Original article

Analysis of the load-carrying capacity of a steel frame under fire conditions

Piotr Bilko[1], Szymon Sawczynski[2]*

[1]Faculty of Geodesy and Geospatial and Civil Engineering, University of Warmia and Mazury, Poland
piotr.bilko@uwm.edu.pl

[2]Faculty of Geodesy and Geospatial and Civil Engineering, University of Warmia and Mazury, Poland
sz.sawczynski@uwm.edu.pl

INFORMATIONS	ABSTRACT
	This article aims at presenting the basic principles of designing steel structures according to Eurocode PN-EN 1993-1-2, i.e. with respect to ensuring the appropriate level of safety for such structures in case of a fire developing into flashover. The load-carrying capacity of a steel structure, serving as an example, was assessed on the basis of a static strain-stress analysis. The analysis was conducted regarding changes in the temperature of structural members, both the exposed and the fireproof protected ones, under fire conditions.
* Corresponding author	

1. Introduction

A fire, which is the uncontrolled propagation of fire, usually results in greater or lesser financial losses. Under favourable circumstances the destructive force of fire or, more specifically, a high temperature accompanying such fire, may lead to a construction disaster, caused by the permanent destruction of a building structure consumed by fire. Steel structures in particular are sensitive to a high temperature. In normal situations the load-carrying capacity of a steel structure is ensured by the selection of appropriate dimensions of a cross-section, or the slenderness of a member, and steel strength, and, then, by the verification of proper conditions for load-carrying capacity. During a fire, the temperature in the building represents an additional factor affecting the load-carrying capacity. From a designer's perspective, the appropriate fire resistance class should be ensured for both structural members and the whole building. The paper presents solutions, adopted in relevant standards, for taking account of temperature in the process of structure sizing. The methods of protecting steel structures against a high temperature are also described. The article contains the analysis of a steel frame under fire conditions. Legal and penal aspects regarding a fire can be

found in reference [7], and reference [2] focuses on the causes, and, thus, on the ways of avoiding uncontrolled fire. Handbooks on the fire safety of steel structures include, for example, Biegus [1] and Maslak [4].

2. Effect of temperature on the load-carrying capacity of a structure

Under fire conditions the load-carrying capacity of respective members, and in particular those heated to the greatest extent, decreases drastically with time during a fire. The higher is the fire temperature the closer is a structural member to achieve the ultimate limit state, not to mention the already exceeded serviceability limit state. The methods of analysing steel structures make it possible to determine the critical load and the related generalised internal forces or critical temperature. The critical load is defined as the minimum applied load that will result in the destruction of a member heated to the temperature T. The critical temperature is defined as the temperature at which material properties become impaired to such an extent that a steel structural member is no longer able to carry a definite load or level of stress [4]. The standard model assumes that the course of a fire over time is described by a single curve $\theta_g - t_{fi}$ (temperature of combustion gases [°C] – duration of a fire). The plotted curve is given by the following formula:

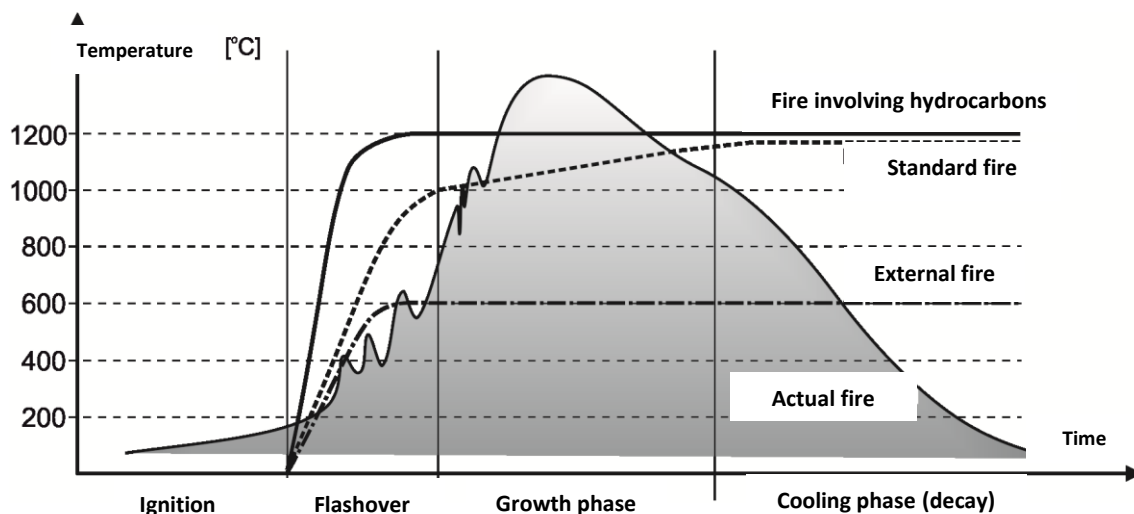$$\theta_g = 20 + 345 \log_{10}(8t_{fi} + 1) \tag{1}$$

Fig. 1. Temperature-time curves depending on the type of fire

Source: own elaboration

The curve described by formula (1) is presented in Figure 1, together with three other curves, which together are referred to as the nominal fire curves. The fire temperature varies over time and with respect to location within the interior of a building. It depends on the type of a fire and material or substance that burns and on the presence of ventilation in the room (or holes in the room). The temperature increases most rapidly during a fire involving hydrocarbons.

According to PN-EN 1993-1-2 [6] different methods can be employed to assess the fire resistance rating, on the basis of which the ultimate limit state under fire conditions can be verified. However, to determine the extent of utilisation of the load-carrying capacity the effects of actions are required. They are determined on the basis of [6], where factors derived from the theory of combinations are taken into account, among others. However, PN-EN 1993-1-2 standard makes use of a simplified method, consisting in conducting a structural analysis at a normal temperature and reducing the obtained design values of effects of actions, according to the following relationship:

$$E_{fi,d} \leq R_{fi,d,t},$$ (2)

where:

$E_{fi,d}$ - design value of effects of actions under fire conditions,

$R_{fi,d,t}$ - appropriate design load-carrying capacity under fire conditions.

The design value of effects of actions is obtained from the following formula:

$$E_{fi,d} = \eta_{fi} E_d,$$ (3)

where:

E_d - design value of effects of actions at a normal temperature and for a combination of normal loads,

η_{fi} - reduction factor used to determine the design value of effects of actions in a fire situation.

It can be observed that the design values of effects of actions depend on one reduction factor. Its value is calculated from the formula which takes account of factors derived from the theory of combinations and characteristic loads. Figure 2 presents the value of factor η_{fi}, depending on the relationship between actions $Q_{k,l}/G_k$.

During a fire, with an increase in temperature, properties of steel undergo changes. Already at a temperature of 80°C physical and mechanical properties of steel start to become impaired. At 300°C strains occur more often than at normal temperatures. Thence, it can be stated that the modulus of elasticity decreases with a rise in temperature. At 700°C the strength of steel decreases to 25% of its baseline value. Steel, while losing such properties as Young's modulus and strength, deteriorates also with respect to its load-carrying capacity and rigidity. It follows that at high temperatures a steel structure will reach the ultimate and serviceability limit states significantly faster.

According to standard [6] the load-carrying capacity under fire conditions is determined in the same way as for PN-EN 1993-1-1 [5], however, in the fire standard the load-carrying capacity is cut down by reduction factors. The effects of actions are decreased depending on the reduction factor . The values of these factors are defined as quotients of a given material property at a high temperature by the same property value corresponding to a temperature of 20°C. The standard provides a general formula for calculating the design values of material mechanical properties $X_{d,fi}$
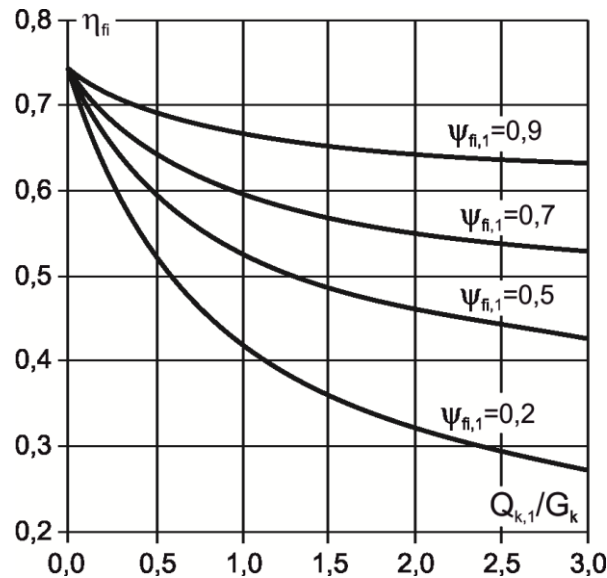
Fig. 2. Values of factor η_{fi}, depending on the relationship $Q_{k,l}/G_k$

Source: own elaboration

$$X_{d,fi} = k_\theta X_k / \gamma_{M,fi} \, , \tag{4}$$

where:

X_k - characteristic value of a strength or strain property (in general f_k or E_k) at a normal temperature, according to PN-EN1993-1-1,

k_θ - reduction factor used to determine material strength and strain properties dependent on temperature,

$\gamma_{M,fi}$ - partial factor concerning material properties in a fire situation.

From relationship (4) any material property at any temperature can be calculated. A reduction in the load-carrying capacity depends also on other factors such as the type of active loading (bending, compression, tension), robustness of a cross-section, type of a member or class of the cross-section under consideration. During a fire greater bucking lengths are taken into account, which significantly affects the load-carrying capacity of the structure. PN-EN 1993-1-2 standard lists three main reduction factors k_θ:

$k_{y,\theta} = f_{y,\theta}/f_y$ - relationship between the effective yield point and the yield point at a temperature of 20°C;

$k_{p,\theta} = f_{p,\theta}/f_y$ - relationship between the proportional limit and the yield point at a temperature of 20°C;

$k_{yE,\theta} = E_{a,\theta}/E_a$ - relationship between the modulus of elasticity and the modulus of elasticity at a temperature of 20°C.

The above factors are used for the specified values of load-carrying capacity, classes and cross-section of steel members.
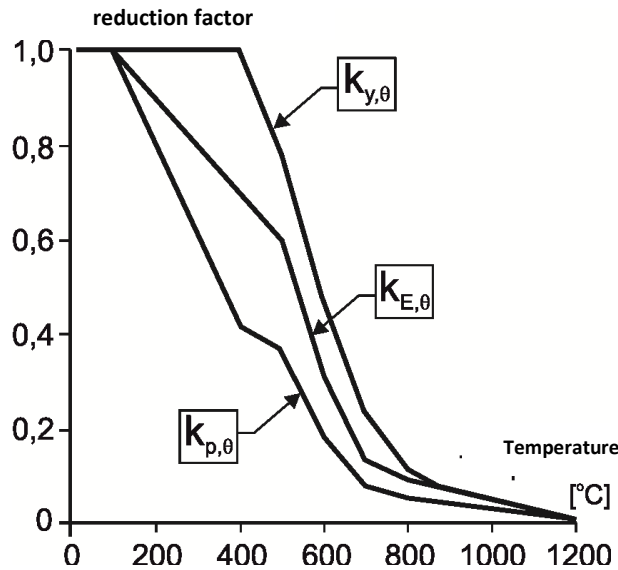

Fig. 3. Reduction factors as a function of temperature [1]

Source: own elaboration

The type of factor is selected depending on the nature of stresses acting on the cross-section, type of a member and class of a cross-section. On the basis of Figure 3 it can be observed that for the temperature of a member of 1200°C the value of each of the factors drops to zero, and hence, according to Eurocode, after this temperature is exceeded, the structure no longer transmits any actions. When a steel structure is designed taking account of fire conditions, it is necessary to obtain the load-carrying capacity which will ensure that the building structure meets the fire resistance criteria for a specified period of time, making it possible to evacuate people staying in the building.

At a given point in time during a fire the temperature of a member is not equal to the ambient temperature. The heating rate of steel depends on its thermal conductivity. The more is steel heated, the lower is its thermal conductivity, and, therefore, with a rise in temperature the heating rate will decrease. The rate of this change depends on the exposed surface area (exposed to the direct operation of fire). The larger is the exposed surface area of a cross-section along the member length, the higher is the heating rate of steel and the shorter is the time needed to reach the limit states.

Since the high temperature occurring during a fire may cause considerable losses, steel members are protected against its effects. Such protection can be ensured by, for example, intumescent paints, different types of spray coatings or gypsum board cladding. Table 1 presents the results of the tests performed by Wozniak [8] regarding the fire resistance of steel beams with IPE400 sections with fireproof protection. The average temperatures of the sections correspond to the moment when they lose their load-carrying capacity. Deflections of the order of $L/30$ were adopted as a criterion for losing the load-carrying capacity. Relying on the analysis of the results of the tests it has

been determined that the kind of fireproof protection applied does not have a significant influence on the average value of critical temperature.

Table 1. Temperature of the investigated steel beams at the point of losing their load-carrying capacity

Insulation type	Temperature [°C]			
	lower flange	web	upper flange	average in a member
Intumescent paint 1	755	793	587	712
Intumescent paint 2	720	706	541	671
Spray coating 1	785	652	500	646
Spray coating 2	724	692	533	650
Cladding 1	684	688	662	678
Cladding 2	676	667	644	662

Source: own elaboration

3. Analysis of the load-carrying capacity of a structure

This section presents the results of analysis of the load-carrying capacity, conducted for a steel structure exposed to the effect of a high temperature during a fire. One of the halls of the poultry farm, which was damaged as a result of a fire, is used as an example. The fire occurred inside the turkey house, in its central part. The effects of the fire can be observed in Figure 4, showing the members characterised by greater slenderness distorted as a result of the effects of the high temperatures. Buckling occurred in both the roof purlins and wall girts, which were made of open sections, and also in the roof bracings made of steel bars with a small cross-section.

Fig. 4. Static diagram of the frame adopted for calculations. Dimensions in [cm]

Source: Sonia Kucharczyk

The building of the analysed turkey house has a ground floor only and it is 90.0 m long and 12.0 m wide. A ridge roof with a roof slope inclination of 16° was covered with trapezoidal metal sheet. The external composite walls were made of profiled metal sheet. The load-bearing structure is formed by single-bay steel frames with a span of 12.0 m and spacing of 4.5 m. The columns and the rafters are made of 2C160 and 2C220 sections, respectively. Taking into consideration the way of seating the rafters on the column and the solution used for the ridge node point, in the static diagram these nodes were adopted as the hinged nodes. It was assumed that the columns were fully fixed in the column footing. The frame was braced with steel pipes Ø80, suspended at the middle by means of a hanger made of the same section. The static diagram adopted for calculations is presented in Figure 5. As the fire occurred in the spring-summer period, the weight of snow was disregarded in the static calculations, which were reduced to permanent loads only, i.e. self-weight of the frame and the roofing.
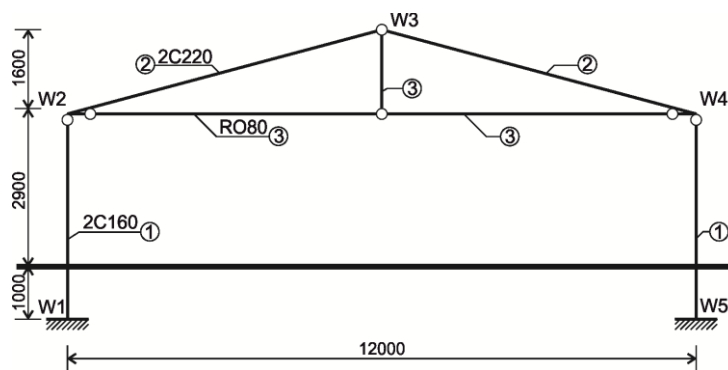

Fig. 5. Static diagram of the frame adopted for calculations. Dimensions in [mm]

Source: own elaboration

On the assumption that the temperature distribution is uniform in the whole analysed frame, its load-carrying capacity can be specified with the use of the critical temperature. For the frame the critical temperature is the temperature at which, as a result of the deterioration of material properties at the flashover phase, the frame is no longer able to carry a definite load and transforms into a mechanism [3]. The formation of a sufficient number of plastic hinges transforming the frame into a mechanism can be adopted as a criterion for the load-carrying capacity. In the analysed example the mechanism is formed as a result of thermal softening of the support nodes. Because of the symmetry it can be assumed that the thermal softening of nodes W1 and W5 occurs simultaneously. For calculation purposes a uniform distribution of temperatures in the whole frame was adopted. The plastic resistance of the cross-section of the column totals $W_{pl,y} = 276 \text{cm}^3$. The static analysis was conducted for different temperatures, thus simulating the point in time corresponding to respective phases of a fire. The results are summarised in Table 2. The third column contains the reduced values of the modulus of elasticity, which were taken into account in the static calculations on an ongoing basis. The sixth column includes the maximum moments in the support nodes which were compared to the reduced, for a specific point of time, plastic moment of the cross-section. At a temperature of 800°C the cross-section moment is

higher than the one related to plastic resistance. It means that after a steel member has reached this temperature at the most strained node, a plastic hinge will be formed. Then, a change to the static diagram occurs. At the constant level of internal forces the plastic hinge makes it possible for plastic deformations to increase in an uncontrolled manner, which is commensurate with the transformation of the structure into a mechanism. It follows that at the moment a temperature of 800°C is reached, the structure ceases to carry loads. The load-carrying capacity, and, concurrently, the critical temperature for this structure can be adopted at the level of approx. 800°C.

Table 2. Results of static calculations of the frame

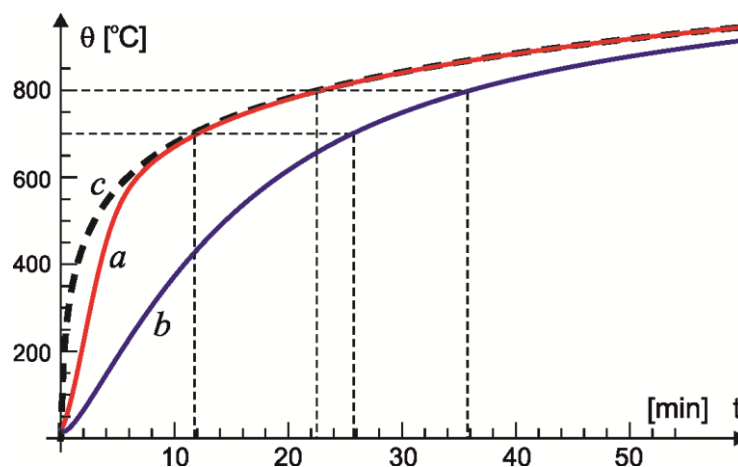| Temp. [°C]	$k_{E,\theta}$ [-]	$E_{a,\theta}$ [GPa]	$k_{y,\theta}$ [-]	$M_{pl,y,\theta}^{1-2}$ [kN·m]	$M_{y,max}^{1-2}$ [kN·m]
20	1.00	205	1.00	81.42	6.36
100	1.00	205	1.00	81.42	11.71
400	0.70	144	1.00	81.42	19.18
500	0.60	123	0.78	63.50	21.98
600	0.31	64	0.48	39.08	17.33
700	0.14	29	0.24	19.54	11.52
800	0.10	21	0.12	9.77	9.99

Source: own elaboration

Fig. 6. Average temperature of a steel member during a fire: a) steel section 2C160 exposed, b) steel section 2C160 with fireproof protection, c) standard temperature

Source: own elaboration

If a standard fire scenario is assumed, the structure will lose its load-carrying capacity after approx. 22 min. When fireproof protection is used, in the form of the box cladding of girders with gypsum boards, the time for evacuation is extended by additional 14 min (Figure 6). For the sake of comparison, a frame with an over-rigid system, i.e. with fixed nodes W2, W3 and W4, was analysed. The critical temperature dropped to 700°C. Fixed nodes are frequently encountered in the single-bay steel frames. The hinged nodes designed for this structure, introducing additional degrees of freedom to the structure, worked to its benefit in this case. Furthermore, the use of fireproof protection would triple the time before the critical temperature is reached.

Due to the effects of a fire the analysed structure was substantially deformed. Bracing bars, which sagged by 7 cm, were the frame members which were subjected to the greatest deformation at the critical temperature. This deformation should not be treated as destructive, as the relation L/f amounts to at least 77. The real fire scenario in the burnt hall was probably different from the one adopted for the analysis and it was quickly extinguished. This explains the relatively moderate damage to the structure. The hinged nodes enabled the rafters to extend to a certain extent under the influence of a high temperature, without creating any additional significant stresses in the frame structure, whereas the secondary members, easy to replace, sustained permanent damage. It is also necessary to repair the damaged paint coating.

Conclusions

The high temperature impairs significantly the properties of steel, due to which the structure reached its limit states in a considerably shorter time. During a fire, because of the high value of thermal conductivity a steel structure is heated faster than structures made of other materials. It results in significant deformations of the steel structure and the redistribution of internal forces. Steel structures are highly susceptible to thermal deformations and formation of plastic hinges during a fire. With unrestricted access to standards and the results of experimental tests it is not difficult to understand the behaviour of structures under fire conditions. It is important, however, to be able to determine properly the level of fire safety of the structure. For this purpose it is possible to employ methods based on fire models, including advanced analyses of heat or mass flows, thermal condition models or mechanical models. In this paper the static analysis was undertaken of the load-carrying capacity of a steel structure subjected to the effects of a fire. The structure of the existing facility destroyed by a fire was selected. The provided example is not a complex one, however, the authors' intention was to present the fundamentals of analysis of steel structures under fire conditions. The problem of more complex structures can be the subject of further studies. In subsequent publications calculations may take account of various fire scenarios, depending on the building geometry, fire load or the extent of air inflow.

Acknowledgement

Conflict of interests

The author declared no conflict of interests.

Author contributions

All authors contributed to the interpretation of results and writing of the paper. All authors read and approved the final manuscript.

Ethical statement

The research complies with all national and international ethical requirements.

ORCID

The authors declared that they have no ORCID ID's

References

1. Biegus A., *Projektowanie konstrukcji stalowych wedlug Eurokodu 3. Bezpieczenstwo pozarowe konstrukcji stalowych*. Wyklady, Wroclaw 2013.

2. Jaskolowski W., *Bezpieczenstwo pozarowe budynkow – prognozy na przyszlosc*, [in:] "Zeszyty Naukowe SGSP" (2014), no. 52(4), p. 116 – 123.

3. Laskowska Z., *Zasady obliczania nosnosci ram stalowych w zaleznosci od scenariusza pozaru*, [in:] "Prace instytutu techniki budowlanej" (2004), no. 4, p. 33-43.

4. Maslak M., *Odpornosc ogniowa. Nosnosc konstrukcji w warunkach pozaru,* [in:] *Budownictwo ogolne*, (ed.) J. Panas, Arkady, Warszawa 2010, t. V, p. 663 – 722.

5. PN-EN 1993-1-1 Eurokod 3: Projektowanie konstrukcji stalowych. Czesc 1-1: Reguly ogolne i reguly dla budynkow.

6. PN-EN 1993-1-2 Eurokod 3: Projektowanie konstrukcji stalowych. Czesc 1-2: Reguly ogolne i obliczanie konstrukcji z uwagi na warunki pozarowe.

7. Witczak D., *Pojecie pozaru w swietle obowiazujacych przepisow prawnych, nauki prawa i orzecznictwa*, [in:] "Zeszyty Naukowe SGSP" (2015), no. 53(1), p. 127 – 144.

8. Wozniak G., *Temperatura krytyczna elementow konstrukcji stalowych*, [in:] "Materialy Budowlane" (2008), no. 7, p. 45 – 47.

Biographical notes

Piotr Bilko – Ph.D., Eng., Junior Lecturer / Scientific discipline and specialisation: civil engineering, structural mechanics / member of the Scientific Committee of the Polish Association of Civil Engineers and Technicians (PZITB), Branch in Olsztyn / Areas of scientific interest: building structures, theory of structures, mechanics.

Szymon Sawczynski – Ph.D., Eng., Junior Lecturer / Scientific discipline and specialisation: civil engineering, geotechnical engineering / member of the Scientific Committee of the Polish Association of Civil Engineers and Technicians (PZITB), Branch in Olsztyn / Areas of scientific interest: building structures, hydraulic structures, geotechnical engineering / ten publications, researcher in two projects awarded grants.