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DESIGN OF REINFORCED CONCRETE ELEMENTS UNDER FIRE

BY

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Abstract. Fire safety regulations can have a major impact on many aspects of the overall design of a building, including layout, aesthetics, function, and cost. Rapid developments in modern building technology in the last decades often have resulted in unconventional structures and design solutions. Because the world is developed continuously, the physical size of buildings increases continually; there is a tendency to build large underground car parks, warehouses, and shopping complexes. As a result, we have a worldwide movement to replace prescriptive building codes with ones based on performance. The paper presents the basic principles for the designing process of reinforced concrete elements under fire.

Key Words: Concrete; Reinforcement; Fire; Safety; Temperature.

1. Introduction

Fire is a physical and chemical phenomenon that is strongly interactive by nature. The interactions between the flame, its fuel, and the surroundings can be strongly nonlinear, and quantitative estimation of the processes involved is often complex. The processes of interest in an enclosure fire mainly involve mass fluxes and heat fluxes to and from the fuel and the surroundings.

Enclosure fires are often discussed in terms of the temperature development in the compartment and divided into different stages accordingly. Fig. 1 shows an idealized variation of temperature with time, along with the growth stages, for the case where there is no attempt to control the fire. The main stages of fire are: ignition, growth, flashover, fully developed fire, decay.

Ignition can be considered as a process characterized by an increase in temperature greatly above the ambient. It can occur either by piloted ignition or by spontaneous ignition. The accompanying combustion process can be either flaming combustion or smoldering combustion.

Following ignition, the fire may grow at a slow or a fast rate, depending on the type of combustion, the type of fuel, interaction with the surroundings, and access to oxygen. The fire can be described in terms of the rate of energy released and the production of combustion gases.

A smoldering fire can produce hazardous amounts of toxic gases while the energy release rate may be relatively low. The growth period of such a fire may be very long, and it may die out before subsequent stages are reached.

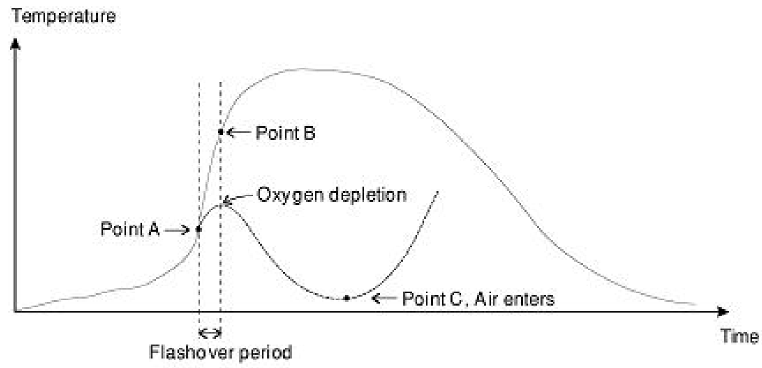

Fig. 1. – Main stages in fire process.

The growth stage can also occur very rapidly, especially with flaming combustion, where the fuel is flammable enough to allow rapid flame spread over its surface, where heat flux from the first burning fuel package is sufficient to ignite adjacent fuel packages, and where sufficient oxygen and fuel are available for rapid fire growth. Fires with sufficient oxygen available for combustion are said to be *fuel-controlled*.

Flashover is the transition from the growth period to the fully developed stage in fire development. The formal definition from the International Standards Organization is: "the rapid transition to a state of total surface involvement in a fire of combustible material within an enclosure". In fire safety engineering, the word is used as the demarcation point between two stages of a compartment fire, *i.e.*, pre-flashover and post-flashover.

Flashover is not a precise term: several variations in definition can be found in the literature. The criteria given usually demand that the temperature in the compartment has reached $500°\ldots600°$C or that the radiation to the floor of the compartment is $15\ldots20$ kW/m^2 or that flames appear from the enclosure openings. These occurrences may all be due to different mechanisms resulting from the fuel properties, fuel orientation, fuel position, enclosure geometry, and conditions in the upper layer. Flashover cannot be said to be a mechanism, but rather a phenomenon associated with a thermal instability.

On fully developed fire stage the energy released in the enclosure is at its greatest and is very often limited by the availability of oxygen. This is called *ventilation-controlled burning* (as opposed to *fuel-controlled burning*), since the oxygen needed for the combustion is assumed to enter through the openings. In ventilation-controlled fires, unburned gases can collect at the ceiling level, and as these gases leave through the openings they burn, causing flames to stick out through the openings. The average gas temperature in the enclosure during this stage is often very high, in the range of 700°... 1,200°C.

Last stage is decay when the energy release rate diminishes and thus the average gas temperature in the compartment declines. The fire may go from ventilation-controlled to fuel-controlled in this period.

2. Material Properties

The material properties at 20°C should be assessed according to Romanian standard STAS 10107/0-90 or Eurocode 2 Part 1: *General Rules and Rules for Buildings*.

The fire action will reduce the strength of materials. If the temperature of the element increases, the strength of material decreases. The standard fire conditions are defined between 20°C and 1,200°C, the properties are also defined between the same limits.

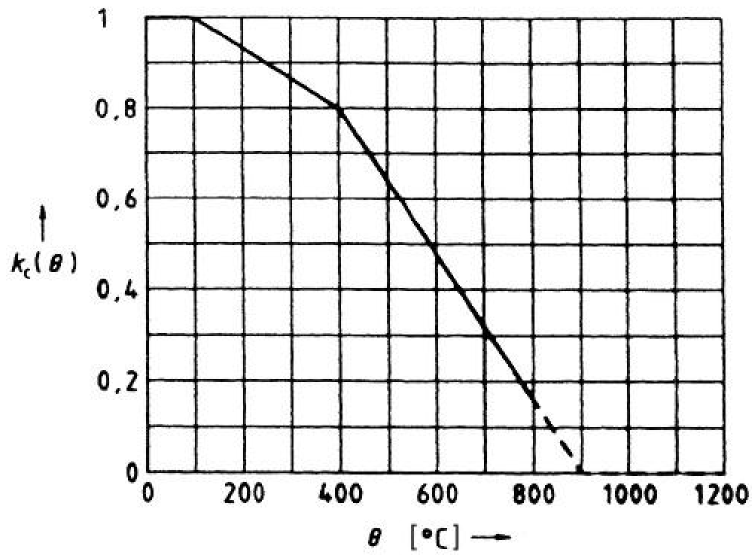

Fig. 2. – Values of coefficient $k_c(\Theta)$ for reduction of the characteristic compressive strength of concrete.

The reduction of the characteristic compressive strength of concrete as a function of the temperature, Θ, is allowed for by the coefficient $k_c(\Theta)$ namely

$$f_{ck}(\Theta) = k_c(\Theta) f_{ck}(20°)\,. \tag{1}$$

In the absence of more accurate information the following $k_c(\Theta)$ values, applicable to concretes with siliceous aggregates, should be used (Fig. 2):

a) $k_c(\Theta) = 1$, for $20°\text{C} \leq \Theta < 100°\text{C}$;
b) $k_c(\Theta) = (1,600 - \Theta)/1,500$, for $100°\text{C} \leq \Theta < 400°\text{C}$;
c) $k_c(\Theta) = (900 - \Theta)/625$, for $400°\text{C} \leq \Theta < 900°\text{C}$;
d) $k_c(\Theta) = 0$, for $900°\text{C} \leq \Theta \leq 1,200°\text{C}$.

The reduction of the characteristic strength of a reinforcing steel as a function of the temperature, Θ, is allowed for by the coefficient $k_s(\Theta)$ as follows:

(2) $$f_{sk}(\Theta) = k_s(\Theta)\, f_{yk}(20°)\,.$$

For tension reinforcement in beams and slabs, where $\varepsilon_{s,fi} \geq 2\%$, the strength reduction may be used as given below (Fig. 3, curve *1*):

a) $k_s(\Theta) = 1$, for $20°\text{C} \leq \Theta < 350°\text{C}$;
b) $k_s(\Theta) = (6,650 - 9\Theta)/3,500$, for $350°\text{C} \leq \Theta < 900°\text{C}$;
c) $k_s(\Theta) = (1,200 - \Theta)/5,000$, for $700°\text{C} \leq \Theta \leq 1,200°\text{C}$.

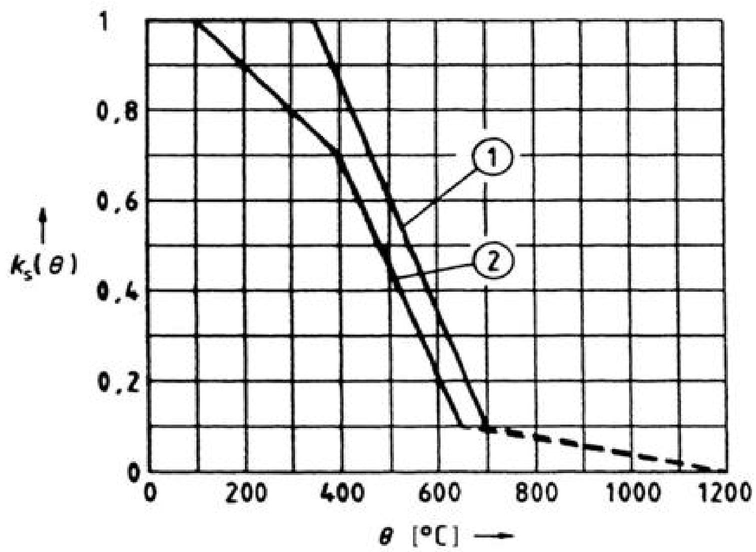

Fig. 3. – Values of coefficient $k_s(\Theta)$ for reduction of the characteristic strength of a reinforcing steel.

For compression reinforcement in columns and compressive zones of beams and slabs the strength reduction at 0.2% proof strain should be used as given below (Fig. 3, curve *2*). This may be applied for tension reinforcement too when $\varepsilon_{s,fi} < 2\%$ using the simplified or general calculation methods namely

a) $k_s(\Theta) = 1$, for $20°\text{C} \leq \Theta < 100°\text{C}$;
b) $k_s(\Theta) = (1,100 - \Theta)/1,000$, for $100°\text{C} \leq \Theta < 400°\text{C}$;
c) $k_s(\Theta) = (8,300 - 12\Theta)/5,000$, for $400°\text{C} \leq \Theta < 650°\text{C}$;

d) $k_s(\Theta) = (1,200 - \Theta)/5,500$, for $650°C \le \Theta \le 1,200°C$.

The reduction of the characteristic strength of a prestressing steel as a function of the temperature, Θ, is allowed for by the coefficient $k_p(\Theta)$ namely

(3) $$f_{pk}(\Theta) = k_p(\Theta)\, f_{pk}(20°)\,.$$

In the absence of more accurate information the following $k_p(\Theta)$ values should be used for prestressing steel bars (Fig. 4, curve *1*):

a) $k_p(\Theta) = 1$, for $20°C \le \Theta < 100°C$;
b) $k_p(\Theta) = (1,600 - \Theta)/1,500$, for $100°C \le \Theta < 250°C$;
c) $k_p(\Theta) = (700 - \Theta)/500$, for $250°C \le \Theta \le 650°C$;
d) $k_p(\Theta) = (1,000 - \Theta)/3,500$, for $650°C \le \Theta \le 1,000°C$;
e) $k_p(\Theta) = 0$, for $1,000°C \le \Theta \le 1,200°C$.

For prestressing steel wires and strands the following $k_p(\Theta)$ values should be used (Fig. 4, curve *2*):

a) $k_p(\Theta) = 1$, for $20°C \le \Theta < 100°C$;
b) $k_p(\Theta) = (850 - \Theta)/750$, for $100°C \le \Theta < 250°C$;
c) $k_p(\Theta) = (650 - \Theta)/500$, for $250°C \le \Theta \le 600°C$;
d) $k_p(\Theta) = (1,000 - \Theta)/4,000$, for $600°C \le \Theta \le 1,000°C$;
e) $k_p(\Theta) = 0$, for $1,000°C \le \Theta \le 1,200°C$.

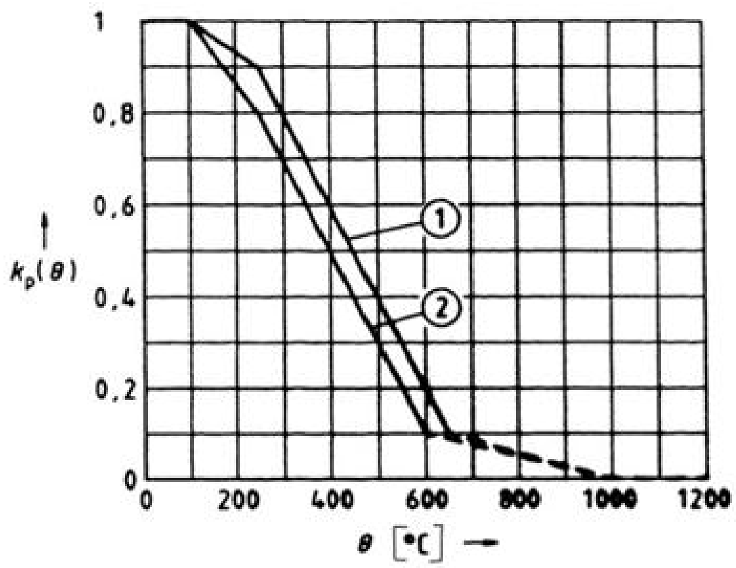

Fig. 4. – Values of coefficient $k_p(\Theta)$ for reduction of the characteristic strength of a prestressing steel.

For more accuracy, coefficients $k_c(\Theta)$, $k_s(\Theta)$) and $k_p(\Theta)$ can be obtained for various materials from experimental tests. These tests are useful for materials defined according with norms from Romania or other countries, which are different from Eurocode 2.

When $k_c(\Theta)$, $k_s(\Theta)$) and $k_p(\Theta)$ are taken from documented data they should be derived from tests performed under constant stress and variable temperature (transient tests).

3. Simplified Design Method

The simplified calculation method described below determines the ultimate load bearing capacity of a heated cross section. The method is applicable to structures subjected to a standard fire exposure until the time of maximum gas-temperature.

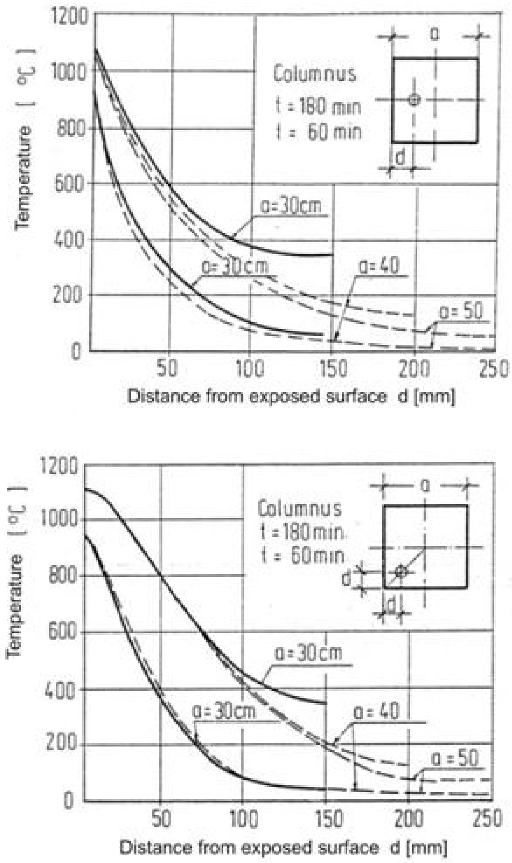

Fig. 5. – Temperature profiles for columns.

The procedure is also applicable for the calculation of the ultimate resistance at a specified time for any other fire exposure, if the temperature profiles corresponding to that exposure are known or calculated, and correct data for material properties corresponding to it are used. However, this method only provides temperature profiles and material data for the standard fire exposure up to the time of maximum gas temperature.

The used procedure recommends firstly to determine the temperature profile of the cross-section, to reduce the concrete cross-section, the strength and the short term modulus of elasticity of concrete and reinforcement and then to calculate the ultimate load bearing capacity of the construction with the reduced cross-section and to compare the capacity with the relevant combination of actions.

To evaluate the strength of materials at high temperatures is important to know the value of this temperature inside of the element. Temperatures in a concrete structure exposed to a fire may be determined from tests or by calculation. Where more accurate information is not available may be used the temperatures given in Fig. 5 for columns, Fig. 6 for slabs and Fig. 7 for beams and girders.

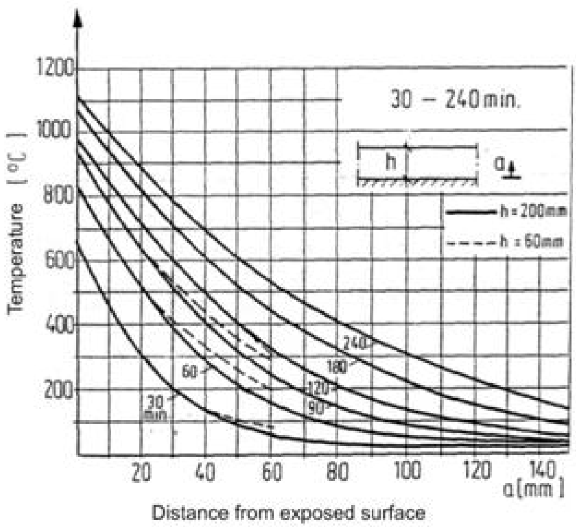

Fig. 6. – Temperature profiles for slabs.

The temperature profiles given in Figs. 5, . . . , 7 are acceptable for determining the temperatures in cross-sections with siliceous aggregate and exposed to a standard fire up to the time of maximum gas temperature. The profiles are conservative for most other aggregates, but not in general for other than the standard fire exposure. It is assumed that the isotherms in the compression zone of a rectangular cross section are parallel with the sides.

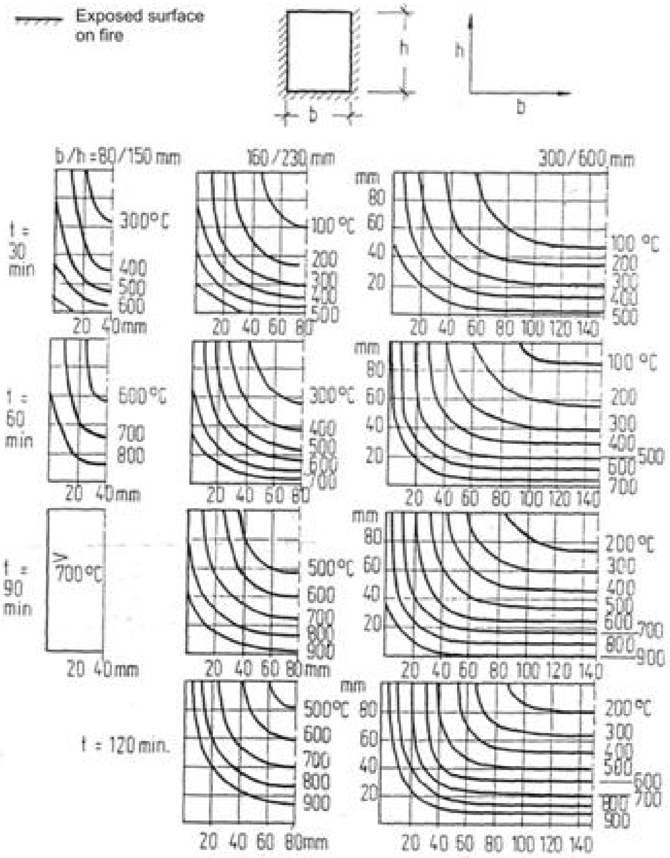

Fig. 7. – Temperature profiles for beams and girders.

The fire damaged cross-section is represented by a reduced cross-section, ignoring a damaged zone of thickness a_z at the fire exposed surfaces, as shown in Fig. 8. For a rectangular shape exposed to fire on one face only the width is assumed to be w, (Fig. 8 *c* and the flange of Fig. 8 *f*).

Where two opposite faces are exposed to fire the width is assumed to be $2w$ (Figs. 8 *a*, 8 *b*, 8 *d*, 8 *e* and 8 *f*).

For any rectangular part of a member an equivalent wall of thickness $2w$ is considered for which the thickness, a_z, is calculated. For example the slab in Fig. 8 *c* is related to the equivalent wall in Fig. 8 *d*, and the flange of Fig. 8 *f* is also related to the equivalent wall in Fig. 8 *d*, but the web of Fig. 8 *f* is related to the equivalent wall of Fig. 8 *a*.

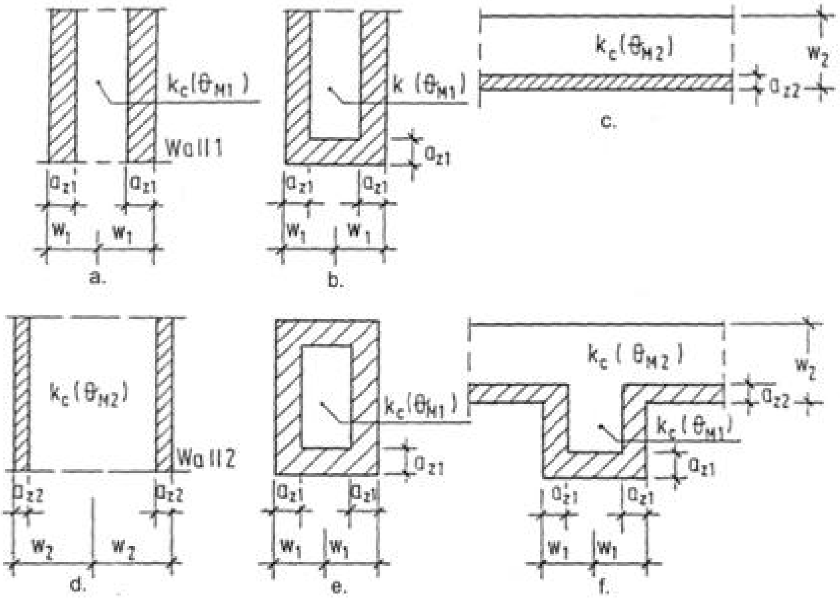

Fig. 8. – Reductions of cross-sections found by means of equivalent walls exposed to fire on both sides

The damaged zone, a_z, is estimated for an equivalent wall exposed on both sides as follows:

a) The half thickness of the wall, w, is divided into n parallel zones of equal thickness, where $n \geq 3$.

b) The temperature is calculated for the middle of each zone.

c) The corresponding reductions coefficient, $k_c(\Theta_i)$, of the compressive strength of the concrete are determined from Fig. 2.

d) The mean reduction coefficient incorporating a factor $(1 - 0.2/n)$ which allows for the variation in temperature within each zone may be calculated using equation

$$k_{c,m} = \frac{1 - 0.2/n}{n} \sum_{i=1}^{n} k_c\left(\Theta_i\right). \tag{4}$$

e) The width of damaged zone for beams, slabs and members subjected to in-plane shear may be calculated using relation

$$a_z = w\left[1 - \frac{k_{c,m}}{k_c\left(\Theta_M\right)}\right], \tag{5}$$

where $k_c(\Theta_M)$ denotes the reduction coefficient for concrete at point M.

f) The width of damaged zone for columns and walls may be calculated using relation

$$a_z = w\left\{1-\left[\frac{k_{c,m}}{k_c\left(\Theta_M\right)}\right]^{1.3}\right\}. \tag{6}$$

The reinforcement is taken into account with reduced strength and modulus of elasticity according to the temperature of each bar, even if it is placed outside the reduced cross-section.

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PROIECTAREA ELEMENTELOR DIN BETON ARMAT LA ACŢIUNEA FOCULUI

(Rezumat)

Dezvoltarea din ce în ce mai accentuată pe verticală a clădirilor impune restricţii speciale privind comportarea structurii la acţiunea focului. Se prezintă principiile de bază ale proiectării elementelor din beton armat la acţiunea focului.