for common use in practice instances of slab thickness and common use column cross-sections have been performed as a base for future consideration of slab-column connection in RC flat slabs exposed to fire. In the second part European and American requirements for determining the effects of loads relevant to the consideration of an accidental design situation of fire will be presented and comparison of these effects with the punching shear capacity of the slab-column connection, which decreases with fire duration will be performed.

2. Punching shear capacity according to European requirements

According to European requirements [24] design (factored) ultimate punching force (V_{Rd}) at an internal column-slab connection for a slab without punching reinforcement can be calculated as the smallest value given by the following formulas:

$$V_{Rd} = \frac{ud}{1.15} v_{Rdc} \tag{2.1}$$

$$V_{Rd} = \frac{u_0 d}{1.15} v_{Rd\,\max} \tag{2.2}$$

where: d – effective depth of a slab, u – perimeter of the control section situated in the distance $2d$ from the column face, u_0 – perimeter of the cross-section of the column

Punching shear resistance stress (v_{Rdc}) of a slab without punching reinforcement is given (in MPa) by the formula:

$$v_{Rdc} = \frac{0.18}{\gamma_C} k \left(100 \rho_L f_{ck}\right)^{\frac{1}{3}} \tag{2.3}$$

but obtained value cannot be smaller than:

$$v_{\min} = 0.035 k^{\frac{3}{2}} f_{ck}^{\frac{1}{2}} \tag{2.4}$$

where: γ_C – partial safety factor for concrete; $\gamma_C = 1.5$ for persistent design situation and $\gamma_C = 1.0$ for accidental design situation of fire, $k = 1 + \sqrt{\frac{200}{d}}$ but obtained value cannot be bigger than 2.0 (d in mm), f_{ck} (in MPa) – characteristic compressive cylinder strength of concrete at 28 days, ρ_L – calculated on the base of the main reinforcement ratios in both directions; $\rho_L = \sqrt{\rho_{Ly} \rho_{Lz}}$, but obtained value cannot be bigger than 0.02

The maximum shear stress ($v_{Rd,\max}$) around the column perimeter (u_0) is given (in MPa) by the formula:

$$v_{Rd,\max} = 0.3 \left(1 - \frac{f_{ck}}{250}\right) \frac{f_{ck}}{\gamma_c} \tag{2.5}$$

3. Punching shear capacity according to American requirements

According to American requirements [25] the design (factored) ultimate punching force (φV_c, in lb) at an internal column-slab connection for a slab without punching reinforcement, made of ordinary dense concrete can be calculated as the smallest value given by the following formulas:

$$\varphi V_c = \varphi\left(\frac{40d}{b_0} + 2\right)\sqrt{f_c'}b_0 d \tag{3.1}$$

$$\varphi V_c = \varphi 4\sqrt{f_c'}b_0 d \tag{3.2}$$

where: d (in inch) – effective depth of a slab, b_0 (in inch) – perimeter of a section need not approach closer than $d/2$ from the perimeter of a column cross-section, f_c' (in psi) – specified compressive strength of concrete, φ – strength reduction factor; $\varphi = 0.75$.

Considering that: $1\text{lb}_{\text{force}} = 4.4482 \cdot 10^{-6}$ MN; 1 in = 0.0254 m; 1 psi = 0.006895 MPa one can notice that:

$$V_c\ [\text{MN}] = 4.4482 \cdot 10^{-6} V_c\ [\text{lb}] = 4.4482 \cdot 10^{-6} \cdot 4\sqrt{f_c'\ [\text{psi}]}\, b_0\ [\text{in}]\ d\ [\text{in}] =$$

$$4.4482 \cdot 10^{-6} \cdot 4\sqrt{\frac{f_c'\ [\text{MPa}]}{0.006895}} \cdot \frac{b_0\ [\text{m}]\ d\ [\text{m}]}{0.0254^2}$$

$$V_c\ [\text{MN}] = 0.08303 \cdot 4\sqrt{f_c'\ [\text{MPa}]}\, b_0\ [\text{m}]\ d\ [\text{m}]$$

and Eq. (3.1) and Eq. (3.2) can be written as follows in metric units:

$$\varphi V_c = \varphi\left(\frac{3.32d}{b_0} + 0.166\right)\sqrt{f_c'}b_0 d \tag{3.3}$$

$$\varphi V_c = \varphi 0.332\sqrt{f_c'}b_0 d \tag{3.4}$$

where: V_c [MN]; f_c [MPa]; b_0, d [m].

4. Decrease of punching shear capacity due to fire

Structural fire analyses can be based on thermal actions given by nominal fires (time-temperature curves) or on physically based thermal actions (estimation of a real fire run) [1, 4]. The second option usually lets to consider a fire which is not as severe as nominal. This way is very useful when big and high spaces of representative buildings or big public, industrial or sport halls are designed [5, 7]. However, punching shear usually becomes a crucial state during designing ordinary building where the area and the height of compartments are relatively small. In these cases the nominal curves give safe and good enough approximation of temperature increase during fire.

Analyses conducted in this paper are based on the standard fire curve [4]. Similar time-temperature relationship is recommended [26] as ASTM curve. The both aforementioned curves are the most commonly used approximation of fire during experimental tests aimed on estimation of a fire resistance of structural members and many building products. The most efficient fire action on RC slabs usually occurs when fire acts on bottom face of a slab.

The Authors calculated a decrease of punching shear capacity for particular cases of columnslab connections described below on the base of following assumptions:

– slab thickness which were considered: $h = 15, 20, 25, 30$ cm; in these cases the effective cross-section depth was assumed $d = 12.0, 16.5, 21.5, 25.5$ cm respectively,
– square column cross-sections which were considered: $b \times h = 20 \times 20$; 30×30; 40×40; 50×50 cm,
– the main upper layer reinforcement ratio: $\rho_L = 0.01$ in both directions,
– concrete compressive strength $f_{ck} = f'_c = 30$ MPa,
– no punching shear reinforcement is provided,
– standard fire [4] influences the bottom slab face and four column surfaces.

Punching shear capacity of a column-slab connection constructed without punching reinforcement strongly depends on two features:

– the load bearing capacity of concrete compression zone which is situated around column cross-section in the bottom face of a slab,
– the load bearing capacity of upper layer reinforcement situated over the cross-section of a column.

In the case when fire is located under the slab, concrete in the compression zone of the slab and concrete of external column surface is destructed systematically due to high temperature action. At this point, the question of whether the compressive or tensile strength of concrete is responsible for the punching shear resistance must be addressed. These two values are interrelated by code formulas and by the nature of compressed concrete specimens failure. Compression along one axis is followed by tension in perpendicular direction. In general the compressive strength of concrete is the primary design parameter and is therefore superior to the tensile strength. Looking at the punching shear phenomenon, it can be observed that the perimeter cracks appearing in the tension zone (above the support) propagate from the tension surface of concrete to the neutral axis of cross-section and then have to go through the compression zone. The mechanism of this phenomenon has been described in the literature [27, 28]. It is based on the observation that the compression zone of reinforced concrete slab around the column (quasi-point support) is compressed in two directions – perimeter and radial, while deformation in the vertical direction (perpendicular to the slab) is free. As a result, tensile strains occur in the vertical direction and lead to cracks propagation into the bottom compression zone. This crack propagation causes the failure of the joint directly, but in the reality for the failure the concrete compressive strength is still responsible. Concrete failure is driven when concrete compression strain limit is reached.

Given that the punching failure of a flat slab without the punching shear reinforcement occurs due to the degradation of the concrete in the compressed zone in the bottom part of the slab and the upper part of the column, the analysis in this work was based on the *500°C Isotherm* method [1]. It is based on clearly defined assumptions:

– *500°C Isotherm* method can be used when a structure is exposed to standard fire or other one but with similar high temperature action,
– a part of the concrete section that is enveloped by the 500°C isotherm is fully active (the temperature lower than 500°C – concrete mechanical properties are as in virgin conditions),
– a part of the concrete section outside the 500°C isotherm is neglected (the temperature higher than 500°C – concrete is totally damaged),
– reinforcement has the mechanical properties corresponding to its actual temperature.

The *500°C Isotherm* method has been proposed by Y. Anderberg almost half a century ago and since it has been recommended [1] and successfully applied to the estimation of load bearing capacity of the RC cross-section exposed to the standard fire. However, in the new Eurocode [29] the *500°C Isotherm* method has been abandoned in favour of more accurate analysis techniques. Yet, perhaps the abandonment of the *500°C Isotherm* method might have been premature. This conclusion follows from the large discrepancies between relationships recommended for determining the reduction of concrete compressive strength due to fire temperature, according to European [1] and American [2] standards recommendations. Figure 2 shows these recommendations together with the basic assumption of *500°C Isotherm* method. One can easy note that *500°C Isotherm* method assumption, being extremely simplified, still well represents European [1] and American [2] standards recommendations.

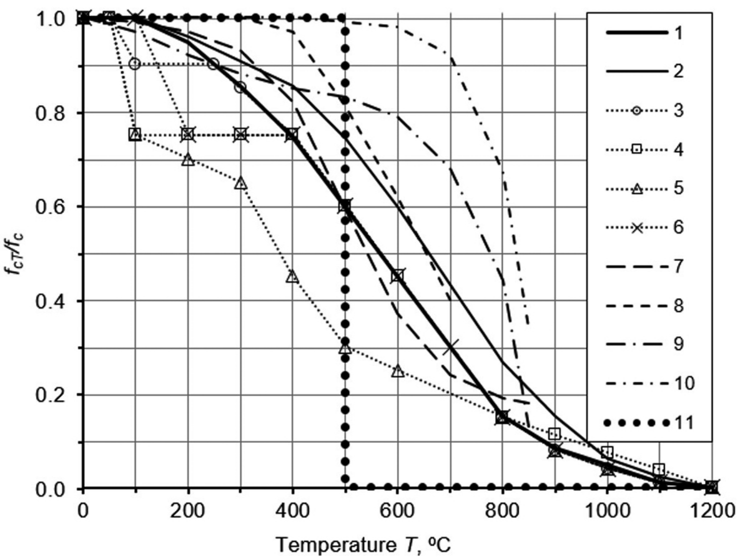
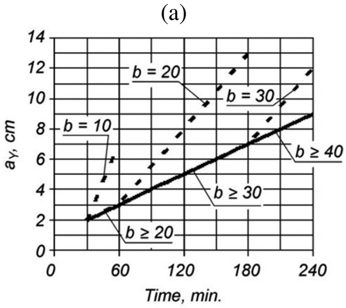


Fig. 2. Relative reduction in compressive strength of concrete (f_{cT}/f_c) at high temperature: 1 – ordinary concrete with siliceous aggregate [1,29] 2 – ordinary concrete with limestone aggregate [1,29], 3 – C55/67, C60/75 [1], 4 – C70/85, C80/95 [1], 5 – C90/105[1], 6 – concrete with $f_{ck} \geq 70$ MPa [29], 7 – concrete with siliceous aggregate, unloaded [2], 8 – concrete with siliceous aggregate, loaded to compressive stress value of $0.4 f_c$ [2], 9 – concrete with limestone aggregate, unloaded [2], 10 – concrete with limestone aggregate, loaded to compressive stress value of $0.4 f_c$ [2], 11 – 500°C isotherm method

For estimation of a position of the *500°C isotherm* a time-consuming calculation of unsteady heat flow in element cross-section is usually needed. However, for practical cases this computational effort can be avoided using simplified diagrams [30].

The *500°C Isotherm* method is recommended only for standard fire conditions and therefore time-temperature curve considered at calculations is always the same [30]. When designing new RC structures or when reassessing existing ones, the parameters influencing the heat flow in concrete (specific heat of concrete, thermal conductivity of concrete and the heat flux through the member surface) are usually not defined. Therefore the values of these parameters in most cases are assumed as the mean code values. The aforementioned limits cause that in practical calculations aimed on estimation the 500°C isotherm position the only variable parameters are usually dimensions of the element cross-section. In [30] calculations of the heat flow in many commonly used cross-sections of beams and slabs have been performed on the basis of the average code values [1,4] of parameters influencing the heat flow in concrete cross-section. On the base of obtained numerical results simplified diagrams for predicting the dimensions of the reduced cross-section and for evaluating the temperature of reinforcing bars have been introduced. Figure 3 [30] shows diagrams for estimation the 500°C isotherm position in a slab and in a column with square cross-section, in accordance with standard fire duration. On the base of Fig. 3 the Authors estimated a position of 500°C isotherm for all cases considered in this paper (Table 1).

(a)



For time $t < 60$ min.:
dashed line for $b = 10$ cm,
solid line for $b \geq 20$ cm,
For time $60 \leq t < 180$ min.:
dashed line for $b = 20$ cm,
solid line for $b \geq 30$ cm,
For time $t \geq 180$ min.:
dashed line for $b = 30$ cm,
solid line for $b \geq 40$ cm.
Interpolate in intermediate cases

(b)
For time:
$t = 30$ min. – $a = 1.0$ cm
$t = 60$ min. – $a = 2.0$ cm
$t = 90$ min. – $a = 3.0$ cm
$t = 120$ min. – $a = 3.5$ cm
$t = 180$ min. – $a = 5.0$ cm
$t = 240$ min. – $a = 6.0$ cm

Fig. 3. Position of the 500°C isotherm [30] a) recommendations for RC columns with a square cross-section (a_Y – measured from the cross-section edge, b – the cross-section width), b) recommendations for slabs (a – measured from the bottom face); t – standard fire duration

Table 1. Values of 500°C isotherm distance from cross-section edge estimated basing on Fig. 3

Standard fire duration, min.	30	60	90	120	150	180	210	240
Slab – a, cm	1.0	2.0	3.0	3.5	4.3	5.0	5.5	6.0
Column 50 × 50 cm – a_Y, cm	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0
Column 40 × 40 cm – a_Y, cm	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0
Column 30 × 30 cm – a_Y, cm	2.0	3.0	4.0	5.0	6.0	7.0	9.7	12.0
Column 20 × 20 cm – a_Y, cm	2.0	3.0	5.7	8.0	10.0	10.0	10.0	10.0

Calculations aimed on estimation of a decrease of punching shear capacity due to fire were performed with Eq. (3.1, 3.2) and Eq. (3.3, 3.4) where reduced slab depth (d_{fi}) and reduced column cross-section width (b_{fi}) were assumed as follows:

$$d_{fi} = d - a \tag{4.1}$$

$$b_{fi} = b - 2a_Y \tag{4.2}$$

European requirements [24] additionally cover the case when a column width is very small and the column-slab connection can be destroyed as a result of concrete compression around cross-section of the column. For instance when the column cross-section width follows to zero the perimeter u_0 follows to zero as well and the load capacity follows to zero too, see Eq. (3.2). American requirements [25] do not consider such a case. According to Eq. (3.3, 3.4) even if the column cross-section width is equal to zero the perimeter b_0 is still bigger than zero. Therefore the Authors in their calculations added one more formula to American requirements:

$$\varphi V_c = \varphi b^2 f_c' \tag{4.3}$$

Eq. (4.3) takes into account the situation when the cross-section of a column is not big enough to carry the force estimated with Eq. (3.3) or Eq. (3.4).

Figure 4 shows relative decrease of un-factored punching capacity which was estimated with Eq. (2.1, 2.2) (European). The vertical axis refers to the ratio of punching shear capacity

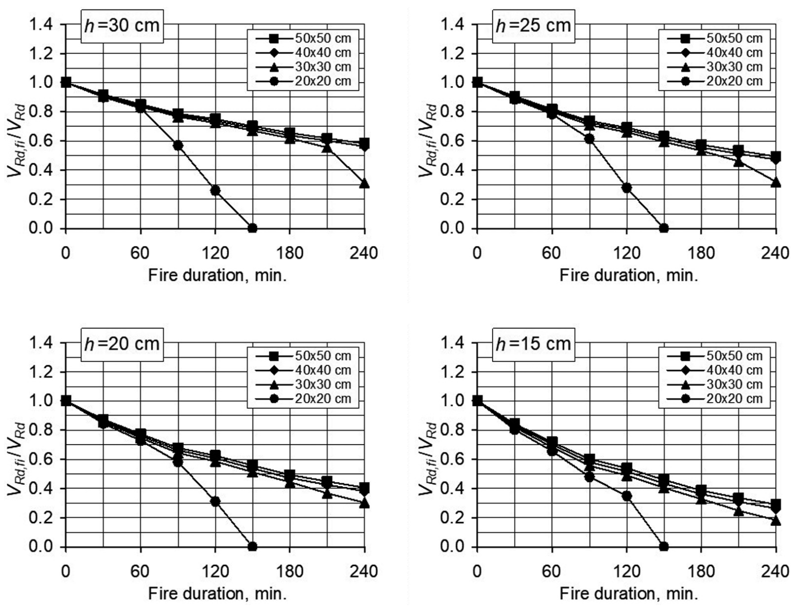


Fig. 4. Relative decrease of punching shear capacity against standard fire duration; results of calculation with European formulas [1, 24]

calculated for fire condition to the punching shear capacity calculated for room temperature. The ratios are shown against standard fire duration. The curves decrease systematically due to high temperature action considered as the reduction of column and slab cross-section dimensions. Figure 5 shows similar values but calculated with Eq. (3.3, 3.4) supplemented by the formula (4.3) (American).

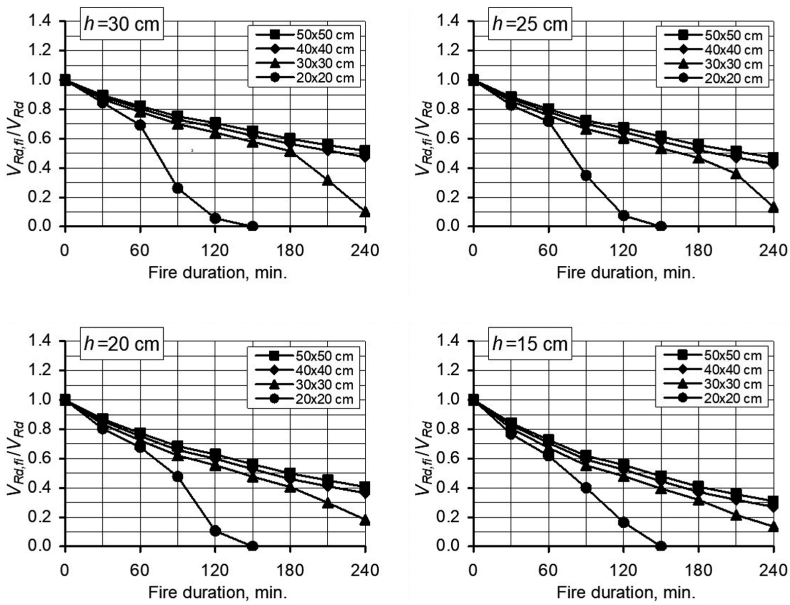


Fig. 5. Relative decrease of punching shear capacity against standard fire duration; results of calculation with American formulas [25]

Comparing diagrams shown in Fig. 4 and Fig. 5 it can be noticed that in all cases the curves “goes down” relatively slowly, almost linearly, unless the area of the column cross-section is not big enough to carrying the load. For the column cross-section 20×20 cm very rapid decrease of diagram occurs after relatively short fire duration. In all considered cases the decrease of punching shear capacity is quicker while the slab thickness is smaller. The relative punching shear capacity decrease calculated with European formulas (Fig. 4) seems to be less depended on the width of the column cross-section that the one obtained with American formulas (Fig. 5).

5. Summary and final remarks

The first part of the article provides information on calculating the punching shear capacity of a slab-column connection in accordance with European and American standards. Calculations for common use in practice instances of slabs (thickness $h = 15, 20, 25$ and 30 cm) supported by common used columns (cross-sections 20×20, 30×30, 40×40 and 50×50 cm) have been performed. Results of these calculations are going to be the base for further calculations of flat RC slabs subjected to fire.

Punching shear capacity of analysed nodes decreases almost linearly, unless the area of the column cross-section is not big enough to carrying the load. The relative punching shear capacity decrease calculated with European formulas (Fig. 4) seems to be less depended on the width of the column cross-section that the one obtained with American formulas (Fig. 5).

The second part of the article will present European and American requirements for determining the effects of loads relevant to the consideration of an accidental design situation of fire and comparing these effects with the punching shear capacity of the slab-column connection, which decreases with fire duration.

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Nośność na przebicie płyt żelbetowych narażonych na działanie pożaru, według wymagań europejskich i amerykańskich. Część 1 – Wymagania normowe

Słowa kluczowe: odporność ogniowa, metoda izotermy 500, przebicie, konstrukcje żelbetowe, połączenie płyta-słup

Streszczenie:

Zapewnienie nośności na przebicie połączenia płyta-słup jest jednym z kluczowych etapów projektowania konstrukcji budynków żelbetowych, również z uwagi na warunki pożarowe. Zarówno w europejskich, jak i amerykańskich normach do projektowania konstrukcji są podane uproszczone (tabelaryczne) wymagania dotyczące minimalnej grubości płyt stropowych oraz minimalnej szerokości przekrojów słupów w zależności od wymaganej klasy odporności ogniowej. Jednak w przypadku projektowania budynków o szczególnym znaczeniu lub dużym zagrożeniu bezpieczeństwa ludzi, w wielu przypadkach konieczne może być dokładniejsze określenie odporności ogniowej konstrukcji. Można to osiągnąć, weryfikując stany graniczne nośności konstrukcji w warunkach pożarowych. W pierwszej części artykułu przedstawiono porównanie wymagań norm europejskich i amerykańskich w zakresie nośności stropu na przebicie oraz wyniki obliczeń nośności na przebicie dla typowych, stosowanych w praktyce inżynierskiej grubości stropów i wymiarów słupów, narażonych na działanie wysokiej temperatury. W drugiej części artykułu przedstawione zostaną wymagania norm europejskich i amerykańskich w zakresie określania oddziaływań w wyjątkowej sytuacji pożaru oraz porównanie efektów tych oddziaływań z nośnością stropu na przebicie, która zmniejsza się w trakcie trwania pożaru.

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