

Assessment of the level of ultra-high temperature effects on structural elements

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Abstract. The article presents the study of the level of fire effects on structures based on their material, size, temperature and duration of the fire. The necessity of technical inspection of affected buildings as an important factor for decision-making on the use of structures subjected to fire exposure is shown. The goal of the technical inspection is the determination of the residual bearing capacity of the fire-damaged structures.

Key words. Ultra-high performance fiber reinforced concrete (UHPFRC), material properties, design procedures, fire effects, residual bearing.

1. Introduction

As a result of fires in buildings their constructions are damaged up to final destruction. Extent of fire impact on building constructions depends on their material, sizes, temperature and duration of the fire. Let us consider each of them:

1.1. Wooden constructions

At fire impact on wooden constructions combustible gases emitted from them burn down out of wood. Under the influence of distillation wood heats up and becomes charred.

Humidity of wood decreases and durability of construction uncharred layers increases. While putting out a fire with water the wood is moistened and its durability becomes equal to that it had before the fire.

At restoration of the wooden burned construction all charred layer of wood has to be removed since it keeps an unpleasant smell for a long time.

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In wooden constructions the cross section of the elements minus the thickness of the charring is defined.

Design resistance of wood is accepted the same as for the wood which has not undergone fire impact [1].

1.2. Steel constructions

Steel constructions are made of low-carbon steel. When heating steel elements above 600 °C they receive big deformations and cannot be used as intended [1].

Strengthening of the steel constructions which have undergone fire effect is made by the same methods as of the constructions not damaged by the fire.

In steel elements there is determined their cross section considering time of the building erection. Deflections in the vertical and horizontal plane are defined.

Design resistance of steel is accepted depending on time of rolled steel production without fire effect. At the same time axis bending existence of the damaged element is taken into consideration.

1.3. Stone constructions

Stone constructions (walls, columns, arches) are damaged from a surface. Damages are expressed by brick peeling depth. Yet constructions from a silicate brick sustain deeper damage in comparison with those from a ceramic brick.

As a result of thermal effect in case of fire stone walls and arches can receive big deformations leading to cracks formation. The stone constructions damaged in a fire are strengthened in the same way as those which have not undergone fire effect [2].

The residual bearing capacity of a stone masonry also depends on temperature and duration of the fire. Stones of masonry and mortar are damaged only on its surface. In calculations of the residual bearing ability it is necessary to consider existence of cracks in a laying.

1.4. Reinforced concrete constructions

The most stratifiable is the accounting of fire damage extent of reinforced concrete constructions in a fire. Heterogeneity of the materials forming reinforced concrete when heating leads to different temperature deformations and bond breaks between cement stone, large and small fillers and reinforcing.

In reinforced concrete elements irreversible changes of mechanical properties, decrease in durability on compression and stretching, additional deflections result. Changes of mechanical properties of concrete when heating and subsequently cooling are estimated very much approximately now.

It complicates definition of the bearing capacity of the reinforced concrete elements subjected to fire and subsequent cooling, particularly for the compressed elements. When calculating the residual bearing capacity of reinforced concrete elements the cross section of an element is divided into strips of different thickness depending on the element cross section sizes, 50–100 mm.

2. Research method

Design resistance of concrete is defined by multiplication of design resistance of the undamaged concrete by the decreasing coefficients calculated according to tables and schedules. When heating concrete reaches over 500 °C, its resistance to compression and resistance of fittings located in it are accepted equal to zero. The stretched fittings of the class A-240, A-300, A-400 and A-500, heated to temperature above 600 °C, also are not considered in calculations [3].

Design resistance R_{bI} to compression of the concrete layers damaged by fire after cooling can be determined by a formula [2]

$$R_{\mathrm{bI}} = \gamma_{\mathrm{bI}} R_{\mathrm{b}}\,, \tag{1}$$

where γ_{bI} is a coefficient of decrease in design resistance of concrete to compression after cooling depending on heating temperature and R_{b} is the initial resistance.

Design resistance $R_{\mathrm{bt.I}}$ at stretching of the concrete layers damaged by fire is determined by the formula

$$R_{\mathrm{bt.I}} = \gamma_{\mathrm{bt.I}} R_{\mathrm{bt}} \tag{2}$$

where $\gamma_{\mathrm{bt.I}} = \gamma_{\mathrm{bI}} - 0.2(1 + 0.1t)$, t being the concrete temperature rise.

The depth of concrete warming up depending on warming up temperature of the construction surface and high temperatures impact duration can be determined from Fig. 1.

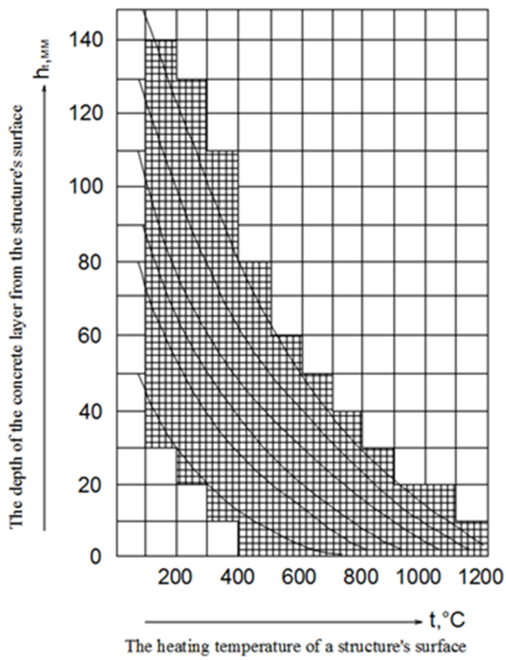

Fig. 1. Dependence of temperature distribution in concrete layers [2]

The modulus of elasticity E_{bI} of concrete which was exposed to heating and the subsequent cooling is determined by the formula [3]

$$E_{\mathrm{bI}} = \beta_{\mathrm{b}} E_{\mathrm{b}}\,, \tag{3}$$

where β_{b} is the coefficient of decrease of the concrete modulus of elasticity and E_{b} is the initial module. Here,

$$\beta_{\mathrm{b}} = 1 - kt\,, \tag{4}$$

where the coefficient k for heavy concrete is equal to $0.17 \cdot 10^{-2}$, for haydite concrete it is $0.10 \cdot 10^{-2}$. Decrease in the modulus of elasticity of concrete when heating is irreversible after cooling.

When heating concrete over 500 °C, the module of elasticity is not considered in calculations of durability and construction deformation.

The values of design resistance R_{sI} of reinforcing to stretching when heating and the subsequent cooling are determined by the formula

$$R_{\mathrm{sI}} = \gamma_{\mathrm{sI}} R_{\mathrm{s}}\,, \tag{5}$$

where γ_{sI} is a coefficient of decrease in design resistance of reinforcing to stretching depending on heating temperature and R_{s} is the initial value.

3. Results

For the reinforcing steel of the classes A-240, A-300, A-400 located in the stretched zone at the size of the smaller side of an element section more than 300 mm and temperature of fittings heating up to 600 °C, and the size of the smaller side of an element section less than 300 mm and temperature of heating up to 600°C, $\gamma_{\mathrm{sI}} = 1$ [3].

At the smaller size of an element section side more than 300 mm and temperature of fittings heating more than 500 °C, and smaller size of the side of an element section less than 300 mm, and temperature of fittings heating more than 600 °C $\gamma_{\mathrm{sI}} = 0$.

For reinforcing steel of the class A-500, A-600 at temperature of fittings heating to 400 °C, and at temperature of fittings heating more than 400 °C $\gamma_{\mathrm{sI}} = 0$.

The values of design resistance R_{sc} of reinforcing to compression can be determined by the formula

$$R_{\mathrm{sc}} = \gamma_{\mathrm{sI}} \gamma_{\mathrm{s2}} R_{\mathrm{s}}\,, \tag{6}$$

where γ_{s2} is the coefficient considering decrease in bond of fittings with concrete after heating and cooling.

For smooth hot-rolled reinforcing rods

$$\gamma_{\mathrm{s2}} = 1 - 0.001t\,. \tag{7}$$

For hot-rolled reinforcing rods of a periodic profile

$$\gamma_{\mathrm{s2}} = 1 - 0.001(0.1 + 0.001t\,. \tag{8}$$

The module of reinforcing steel deformation after heating and the subsequent cooling is accepted at $\gamma_{\mathrm{sI}} = 1$ as for fittings which have not undergone heating.

When considering irregularity in distribution of the concrete strength on the thickness of the element which has been subjected to fire, reduction of the unequally heated layers of concrete to a homogeneous material is made.

The reduction coefficient of parts (concrete layers) of the section of the element damaged by fire $\alpha_{\mathrm{bt.I}}$ should be accepted proportional to the ratio of concrete strength of the considered layer $R_{\mathrm{bt.I}}$ to the strength of the base layer $R_{\mathrm{b\,loc}}$

$$\alpha_{\mathrm{bt.I}} = \frac{R_{\mathrm{bt.I}}}{R_{\mathrm{b\,loc}}} \tag{9}$$

For decision-making on the use of constructions subjected to fire exposure in case of fire, a technical inspection of affected buildings is made. The purpose of the technical inspection is to determine the residual bearing capacity of the fire-damaged constructions.

Two examples of validation of the model with experimental results are shown in Figs. 2 and 3, considering tests results from [4] and [5], respectively on UHPC and HSC specimens. From the comparison, one can observe a good approximation between the proposed model and the experimental curves.

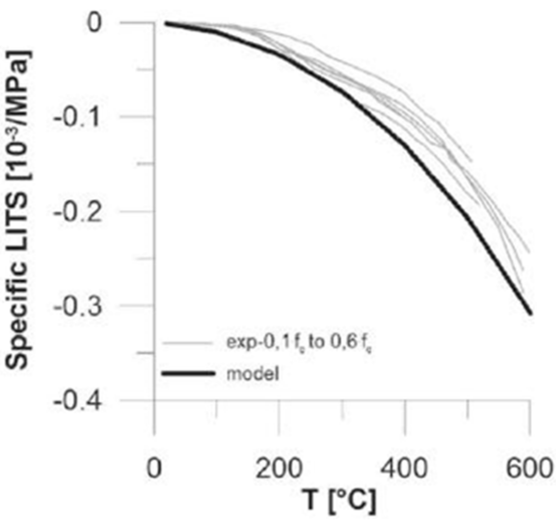

Fig. 2. Comparison between LITS model and UHPC experimental results [4]

Usually after the fire there is no accurate data on heating temperature of constructions and duration of the fire. It reduces the accuracy of determination of residual strength of reinforced concrete elements after the fire. Assessment of key parameters of the fire is made by two methods.

The pressure regarding distance from fire is illustrated in Fig. 4. The pressure peak is located at the saturated layer where increasing temperature leads to the saturated vapor pressure increase. This mechanism explains partly why pressure

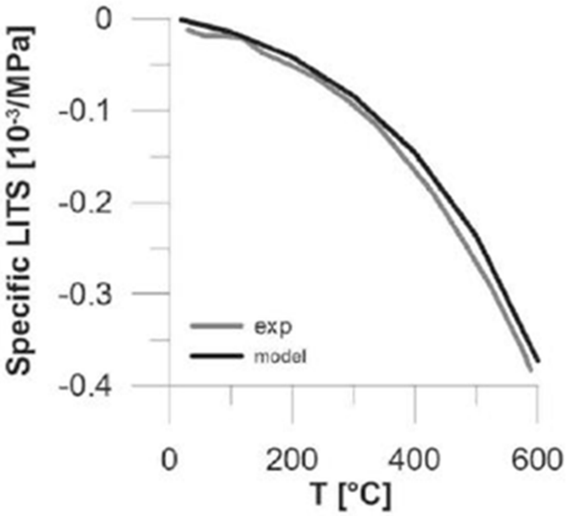

Fig. 3. Comparison between LITS model and HSC experimental results [5]

peaks dramatically increase around $T = 200\,^{\circ}\mathrm{C}{\sim}230\,^{\circ}\mathrm{C}$ because of abrupt increase of saturated vapor pressure thermodynamically at that temperatures.

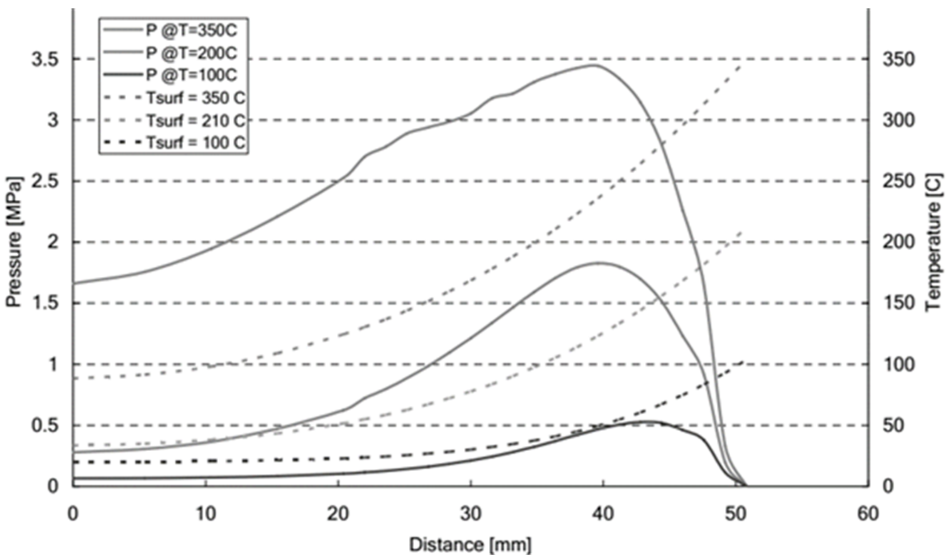

Fig. 4. Pressure versus temperature profile from core to outer surface

4. Conclusion

The theoretical method is based on dependence of the design parameters of the fire on the type of the room, the size of window and door openings and ventilation conditions. While there are factors characterizing the constructions and the factors that determine burning conditions. Changes in these factors affect the development of fires characterized by time, intensity of development and the thermal effects on the protecting structures of the room.

In the calculations the parameters are given to the standard mode, which explore the limits of fire resistance of building structures. The calculations for fires parameters in buildings with building structures made of insulating combustible and non-combustible materials are produced according to the dependencies allowing to evaluate the integral thermal parameters of the freely developing fire such as the temperature of the gaseous medium in the fire center, the temperature of the walls and floors, the density of the heat flow.

Temperature mode of the three-dimensional fire regulated by ventilation when burning, is calculated provided that the fire load is uniformly distributed over the floor area, window openings during the fire and open area ratio of openings to floor area is 2.5–35 %. The minimum duration of the initial stage of the fire is calculated provided that openings of the premises are closed until the moment of the flash defined by the temperature of the flash [4].

The mechanical characterization of the material can be carried out by means of bending tests, followed by inverse analysis, to determine the tensile constitutive law. In this case, models based on continuum mechanics, assuming a cross-sectional equilibrium and Bernoulli principle (plane section), can be applied. The characteristic length (crack-opening-strain relationship) can be taken as the average space between cracks during strain hardening and as function of the beam depth during crack localization. The experimental and theoretical evaluation method is based on determination of the fire temperature when analyzing the appearance, condition and color of various materials located in a fire zone. Data on changes in the external parameters of bearing and enclosing structures, as well as materials located in the fire area at high temperatures are presented in the normative literature [5].

By the results of inspection constructions are classified by a damage rate and, if necessary, detailed inspection of the bearing structures by the destructive and nondestructive methods for determination of the residual bearing capacity of the fire-damaged structures is appointed.

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