CALCULATIONS OF REINFORCED CONCRETE STRUCTURES FIRE RESISTANCE

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Abstract

The paper presents steps necessary to perform a calculation of the fire resistance of a RC structure. A simple tool for evaluating this parameter are tabulated data which provide minimum requirements in terms of cross-section dimensions and distance of the reinforcement to the surface of the cross-section exposed to fire, depending on the type of the element and required fire resistance. More precise prediction of fire safety of a structure (or its elements) can be obtained when the fire is considered as an accidental design situation for which ultimate limit states of the structure are analysed. The paper presents the essence of considering the fire as an accidental design situation. The design effect of actions adopted in accidental design situation of fire is usually lower than the one adopted in a persistent design situation. In transition from the persistent to the accidental fire situation, an increase of the design load bearing capacity occurs. Afterwards, as the fire progresses, the load bearing capacity decreases in the result of worsening strength properties of concrete and steel caused by high temperature. After the critical duration of fire the ultimate limit state occurs. In order to perform an analysis of such state, it is necessary to determine direct actions occurring in fire situation, adopt a design fire model, calculate temperature fields at selected parts of the structure, take into account the reduction in strength of concrete and reinforcing steel and calculate the load bearing capacity of the structure.

Streszczenie

W pracy scharakteryzowano czynności, które należy przeprowadzić w celu obliczeniowego sprawdzenia odporności ogniowej konstrukcji żelbetowej. Prostym narzędziem służącym do oceny tego parametru są tablice, w których, w zależności od rodzaju elementu i wymaganej odporności ogniowej, są podane minimalne wymiary przekroju elementu i minimalna odległość środka ciężkości przekroju zbrojenia od krawędzi przekroju elementu. Dokładniejszą prognozę odporności ogniowej konstrukcji (lub jej elementów) można uzyskać rozpatrując pożar jako wyjątkową sytuację obliczeniową, w której są sprawdzane stany graniczne nośności. W pracy scharakteryzowano istotę rozpatrywania pożaru jako wyjątkowej sytuacji obliczeniowej. Obliczeniowy efekt oddziaływań występujących w sytuacji pożaru jest najczęściej mniejszy od efektu rozpatrywanego w trwałej sytuacji obliczeniowej. Przy „przejściu" z sytuacji trwałej do wyjątkowej pożaru obliczeniowa nośność konstrukcji zwiększa się. Następnie, w miarę upływu czasu trwania pożaru, nośność obliczeniowa konstrukcji maleje w wyniku pogarszania się cech wytrzymałościowych betonu i stali zbrojeniowej pod wpływem wysokiej temperatury. Po pewnym krytycznym czasie trwania pożaru w konstrukcji występuje obliczeniowy stan graniczny nośności. W celu przeprowadzenia analizy obliczeniowej tego stanu należy: określić oddziaływania bezpośrednie występujące w sytuacji pożaru, przyjąć obliczeniowy model przebiegu pożaru, obliczyć temperatury w wybranych miejscach konstrukcji, uwzględnić zmniejszenie wytrzymałości betonu i stali zbrojeniowej oraz obliczyć nośność konstrukcji.

Keywords: **Reinforced concrete structure; Fire resistance; Model of fire.**

1. INTRODUCTION

Changes of legal regulations concerning safety of buildings [1, 2] made it impossible at present to design, construct or use building structures without the consideration of fire safety problem. From the practical point of view of a structural engineer, the fire safety considerations focus on providing solutions which ensure required fire resistance for the structure (or structural elements). In doing so, the following three aspects of the parameter should be considered:

- resistance (R), which refers to protecting the structure against load bearing capacity loss or stability failure,
- integrity (E), which refers to protecting the structure against hot gases and flame penetration of the unheated side of a building partition,
- insulation (I), which refers to protecting the structure against temperature increase at the unheated side of a building partition.

The measure of fire resistance (R, E or I) is the period of time in which the structure can fulfil the requirements when it is subjected to specified normalized fire conditions. Please note that fire resistance does not refer to the behaviour of the structure (or structural elements) in real fire which can take an unforeseeable course.

In most cases, the required fire resistance of reinforced concrete structures can be achieved by using the tabulated data [3]. These guidelines provide minimum requirements in terms of cross-section dimensions and distance of the reinforcement to the surface of the cross-section exposed to fire. If the element cross-section is big enough, the inner part of the cross-section is not heated significantly in the result of the fire and in this way the inner part of the cross-section can carry the load for the required period of time. If the reinforcement is covered by concrete well enough, the reinforcement temperature increases slowly and in this way the reinforcement can carry the force for the required period of time. Based on the tabulated data one should take into consideration that dimensions of the cross-section are really important only in cases where the failure of the structure might occur as a result of a damage of the concrete compressed zone. The concrete cover of the reinforcement is really important only in cases when the failure of the structure might occur in the result of the yield of the bars.

Tabulated data are convenient in practical prediction of fire resistance of the structure, but sometimes a better accuracy might be required. In such case, the fire has to be considered as the accidental design situation and an analysis of ultimate limit states of the structure subjected to fire is required [3, 4, 5].

2. FIRE AS THE ACCIDENTAL DESIGN SITUATION

When a persistent design situation is considered, the decisive factor for the structure to meet the predefined requirements is the ultimate limit state inequality:

$$E_d \le R_d, \tag{1}$$

where the design effect of the actions (loads) E_d and the calculated resistance of the structure R_d are compared [4]: In some cases, the serviceability limit state inequality is more significant:

$$E_d \le C_d \tag{2}$$

In case of reinforced concrete structure analysis, C_d usually refers to the deflection limit or crack width limit and rarely to crack appearance.

The design value of load for the analysis of ultimate limit states in persistent design situation can be estimated according to the formula [4]:

$$E_d = \sum_{j\ge 1} \gamma_{G,j} G_{k,j} + \gamma_{Q,1} Q_{k,1} + \sum_{i>1} \gamma_{Q,i} \psi_{0,i} Q_{k,i}\,, \tag{3}$$

or alternatively as less favourable of formulas [4]:

$$E_d = \sum_{j\ge 1} \gamma_{G,j} G_{k,j} + \gamma_{Q,1} \psi_{0,1} Q_{k,1} + \sum_{i>1} \gamma_{Q,i} \psi_{0,i} Q_{k,i} \tag{3a}$$

$$E_d = \sum_{j\ge 1} \xi_j \gamma_{G,j} G_{k,j} + \gamma_{Q,1} Q_{k,1} + \sum_{i>1} \gamma_{Q,i} \psi_{0,i} Q_{k,i}\,. \tag{3b}$$

In formulas (3):

- G and Q refer to characteristic values of permanent and variable actions (imposed loads) respectively,
- γ_G and γ_Q are partial safety factors for permanent and variable actions respectively,
- ψ_0 is the factor for combination value of variable action,
- ξ is the reduction factor.

The design value of the structure (structural element) resistance for the analysis of ultimate limit states in persistent design situation can be shown as function [4]:

$$R_d = R\{\frac{f_{ck}}{\gamma_C}, \frac{f_{yk}}{\gamma_S}, a_d\} = R\{f_{cd}, f_{yd}, a_d\}\,, \tag{4}$$

where:

- f_{ck}, f_{yk}, f_{cd}, and f_{yd} refer to characteristic and design values of concrete compressive strength and steel yield strength respectively,
- γ_C and γ_S are partial safety factors for concrete and steel respectively,
- a_d is the design value of geometrical data.

When the accidental design situation of fire is considered, only the analysis of ultimate limit states is important. In this case, the design value of load $E_{d,fi}$ can be estimated according to the formula [4]:

$$E_{d,fi} = \sum_{j\geq 1} G_{k,j} + A_d + (\psi_{1,1} \text{ lub } \psi_{2,1}) \cdot Q_{k,1} + \sum_{i>1} \psi_{2,i} Q_{k,i}\,, \tag{5}$$

where:

- ψ_1 is the factor for frequent value of variable action,
- ψ_2 is the factor for quasi-permanent value of variable action,
- A_d is the design value of accidental action.

According to [3, 4] the choice between ψ_1 or ψ_2 factor in formula (5) should be related to the relevant design situation. According to [5], the ψ_2 factor should be adopted in formula (5).

In practice, when the accidental design situation of fire is considered, accidental actions are very rarely taken into account. If one neglects accidental actions and assumes that partial factors adopt values: $\gamma_G = 1.35$, $\gamma_Q = 1.5$, and ψ_2 factor adopts values lower than one, it is possible to conclude that the design value of load for accidental design situation of fire (5) is lower than the design value of load for persistent design situation (3):

$$E_{d,fi} < E_d. \tag{6}$$

It is also worth noticing that the design value of action (load) $E_{d,fi}$ is constant through the whole duration of fire.

The design structure resistance in case of fire $R_{d,fi}$ depends on concrete and steel mechanical properties, which are decreasing due to high temperature influence. Therefore, formula (4) has to account for the concrete and steel strength decrease:

$$R_{d,fi} = R_{fi}\{\frac{k_{c,\theta} f_{ck}}{\gamma_C}, \frac{k_{s,\theta} f_{yk}}{\gamma_S}, a_d\}, \tag{7}$$

where $k_{c,\theta}$ and $k_{s,\theta}$ factors describe the reduction of concrete and steel strength respectively.

When a persistent design situation is considered, it is recommended [6] to adopt the material factors: $\gamma_C = 1.50$, $\gamma_S = 1.15$ however, these values can differ in particular countries according to the National Annexes [6] – For instance in Poland lower value $\gamma_C = 1.40$ is recommended in the National Annex [6]. For accidental design situation of fire according to [6], it is recommended to take: $\gamma_C = 1.20$, $\gamma_S = 1.00$ and according to [3], it is recommended to take: $\gamma_C = 1.00$, $\gamma_S = 1.00$. This means that if one switches from a persistent design situation to an accidental situation of fire, the design value of structure resistance increases by at least 15%.

At the beginning of fire when its duration time is $t = 0$, the material strength decrease does not exist which means that $k_{c,\theta}$ and $k_{s,\theta}$ factors are equal to one. Comparing formulas (4) and (7) one can conclude that:

$$R_d < R_{d,fi}(t=0). \tag{8}$$

If one transits from the persistent design situation to the accidental situation of fire, the design effect of actions decreases (formula (3) is replaced by (5)) and the design value of structure resistance increases (formula (4) is replaced by (7)). Combining formulas (1, 6 and 8), when the duration of fire is $t = 0$ one results in the following inequality:

$$E_{d,fi} < E_d < R_d < R_{d,fi}(t=0), \tag{9a}$$

which finally adopts the following shape:

$$E_{d,fi} << R_{d,fi}(t=0). \tag{9b}$$

At the beginning of fire in case of appropriately designed and correctly constructed structure, there remains a considerable design reserve of load bearing capacity.

As a result of fire, the design value of structure resistance (7) is decreasing and after the critical time of fire ($t = t_{cr}$) the structure resistance becomes equal to the design effect of load. It means that the ultimate limit state of the structure has occurred and inequality (9b) has changed into the equation:

$$E_{d,fi} = R_{d,fi}(t = t_{cr}). \tag{10}$$

Fig. 1 summarises the comparison between the design effect of load and the design structure resistance in the persistent design situation and in the accidental design situation of fire. Additional information about fire as the accidental design situation of the structure can be also found in [7, 8].

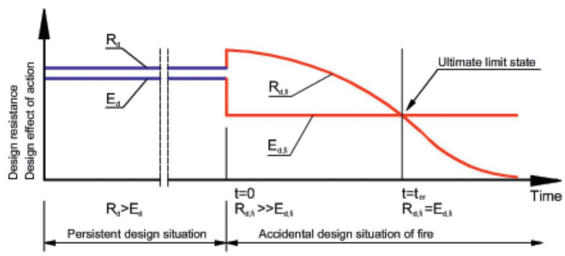

Figure 1.
A comparison between designed effect of load and the design structure resistance in persistent design situation and accidental situation of fire

If fire conditions which are considered in the calculations are appropriate for structure fire resistance estimation (normalised test conditions), the critical time of fire t_{cr} obtained as a result of calculation (from equation (10)) may be treated as fire resistance of the structure. This technique is usually used in practice when an existing structure is analysed.

During ordinary practical structural designing of new structures, the required fire resistance of the structure R_{req} is usually assumed pursuant to legal requirements and the final step of analysis consists of checking the following inequality:

$$E_{d,fi} \leq R_{d,fi}(t = R_{req}). \tag{11}$$

3. ANALYSIS OF THE STRUCTURE IN ACCIDENTAL SITUATION OF FIRE

According to Eurocode [3], one can consider fire as the accidental design situation at three alternative levels:
- member analysis,
- analysis of a part of the structure,
- global structural analysis.

In case when the member analysis is performed, the element should be considered as separate from the structure. High temperature is the sole effect of fire taken into consideration. Fire resistance can be estimated on the basis of tabulated data as well results of simplified or advanced calculation methods.

While analysing a part of the structure, one should consider the group of elements isolated from the entire structure with appropriate boundary conditions. Apart from high temperature influence, some indirect actions should be also analysed. These actions should be considered as constant in time. Fire resistance estimation can be based on the results of simplified or advanced calculation methods.

While performing a global structural analysis, only advanced calculation methods can be used. One should consider all aspects of fire influence on the structure.

In the author's opinion, concepts: "structure", "part of the structure" and "structural member" overlap in practical structural designing. Each practical structural analysis usually begins with the global consideration of the entire structure and finishes with the calculation of critical cross-sections of selected members.

Regardless of whether the structure is analysed in whole, in part or as a member, if one needs to check its ultimate limit state in fire condition, one should perform the following steps:
- estimation of actions (loads) appropriate for the accidental situation of fire followed by a calculation of internal forces induced by these actions,
- choice of the hypothetical fire scenario and estimation of the appropriate model of fire,
- calculation of the temperature field in selected cross-sections of the structure after the assumed duration of the selected model of fire,
- consideration of the decrease of mechanical properties of concrete and steel due to the influence of high temperature,
- calculation of the load bearing capacity of analysed cross-sections of the structure and comparing them with the design effects of actions (11).

4. CHARACTERISTICS OF FIRE ANALYSIS STEPS

4.1. Actions

While analysing an accidental design situation of fire, the following actions are typically considered:
- direct static actions (imposed loads),
- accidental loads,
- indirect actions.

Accidental loads may occur in fire when the structure is hit or burdened with destroyed structural parts, bears the load of water used for fire-fighting (when it amasses in great quantities on floors, roofs or terraces), suffers effects of unintentional actions in the course of fire-fighting (e.g is hit by the equipment or must bear its load) or when the secondary structural mechanism is generated.

However, taking into account all possible accidental loads would lead to a considerable increase of the calculation value of the effect of actions, and consequently, would generate oversize member cross-sections which are unacceptable in practice. The rational design of individual venues requires therefore a conscious rejection of certain accidental loads in favour of others. The standards do not give uniform guidelines on the subject, which delegates the decision making to the structural engineer, who has to consider the cost of the design implementation and the consequences of any damage to the structure.

The most important indirect actions are usually the internal forces induced due to the restraint of element expansion that appears as a result of the increase of material temperature. These forces can be particularly dangerous in case of frame precast structures when the damage of element joints might appear even at the beginning of fire.

Estimation of imposed static loads in practice is usually limited to the consideration of a reduction factor η_{fi}. This factor is the ratio of the sum of actions considered in accidental situation of fire (5) to the sum of actions considered in persistent design situation (3) (or the max. of (3a) or (3b)). In the simplest case, when only one variable action is considered and the reduction of actions according to (3a) or (3b) is neglected, the reduction factor can be calculated as follows [3]:

$$\eta_{fi} = \frac{\sum_{j\geq 1} G_{k,j} + \psi_{2,1} Q_{k,1}}{\sum_{j\geq 1} \gamma_{G,j} G_{k,j} + \gamma_{Q,1} Q_{k,1}}. \qquad (12)$$

Table 1 shows some examples of ψ_2 factor values recommended for buildings. Figure 2 shows the graphs of the reduction factor η_{fi} against the ratio of characteristic variable load $Q_{k,1}$) to the entire load $\sum G_{k,j} + Q_{k,1}$. The graphs are prepared for various categories of area for partial factors: $\gamma_{G,j} = 1.35$, a $\gamma_{Q,1} = 1.50$. Information about loads considered in fire situation might be found also in [7, 9].

Table 1.
Recommended values of ψ_2 factor for buildings [4]

Category of area (action)	ψ_2
A: domestic, residential areas	0.3
B: offices areas	0.3
C: congregation areas	0.6
D: shopping areas	0.6
E: storage areas	0.8
F: traffic area vehicle weight G ≤ 30 kN	0.6

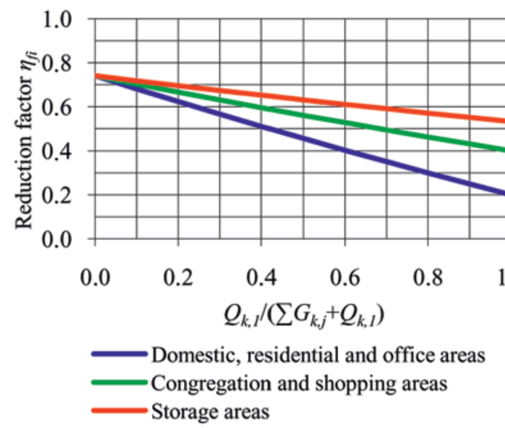

Figure 2.
The reduction factor η_{fi} against the ratio of characteristic variable load $Q_{k,1}$ to the entire load $\sum G_{k,j} + Q_{k,1}$ for various categories of area

4.2. Model of fire

Fires are generally divided into those which occur in relatively small premises (area up to 500 m^2, height up to 4 m) and those which occur in large venues.

Fires occurring in small premises initially cover single items such as pieces of furniture or waste baskets etc. In such case, human safety is at risk, but the structure itself is not exposed to significant damage. When the fire develops, the flash over may occur causing all combustible materials in the premises to burn. The flash over typically occurs in temperatures ranging between 400 and 600°C. The time between the ignition and the occurrence of such temperature can differ considerably, however in the least favourable instances it may take only a few minutes (3-5). After the flash over, the fire reaches the developed stage with temperature exceeding 1000°C. A fully developed fire endangers the safety of the structure.

The most common fire model is reflected by the standard curve [5] (Fig. 4):

$$\theta_g = 20 + 345 \log_{10}(8t+1). \qquad (13)$$

Figure 3.
The appearance of premises after the fire – damage is clearly visible at the ceiling, while no structural damage is present in the lower part. (photo by the author)

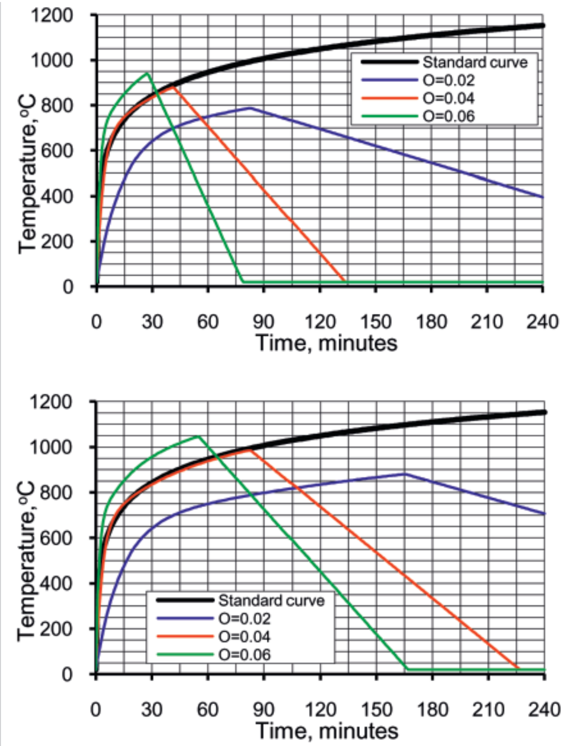

Figure 4.
Fire standard curve and examples of parametric temperature-time curves [11] for various values of the opening factor (O, $m^{1/2}$): a) fire load density q_{fd} = 500 MJ/m^2, b) fire load density q_{fd} =1000 MJ/m^2

In formula (13), θ_g stands for the temperature of gases in °C occurring in the fire zone, and t – for the fire time in minutes. The standard curve is a conventional representation of the course of a fully developed fire occurring in standard premises (e.g residential, office or public). It is most often applied in the experimental tests on fire resistance of structural elements. Pursuant to [5], structural analyses which must meet legal requirements concerning fire resistance, may assume that the calculated fire is described with the standard curve (13). It is assumed that a fire described with the standard curve is a single-zoned one, which means that uniform temperature is reached on the whole space affected by fire. This simplifying assumption is very far fetched, as in fact, the temperature under the ceiling of the premises is most frequently higher from one over the floor, which can be observed at Fig. 3.

The standard curve is a safe estimate for the course of fire occurring in small premises. The parametric temperature-time curves [5] provide a more exact forecast, as they allow to take into account the geometry of the premises for which the fire analysis is performed, the number and size of openings, and thermal properties of the partitions concerned. Figure 4 presents the standard curve and examples of parametric temperature-time curves, defined according to [10] for an example of premises with average and relatively high fire load density and for various opening factors. The latter term determines the ventilation (oxygen inlet) of the premises in fire [5].

Scenarios of fire safety analyses for large, spatially complex venues (such as atria, large and high spaces in shopping malls, factories, storage facilities, churches etc.) most frequently assume that the flash over does not occur. The smoke in large spaces cools down after mixing with cold air. In such instances, one has to use advanced fire models or local fire model [5], which assumes that the fire load occurs only on a limited area of a large venue.

4.3. Temperature field calculation

To calculate the temperature field in the cross-section of a RC element, one must know the specific heat and thermal conductivity of concrete, as well as the heat

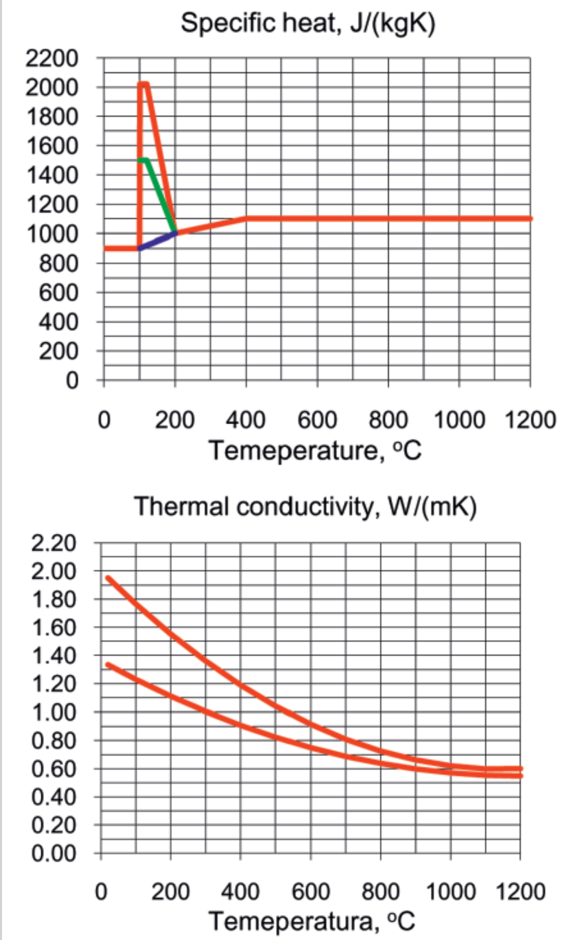

Figure 5.
Thermal properties of concrete with siliceous aggregate [3]

flux penetrating the element through its surface. When calculating the temperature field in RC elements, the reinforcement can be ignored [3].

Figure 5 shows the specific heat and thermal conductivity of concrete against its temperature [3]. The specific heat in the temperature range between 100 and 200°C depends on the moisture contents of the concrete. The highest part of the curve describes concrete with moisture contents of 3%, the medium part of – 1,5%, and the lowest describes dry concrete (0%). Figure 5 shows respectively the upper and lower limits of thermal conductivity of concrete.

When determining the heat flux, convection and radiation [5] should be considered.

The convective heat flux can be determined as [5]:

$$h_{net,c} = \alpha_c \cdot (\theta_g - \theta_m), \tag{14}$$

where θ_g is the gas temperature in the vicinity of the fire exposed member, θ_m is the surface temperature of the member and α_c is the coefficient of heat transfer by convection. When standard fire is considered, α_c = 25 W/(m^{2}K) is recommended [5].

The formula for determining the radiative heat flux [5] includes parameters whose recommended values are given in [5]. After putting these values in, one obtains the following formula:

$$h_{net,r} = 4,61 \cdot 10^{-8} \cdot [(\theta_g + 273)^4 - (\theta_m + 273)^4]. \tag{15}$$

In the appendix of [3], one can find some examples of the temperature profiles calculated for selected cross-sections exposed to standard fire.

4.4. Mechanical properties of concrete and steel heated up to high temperature

Eurocode [3] specifies mathematical models of stress-strain relationships for reinforcing steel subjected to tension or concrete compressed in high temperature. These models are indispensable to perform advanced analyses of structures in fire. However, the

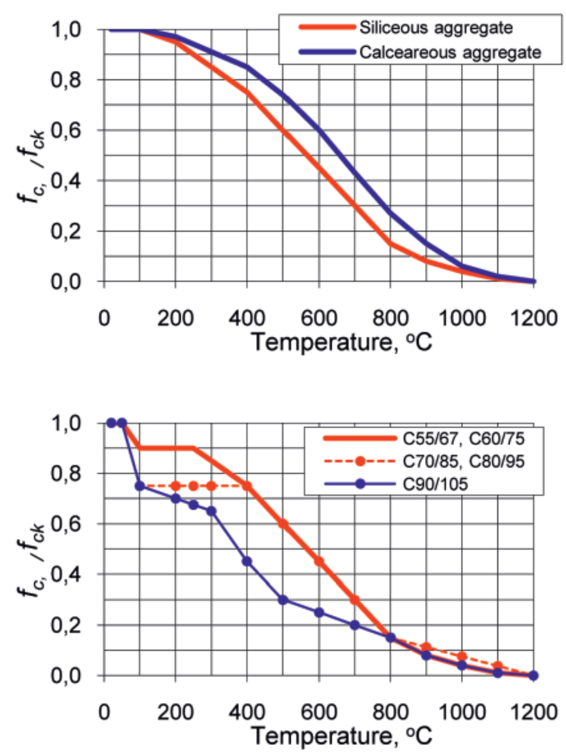

Figure 6.
Relative concrete compressive strength decrease in high temperature [3]: a) ordinary strength concrete, b) high strength concrete

knowledge of the relative reduction of material strength properties due to high temperature is most helpful for simplified calculations of fire resistance of RC structures.

Figure 6 presents relative decrease of the compressive strength of various types of concrete against temperature, prepared on the basis of Eurocode [3]. Behaviour of concrete heated to high temperature and phenomena occurring in concrete in fire are described, among others, in [11-13].

Figure 7 presents relative reinforcing steel yield strength decrease against temperature, prepared on the basis of Eurocode [3]. Information on the behaviour of reinforcing steel in high temperature and the Eurocode [3] model is also available in [14-16].

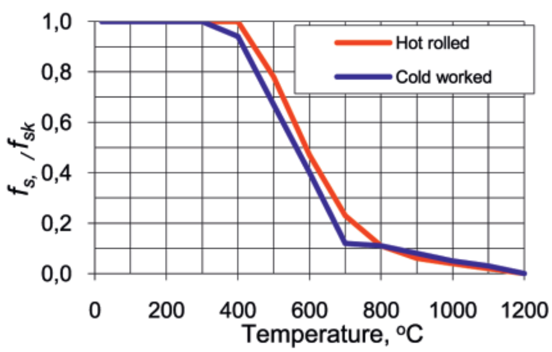

Figure 7.
Relative reinforcing steel strength (yield strength) decreases in high temperature [3]

4.5. Calculation of cross-sections load bearing capacity

Eurocode [3] recommends for the calculation of the load bearing capacity of reinforced member cross-section two simplified methods: the "500°C isotherm" method and the "zone" method. Both of them are appropriate when standard fire is considered.

The "500°C isotherm" method neglects the external part of the cross section where the temperature is higher than 500°C. The decrease of concrete strength due to high temperature is neglected in the internal part of the cross-section where the temperature is lower than 500°C. In this way, the "500°C isotherm" method considers the reduced cross section of concrete with the same strength as the one in room temperature. The decrease of the reinforcement yield strength can be taken into calculation according to Fig. 7 irrespective of whether the bars are situated inside or outside of the area limited with the isotherm 500°C.

In the "zone" method [3], the rules recommended for the determination of the reduced concrete cross section dimensions are a little bit more sophisticated. First it is necessary to divide the cross-section into several zones. Then the temperature in the middle of each zone has to be calculated. The range of the external damaged zone of the cross-section can be calculated according to the coefficients of concrete compressive strength decrease (Fig. 6). In the reduced cross-section, the reduced concrete compressive strength is adopted. The decrease of the concrete strength is adopted according to the temperature calculated for the point chosen arbitrarily within the middle part of the reduced cross section. The requirements on how to account for the influence of the reinforcement bars are the same as those recommended in the "500°C isotherm" method.

Information on calculating the load bearing capacity of the reinforced member cross-section in fire can be also found in [8, 13, 17-20].

5. CONCLUSIONS

In practice, from the viewpoint of a structural engineer, the consideration of fire safety requirements is most frequently reduced to applying solutions which ensure suitable fire resistance of all structural elements. Fire resistance does not refer to the behaviour of the structure (or structural elements) in real fire which can take an unforeseeable course, but concerns precisely determined conditions of fire tests, usually with the fire standard curve adopted.

In simple cases, the required fire resistance of reinforced concrete structures can be achieved by using the tabulated data [3]. The tables provide minimum requirements in terms of cross-section dimensions and distance of the reinforcement to the surface of the cross-section exposed to fire.

A more precise prediction of fire safety of a structure (or its elements) can be obtained when the fire is considered as an accidental design situation. In such case, ultimate limit states of the structure are analysed.

The design effect of actions adopted in accidental design situation of fire is usually lower than the one adopted in a persistent design situation. In transition from the persistent to the accidental fire situation, an increase of the design load bearing capacity occurs.

Afterwards, as the fire progresses, the load bearing capacity decreases in the result of worsening strength properties of concrete and steel caused by high temperature. After the critical duration of fire, the design resistance of the structure is equal to the design effect of the action and the ultimate limit state occurs (Fig. 1).

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