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DESIGN OF STEEL STRUCTURES FOR FIRE SAFETY

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1 INTRODUCTION

Safety in general and fire safety in particular, after several major disasters, has become a subject of increasing importance in recent years. A general definition for the fire resistance of construction elements can be the following: the time after which an element, when submitted to the action of a fire, ceases to fulfil the functions for which it has been designed.

Fire design of steel structures is usually based on the thermal conductivity of the protective material. Design values of thermal conductivity can be determined by full-scale fire tests. These tests also show whether the protective material stays attached to the steel structure and protects it against fire as long as required.

Fire resistance of load-bearing structures can be evaluated by both full-scale fire tests and calculations. Computational determination of fire resistance requires that cross-sectional temperature distribution is known.

The beams and column parts are subject to bending and compression, thus, stress constraints should be formulated for beam and column profiles according to Eurocode 3 [1, 2].

The steel can be protected by materials such as mineral fibres, gypsum boards, concrete, intumescent paints and water-filled structures.

2 CALCULATION OF THE STEEL MECHANICAL PROPERTIES AT ELEVATED TEMPERATURES

The calculation of the yield stress and Young's modulus on elevated temperatures is according to [2]. Figure 1 and Table 1 show the reduction factors in the function of temperature between 20 and 1200 C°.

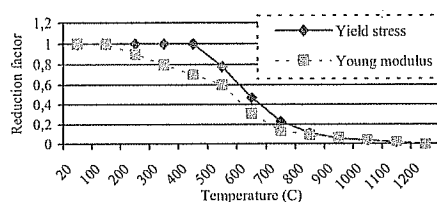


Figure 1 The yield stress and the Young's modulus reduction factors in the function of temperature

The yield strength at a given temperature can be calculated by $k_{y,\theta}$ reduction factor

$$f_{y,\theta} = k_{y,\theta} f_y \quad (1)$$

The yield strength at a given temperature can be calculated by $k_{E,\theta}$ reduction factor

$$E_{a,\theta} = k_{E,\theta} E_a. \quad (2)$$

The thermal conductivity of steel λ_a should be determined from the following:

$$\text{If } 20 [^{\circ}\text{C}] \leq \theta_a < 800 [^{\circ}\text{C}] \text{ then } \lambda_a = 54 - 3,33 \times 10^{-2} \theta_a \text{ [W/mK]}. \quad (3)$$

$$\text{If } 800 [^{\circ}\text{C}] \leq \theta_a \leq 1200 [^{\circ}\text{C}] \text{ then } \lambda_a = 27.3 \text{ [W/mK]}, \quad (4)$$

where: θ_a is the steel temperature.

The specific heat of steel can be calculated as a function of temperature as follows:

$$\text{If } 0 < \theta_a \leq 600 [^{\circ}\text{C}] \text{ then } c_a = 425 + 7.73 \times 10^{-1} \theta_a - 1.69 \times 10^{-3} \theta_a^2 + 2.22 \times 10^{-6} \theta_a^3 \text{ [J/kgK]}. \quad (5)$$

$$\text{If } 600 < \theta_a \leq 735 [^{\circ}\text{C}] \text{ then } c_a = 666 + 13002 / (738 - \theta_a) \text{ [J/kgK]}. \quad (6)$$

$$\text{If } 735 < \theta_a \leq 900 [^{\circ}\text{C}] \text{ then } c_a = 545 + 17820 / (\theta_a - 731) \text{ [J/kgK]}. \quad (7)$$

$$\text{If } 900 < \theta_a \leq 1200 [^{\circ}\text{C}] \text{ then } c_a = 650 \text{ [J/kgK]}. \quad (8)$$

The value of specific heat in the function of temperature can be seen on Figure 2.

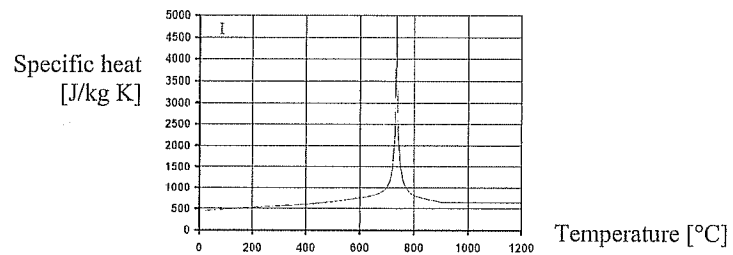


Figure 2 The value of specific heat in the function of temperature

3 CALCULATION OF THE ACTIONS FOR THE FIRE SITUATION

A general definition for the *fire resistance* of construction elements can be as follows: *the time after which an element, when submitted to the action of a fire, ceases to fulfil the functions for which it has been designed.* For the time being, the fire resistance required in most national fire safety regulations for the construction elements is referred to the standard fire (ISO 834 [3]). Therefore, since structural elements have a load carrying function, their standard fire resistance represents the time after which, when subjected to the standard fire, they can no longer resist the effects of the accidental load combination [4]

$$\sum \gamma_{GA} \cdot G_k + \psi_{1,i} \cdot Q_{k,i} + \sum \psi_{2,j} \cdot Q_{k,j} + \sum A_d(t), \quad (9)$$

where:

G_k Characteristic values of permanent actions,

$Q_{k,i}$ Characteristic value of the main variable action,

$Q_{k,j}$ Characteristic value of the other variable actions,

$A_d(t)$ Design values of actions from fire exposure, or indirect fire actions,

γ_{GA} Partial safety factor for permanent actions in the accidental situation,

$\psi_{1,i}, \psi_{2,j}$ Combination coefficients for buildings according to EN 1991-1-2.

The last term of this load combination represents the interaction between the heated element and the cold structure from which it is a part. The first terms represent the mechanical action on the heated element at the beginning of the fire, that is, the design effect of actions in fire situation at time $t = 0$, $E_{fi,d,t=0}$. For the analysis of the fire resistance of a single member, the Eurocodes state that "the internal forces and moments at supports and ends of members applicable at time $t = 0$, may be assumed to remain unchanged throughout the fire exposure", that is, $E_{fi,d,t} = E_{fi,d,t=0}$. For member analysis a reduction factor for load combination should be taken. In our case, when we have applied the calculation of the frame, which is for supporting pressure vessels, no variable loading can be considered, so $Q_{k,1}/G_k$ according to Figure 1 of EC 3₂ is $\eta_{fi} = 0.74$, the maximum.

$$E_{fi,d} = \eta_{fi} E_d \quad (10)$$

3.1 Simple calculation models

The load-bearing function of a steel member shall be assumed to be maintained after a time t in a given fire if:

$$E_{fi,d} \leq R_{fi,d,t} \quad (11)$$

where

$E_{fi,d}$ is the design effect of actions for the fire design situation, according to EN 1991-1-2 [2];

$R_{fi,d,t}$ is the corresponding design resistance of the steel member, for the fire design situation, at time t .

The design resistance $R_{fi,d,t}$ at time t shall be determined, usually in the hypothesis of a uniform temperature in the cross-section, by modifying the design resistance for normal temperature design to EN 1993-1-1 [1], to take account of the mechanical properties of steel at elevated temperatures.

3.2 Member analysis

The effect of actions should be determined for time $t = 0$ using combination factors $\psi_{1,1}$ or $\psi_{2,1}$.

As a simplification to this, the effect of actions $E_{fi,d}$ may be obtained from a structural analysis for normal temperature design as:

$$E_{fi,d} = \eta_{fi} E_d \quad (12)$$

where:

E_d is the design value of the corresponding force or moment for normal temperature design, for a fundamental combination of actions,

η_{fi} is the reduction factor for the design load level for the fire situation.

The reduction factor η_{fi} for load combination (6.10) in EN 1990 should be taken as:

$$\eta_{fi} = \frac{G_k + \psi_{fi} Q_{k,1}}{\gamma_G G_k + \gamma_{Q,1} Q_{k,1}} \quad (13)$$

The value of $\psi_{fi,1}$ is according to Figure 3.

The cross-sections may be classified as for normal temperature design with a reduced value for ε as given in (14),

$$\varepsilon = 0.85 \sqrt{\frac{235}{f_y}}, \quad (14)$$

where: f_y is the yield strength at 20 °C.

The reduction factor 0.85 considers influences due to increasing temperature.

3.3 Resistance of tension members

The design resistance $N_{fi,t,Rd}$ of a tension member with a uniform temperature θ_a should be determined from:

$$N_{fi,t,Rd} = k_{y,\theta} N_{Rd} [\gamma_{M,1} / \gamma_{M,fi}], \quad (15)$$

where:

$k_{y,\theta}$ is the reduction factor for the yield strength of steel at temperature θ_a , reached at time t ;

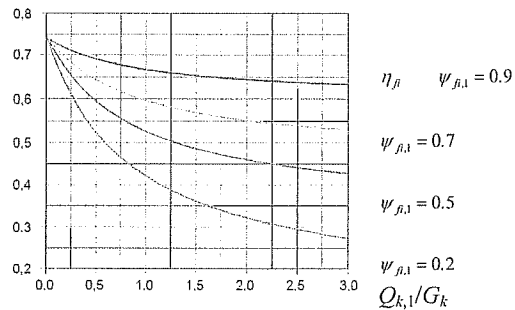


Figure 3. The value of the combination factor

N_{Rd} is the design resistance of the cross-section $N_{pl,Rd}$ for normal temperature design, according to EN 1993-1-1 [1].

3.4 Compression members with Class 3 cross-sections

The design buckling resistance $N_{b,fi,t,Rd}$ at time t of a compression member with a Class 3 cross-section with a uniform temperature θ_a should be determined from:

$$N_{b,fi,t,Rd} = \chi_{fi} A k_{y,\theta} f_y / \gamma_{M,fi}, \quad (16)$$

where: χ_{fi} is the reduction factor for flexural buckling in the fire design situation; $k_{y,\theta}$ is the reduction factor from Section 4.2 for the yield strength of steel at the steel temperature θ_a reached at time t .

The value of χ_{fi} should be taken as the lesser of the values of $\chi_{y,fi}$ and $\chi_{x,fi}$ determined according to:

$$\chi_{fi} = \frac{1}{\varphi_{\theta} + \sqrt{\varphi_{\theta}^2 - \lambda_{\theta}^2}}, \quad (17)$$

$$\text{with} \quad \varphi_\theta = \frac{1}{2} (1 + \alpha \bar{\lambda}_\theta + \bar{\lambda}_\theta^2). \quad (18)$$

The non-dimensional slenderness for the temperature θ_a , is given by:

$$\bar{\lambda}_\theta = \bar{\lambda} \left(\frac{k_{y,\theta}}{k_{E,\theta}} \right)^{0.5}, \quad \bar{\lambda} = \frac{KL}{r \lambda_E}; \quad \lambda_E = \pi \sqrt{\frac{E}{f_y}}; \quad r = \sqrt{\frac{I}{A}}, \quad (19)$$

where KL is the buckling length, r radius of gyration, I the section moment of inertia, A is the cross-section area.

$$\alpha = 0.65 \sqrt{\frac{235}{f_y}}. \quad (20)$$

3.5 Beams with Class 3 cross-sections

The design moment resistance $M_{fi,t,Rd}$ at time t of a Class 3 cross-section with a uniform temperature should be determined from:

$$M_{fi,t,Rd} = k_{y,\theta} M_{Rd} [\gamma_{M,1}/\gamma_{M,fi}] \quad (21)$$

where: M_{Rd} is the elastic moment resistance of the gross cross-section $M_{el,Rd}$ for normal temperature design, or the reduced moment resistance allowing for the effects of shear if necessary;

$k_{y,\theta}$ is the reduction factor for the yield strength of steel at the steel temperature θ_a .

The design moment resistance $M_{fi,t,Rd}$ at time t of a Class 3 cross-section with a non-uniform temperature distribution may be determined from:

$$M_{fi,t,Rd} = k_{y,\theta,max} M_{Rd} [\gamma_{M,1}/\gamma_{M,fi}] / \kappa_1 \kappa_2, \quad (22)$$

where:

M_{Rd} is the elastic moment resistance of the gross cross-section $M_{el,Rd}$ for normal temperature design or the reduced moment resistance allowing for the effects of shear if necessary according to EN 1993-1-1 [1];

$k_{y,\theta,max}$ is the reduction factor for the yield strength of steel at the maximum steel temperature $\theta_{a,max}$ reached at time t ;

κ_1 is an adaptation factor for non-uniform temperature in a cross-section;

κ_2 is an adaptation factor for non-uniform temperature along the beam

The design lateral torsional buckling resistance moment $M_{b,fi,t,Rd}$ at time t of a laterally unrestrained beam with a Class 3 cross-section should be determined from:

$$M_{b,fi,t,Rd} = \chi_{LT,fi} W_{el,y} k_{y,\theta,com} f_y / \gamma_{M,fi}. \quad (23)$$

Conservatively $\theta_{a,com}$ can be assumed to be equal to the maximum temperature $\theta_{a,max}$.

The design shear resistance $V_{fi,t,Rd}$ at time t of a Class 3 cross-section should be determined from:

$$V_{fi,t,Rd} = k_{y,\theta,web} V_{Rd} [\gamma_{M,1}/\gamma_{M,fi}], \quad (24)$$

where: V_{Rd} is the shear resistance of the gross cross-section for normal temperature design, according to EN 1993-1-1 [1].

3.6 Members with Class 3 cross-sections, subject to combined bending and axial compression

The design buckling resistance $R_{\beta,t,t}$ at time t of a member subject to combined bending and axial compression should be verified by satisfying expressions for a member with a Class 3 cross-section.

$$\frac{N_{\beta,Ed}}{\chi_{min,\beta} A k_{y,\theta} \frac{f_y}{\gamma_{M,\beta}}} + \frac{k_y M_{y,\beta,Ed}}{W_{el,y} k_{y,\theta} \frac{f_y}{\gamma_{M,\beta}}} + \frac{k_z M_{z,\beta,Ed}}{W_{el,z} k_{y,\theta} \frac{f_y}{\gamma_{M,\beta}}} \leq 1, \quad (25a)$$

$$\frac{N_{\beta,Ed}}{\chi_{z,\beta} A k_{y,\theta} \frac{f_y}{\gamma_{M,\beta}}} + \frac{k_{LT} M_{y,\beta,Ed}}{\chi_{LT,\beta} W_{el,y} k_{y,\theta} \frac{f_y}{\gamma_{M,\beta}}} + \frac{k_z M_{z,\beta,Ed}}{W_{el,z} k_{y,\theta} \frac{f_y}{\gamma_{M,\beta}}} \leq 1, \quad (25b)$$

where:

$$\chi_{LT,\beta} = \frac{1}{\Phi_{LT,\theta,com} + \sqrt{(\Phi_{LT,\theta,com})^2 - (\bar{\lambda}_{LT,\theta,com})^2}}, \quad (26)$$

$$\Phi_{LT,\theta,com} = \frac{1}{2} \left[1 + \alpha \bar{\lambda}_{LT,\theta,com} + (\bar{\lambda}_{LT,\theta,com})^2 \right], \quad (27)$$

$$\alpha = 0.65 \sqrt{235 / f_y}, \quad (28)$$

$$\bar{\lambda}_{LT,\theta,com} = \bar{\lambda}_{LT} \sqrt{k_{y,\theta,com} / k_{E,\theta,com}}, \quad (29)$$

where:

$k_{E,\theta,com}$ is the reduction factor from Section 4.2 for the slope of the linear elastic range at the maximum steel temperature in the compression flange $\theta_{a,com}$ reached at time t .

$$k_{LT} = 1 - \frac{\mu_{LT} N_{\beta,Ed}}{\chi_{z,\beta} A k_{y,\theta} \frac{f_y}{\gamma_{M,\beta}}} \leq 1, \quad (30)$$

where

$$\mu_{LT} = 0.15 \bar{\lambda}_{z,\theta} \beta_{M,LT} - 0.15 \leq 0.9, \quad (31)$$

$$k_y = 1 - \frac{\mu_y N_{\beta,Ed}}{\chi_{y,\beta} A k_{y,\theta} \frac{f_y}{\gamma_{M,\beta}}} \leq 3, \quad (32)$$

where

$$\mu_y = (1.2 \beta_{M,y} - 3) \bar{\lambda}_{y,\theta} + 0.44 \beta_{M,y} - 0.29 \leq 0.8, \quad (33)$$

$$k_z = 1 - \frac{\mu_z N_{\beta,Ed}}{\chi_{z,\beta} A k_{y,\theta} \frac{f_y}{\gamma_{M,\beta}}} \leq 3, \quad (34)$$

where

$$\mu_z = (2 \beta_{M,z} - 5) \bar{\lambda}_{z,\theta} + 0.44 \beta_{M,z} - 0.29 \leq 0.8. \quad (35)$$

4 STEEL TEMPERATURE DEVELOPMENT

4.1 Unprotected internal steelwork

For an equivalent uniform temperature distribution in the cross-section, the increase of temperature $\Delta \theta_{a,t}$ in an unprotected steel member during a time interval Δt should be determined from:

$$\Delta\theta_{a,t} = k_{sh} \frac{A_m/V}{c_a \rho_a} \dot{h}_{net,d} \Delta t, \quad (36)$$

where: k_{sh} is correction factor for the shadow effect, from 5.2.5.1(2) in EC 3;

A_m/V is the section factor for unprotected steel members;

A_m is the surface area of the member per unit length [m²];

V is the volume of the member per unit length [m³];

c_a is the specific heat of steel, from section 4.2.4 [J/kgK];

$\dot{h}_{net,d}$ is the design value of the net heat flux per unit area [W/m²];

Δt is the time interval [seconds];

ρ_a is the unit mass of steel [kg/m³].

For I-sections under nominal fire actions, the correction factor for the shadow effect may be determined from:

$$k_{sh} = 0.9 \frac{[A_m/V]_b}{[A_m/V]}, \quad (37)$$

where: $[A_m/V]_b$ is box value of the section factor. In all other cases, the value of k_{sh} shall be taken as:

$$k_{sh} = \frac{[A_m/V]_b}{[A_m/V]}. \quad (38)$$

For cross sections with a convex shape (e.g. rectangular or circular hollow sections) fully embedded in fire, the shadow effect does not play a role and consequently the correction factor k_{sh} equals unity. Ignoring the shadow effect (i.e. $k_{sh} = 1$), leads to conservative solutions.

The value of Δt should not be taken as more than 5 seconds. The value of the section factor A_m/V should not be taken as less than 10 m⁻¹.

4.3 The calculation of the evolution of steel temperature

For unprotected steel structure the calculation of the evolution of the steel temperature is as follows with an iteration process (EC3₂ [2], ISO 1975 [3]):

The time at the beginning of the fire is

$$t_i = 0, \text{ and every time period: } \Delta t_i = 5 \text{ we calculate it } t_{i+1} = t_i + \Delta t_i \text{ [sec]}. \quad (41)$$

$$\text{Changing the time from } 0 \leq t_i \leq t_{max} \text{ [sec]}, \quad (42)$$

where t_{max} can be 1/2, 1, 1 1/2, 2, 4 hours, means 1800, 3600, 5400, 7200, 14400 [sec].

The temperature of the steel can be between

$$20 \text{ [}^\circ\text{C]} \leq \theta_a \leq 1200 \text{ [}^\circ\text{C]}. \quad (43)$$

The starting values are as follows:

$$\theta_a = 20 \text{ }^\circ\text{C}, \Delta\theta_a \text{ [}^\circ\text{C]}, \rho_a = 7850 \text{ kg/m}^3. \quad (44)$$

The gas temperature in the vicinity of the fire exposed member (standard temperature-time curve):

$$\theta_g = 20 + 345 \log \left(8 \frac{t_i}{60} + 1 \right) \text{ [}^\circ\text{C]}. \quad (45)$$

The net convection heat flux:

$$\dot{h}_{netc} = \alpha_c (\theta_g - \theta_a), \quad (46)$$

where the coefficient of heat transfer by convection $\alpha_c = 25 \text{ W/m}^2\text{K}$. (47)

The net radiative heat flux

$$\dot{h}_{netr} = \phi \varepsilon_m \varepsilon_f \sigma [(\theta_g + 273)^4 - (\theta_a + 273)^4] \text{ [W/m}^2\text{]}, \quad (48)$$

where:

the configuration factor $\phi = 1$,

the surface emissivity of the member $\varepsilon_m = 0.8$,

the emissivity of the fire $\varepsilon_f = 1.0$,

the Stephan Boltzmann constant $\sigma = 5.67 \times 10^{-8} \text{ W/m}^2\text{K}^4$. (49)

The total net heat flux can be calculated as the sum of convection and radiative heat fluxes:

$$\dot{h}_{netd} = \dot{h}_{netc} + \dot{h}_{netr}. \quad (50)$$

For a tube exposed to fire on all sides:

$$\frac{A_m}{V} = \frac{1}{10^{-3} t_0}, \text{ where } t_0 \text{ is the cross section thickness.} \quad (51)$$

$$\text{The temperature changing: } \Delta\theta_a = k_{sh} \frac{\frac{A_m}{V} \dot{h}_{netd} \Delta t_i}{c_a \rho_a}, \quad (52)$$

where $k_{sh} = 1$. (53)

The surface temperature of the steel member in every iteration step is the following:

$$\theta_a^n = \theta_a^{n-1} + \Delta\theta_a^{n-1} \quad (54)$$

The iteration is stopped, when either the time, or the temperature limit is reached.

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