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Fire Design of Concrete Structures According to the Eurocodes: A Review

by L.R. Taerwe

Synopsis: Whereas traditionally the verification of fire safety is based on prescriptive measures and criteria, an evolution toward performance-based design can be noticed, which is reflected in the design approaches given in the fire parts of Eurocodes 1 and 2. In Part 1-2 of Eurocode 11, general design aspects of structures exposed to fire are given as well as specific load combinations, design values of thermal and mechanical material properties, fire models, and heat transfer models. Most of these design principles are applicable to all types of construction materials. In Part 1-2 of Eurocode 22, specific approaches related to concrete structures are given, i.e., models giving the influence of high temperatures on material characteristics, a method based on tabulated values, simplified verification methods, and the basic principles of advanced calculation methods. In this paper, a review is presented of the most relevant clauses of the mentioned documents. For practical applications, the complete documents should be consulted.

Keywords: Eurocodes; fire resistance; simplified verification methods; tabulated values

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INTRODUCTION

The Structural Eurocode program comprises the following standards, which generally consist of different parts:

Eurocode : Basis of structural design (EN 1990)

Eurocode 1 : Actions on structures (EN 1991)

Eurocode 2 : Design of concrete structures (EN 1992)

Eurocode 3 : Design of steel structures (EN 1993)

Eurocode 4 : Design of composite steel and concrete structures (EN 1994)

Eurocode 5 : Design of timber structures (EN 1995)

Eurocode 6 : Design of masonry structures (EN 1996)

Eurocode 7 : Geotechnical design (EN 1997)

Eurocode 8 : Design of structures for earthquake resistance (EN 1998)

Eurocode 9 : Design of aluminium structures (EN 1999)

Before the introduction of the Eurocode system, no systematic and unified approach to fire safety design of structures was available in most countries. With this set of codes the same design principles can be applied to different building materials.

Each country can determine numerical values of a specified series of parameters (NDP : Nationally Determined Parameters) which can be found in its National Annex. The introduction of this procedure was necessary to cope with the specific backgrounds and experiences of different countries and yet to arrive at unified design formats and formulae. However, the aim was to keep the number of NDP's as small as possible.

Part 1-2 of Eurocode 1 (EN 1991-1-2) is entitled "General actions – Actions on structures exposed to fires" and Part 1-2 of Eurocode 2 (EN 1992-1-2) is entitled "General rules – structural fire design". This latter part deals with the design of concrete structures for the accidental situation of fire exposure and only covers passive methods of fire protection, thus not including active measures such as sprinkler systems. Whereas traditionally fire safety is based on prescriptive measures and criteria, an evolution towards performance based design can be noticed, which is based on realistic fire scenarios and a probabilistic risk analysis. This latter analysis is based on the assessment of the probability of fire ignition and the probability of success of safety measures including manual extinguishing, sprinklers, alarm systems, intervention of fire brigades etc. The mentioned Eurocodes include basic principles and data which constitute the basis for a performance based design.

RESEARCH SIGNIFICANCE

The system of Eurocodes offers a unified approach for fire safety design of structures built with different materials. They include basic design principles for the accidental fire situation, fire models with different degrees of sophistication and models for thermal an mechanical material properties which constitute the basis for a performance based design. The paper presents a systematic synthesis of the major procedures and data for fire safety design of concrete structures and some explanation and background information of the most current approaches.

GENERAL ASPECTS

Performance criteria

Three basic performance criteria are defined:

- Criterion "R" is assumed to be satisfied when the load bearing function is maintained during the required time of fire resistance.
- Criterion "I" may be assumed to be satisfied when the average temperature rise over the whole of the non-exposed surface is limited to 140 K, and the maximum temperature rise at any point of that surface does not exceed 180 K. In this way, ignition of combustable materials at that side of a compartment wall which is not exposed to the fire, may be avoided.
- Criterion "E" requires that no cracks, holes or openings occur through which flames or hot gasses may escape from the compartment under fire, to an adjacent compartment where no fire occurs.

For the standard fire exposure, members shall comply with criteria R. E and I as follows:

- load bearing function only: mechanical resistance (criterion R)
- separating function only: integrity (criterion E) and, when requested, insulation (criterion I)

- separating and load bearing function: criteria R, E and, when requested, criterion I.

As such, the notations R30, R60, E30, E60 and I30, I60 mean that an element complies with criteria R, E and I respectively during at least 30 or 60 minutes for exposure to a standard fire. The notation REI90 indicates that an element complies at least during 90 minutes with the three criteria simultaneously, whereby the most critical criteria is decisive for the classification.

The mentioned criteria are verified within a structural fire design analysis which encompasses the following steps:

- Selection of the relevant design fire scenarios on the basis of a fire risk assessment
- Determination of the corresponding design fire, which should be applied only to one fire compartment of the building at the same time
- Calculation of the temperature evolution within the structural members. For external members, fire exposure through openings in facades and roofs should be considered.
- Calculation of the mechanical behaviour of the structure exposed to fire.

Fire models

The evolution of a fire is determined by several parameters among which:

- the geometrical characteristics of the compartment
- the thermal properties of the compartment walls
- the position and size of openings in the walls which influence the ventilation conditions
- the fire load density which depends on the occupancy and the net calorific value of the combustible materials.

Depending on the experience and judgment of the designer and the applicable fire regulations, a choice is made between:

a) nominal temperature–time curves

b) natural fire models

b.1) simplified fire models: compartment fires and localised fires; data for design fire loads and gas temperature models based on ventilation characteristics, can found in the annexes of Part 1-2 of Eurocode 1

b.2) advanced fire models: one-zone models, two-zone models and Computational Fluid Dynamic models (CFD)

The nominal temperature–time curves give a relation between the gas temperature θ_g in the compartment and the time t, expressed in minutes. No parameters related to the compartment characteristics are considered. The following models are given in EN 1991-1-2:

- Standard temperature-time curve (ISO834)

$$\theta_g = 20 + 345 \log_{10} (8t + 1) \qquad [°C] \tag{1}$$

- External fire curve

$$\theta_g = 20 + 600 \left(1 - 0{,}687\, e^{-0{,}32t} - 0{,}313\, e^{-3{,}8t}\right) \qquad [°C] \tag{2}$$

- Hydrocarbon curve

$$\theta_g = 20 + 1080 \left(1 - 0{,}325\, e^{-0{,}167t} - 0{,}675\, e^{-2{,}5t}\right) \qquad [°C] \tag{3}$$

The corresponding curves are shown in Fig. 1.

MECHANICAL ACTIONS

Representative values for a live load Q are (EN 1990) :

- Q_k = the characteristic value
- $\psi_o Q_k$ = the combination value
- $\psi_1 Q_k$ = the frequent value
- $\psi_2 Q_k$ = the quasi-permanent value

In the order given, the probability of exceedance of these values increases. Typical values of ψ_o, ψ_1 and ψ_2 are given in Table 1.

For normal temperature design (persistent design situation), the design value E_d of an action effect is calculated from the basic load combination which, in general and including prestressing, can be written as

$$\sum_{j \geq 1} \gamma_{Gj}\, G_{kj} + \gamma_P\, P_k + \gamma_{Q1}\, Q_{k1} + \sum_{i > 1} \psi_{0i}\, \gamma_{Qi}\, Q_{ki} \tag{4}$$

where "+" stands for "combined effect of " and

G_{kj} = characteristic value of permanent action j

P_k = characteristic value of the prestressing force

Q_{k1} = characteristic value of the leading variable action (designated by 1)

Q_{ki} = characteristic value of other independent variable actions (i > 1)

γ_G, γ_Q , γ_P = partial safety factors; typically $\gamma_G = 1.35$; $\gamma_Q = 1.50$ and $\gamma_P = 1.0$ or 1.3

For an accidental design situation such as a fire, the following load combination is considered:

$$\sum_{j\geq 1}\gamma_{GAj}\, G_{kj} + \gamma_{PA}\, P_k + \gamma_A\, A_k + \psi_{x,1}\, Q_{k1} + \sum_{i>1}\psi_{2,i}\, Q_{ki} \tag{5}$$

where $\gamma_A\, A_k$ is the design value of the accidental action and $\psi_{x,1}$ is equal to $\psi_{1,1}$ or $\psi_{2,1}$ depending on the national choice of a country. For accidental design situations $\gamma_A = \gamma_{GA} = \gamma_{PA} = 1$ and Eq. (5) becomes

$$\sum_{j\geq 1} G_{kj} + P_k + A_k + \psi_{x,1}\, Q_{k1} + \sum_{i>1}\psi_{2,i}\, Q_{ki} \tag{6}$$

In the case of a fire, A_k represents the indirect actions due to the fire, caused by internal or external restraint of deformations. The action effect resulting from Eq. (6) is denoted as $E_{fi,d}(t)$.

According to EN 1991-1-2, indirect actions from adjacent members need not be considered when fire safety requirements refer to members under standard fire conditions. In this case $E_{fi,d}$ can be considered as being time independent. From comparison of Eqs. (4) and (6) it follows that, in general, the ratio

$$\eta_{fi} = \frac{E_{fi,d}}{E_d} \tag{7}$$

is substantially smaller than 1. For member analysis according to EN 1992-1-2, the following simplification may be applied:

$$E_{fi,d} = \eta_{fi}\, E_d \tag{8}$$

with

$$\eta_{fi} = \frac{G_k + \psi_{x,1}\, Q_{k1}}{\gamma_G\, G_k + \gamma_{Q1}\, Q_{k1}} = \frac{1 + \Psi_{x,1}\,\zeta}{1.35 + 1.50\,\zeta} \tag{9}$$

where $\zeta = Q_{k1}/G_k$ and E_d is obtained from normal temperature design. Fig. 2 shows η_{fi} as a function of ζ and $\psi_{1,1}$. As a simplification a recommended value of $\eta_{fi} = 0.7$ may be used. Application of Eq. (8) is subjected to several limitations, which are not discussed here.

MATERIAL PROPERTIES

Basic aspects

In the accidental fire situation, design values of mechanical material properties are defined as:

$$X_{d,fi} = \frac{X_k\ (\theta)}{\gamma_{M,fi}} = \frac{k_\theta\ X_k}{\gamma_{M,fi}} \tag{10}$$

where

- X_k is the characteristic value of the property considered for normal temperature design
- X_k (θ) is the characteristic value of the property considered for a temperature θ
- k_θ is a reduction factor equal to X_k (θ)/X_k
- $\gamma_{M,fi}$ is the partial safety factor for the property considered; for accidental design situations the recommended value is equal to 1.

The values of X_k (θ) = k_θ X_k are used to calculate $R_{fi,d}$ (t). Values of k_θ for the basic mechanical properties of concrete, reinforcing steel and prestressing steel can be found in EN 1992-1-2. In Figs. 3 and 4 values of k_c (θ) = $f_{ck}(\theta)/f_{ck}$ and k_s (θ) = f_{yk} (θ)/f_{yk} are given. The curves shown in Fig. 3 are valid for $f_{ck} \leq 50$ MPa. For high strength concrete (HSC) specific models, resulting in a steeper descent, are given Models for the influence of high temperatures on the stress-strain curves and the basic physical and thermal properties of concrete and reinforcing steel are also given.

Explosive spalling

Explosive spalling of concrete shall be avoided or its influence on the performance requirements shall be taken into account (e.g. reduced cross section). According to EN1992-1-2, explosive spalling is unlikely to occur when the moisture content is less than 3% by weight (recommended value for NDP). Although a limitation of the Relative Humidity is commonly used in some countries, it was found appropriate to put a limit on the moisture content. Above 3% accurate assessment of moisture content, type of aggregate, permeability of concrete and heating rate should be considered. For HSC with 55 MPa $\leq f_{ck} \leq$ 80 MPa, the previous rules are still applicable provided that the maximum content of silica fume is less than 6% by weight of cement. In case of higher silica fume contents and for HSC with 80 MPa $\leq f_{ck} \leq$ 90 MPa, specific measures should be taken to avoid spalling. A possibility is to include in the concrete mix at least 2 kg/m³ of monofilament polypropylene fibres. This limit was chosen based on a wide range of available test results.

MECHANICAL ANALYSIS

Basic aspects

The basis of the mechanical analysis is shown schematically in Figs. 5a and 5b, where the following notations are used:

E_d = design value of an action effect at room temperature e.g. M_{Ed}, N_{Ed}, V_{Ed}, …

R_d = design value of the resistance of a structural member at room temperature e.g. M_{Rd}, N_{Rd}, V_{Rd}, …

$E_{fi,d}$ (t) = design value of an action effect in the fire situation at time t, including the effect of indirect actions

$R_{fi,d}$ (t) = design value of the relevant resistance of a structural member in the fire situation at time t

$t_{fi,req}$ = required fire resistance time give as a multiple of 30 min

$t_{fi,d}$ = design value of the fire resistance, which is determined by $R_{fi,d}$ (t) = $E_{fi,d}$ (t)

The general case of varying $E_{fi,d}(t)$ is shown in Fig. 5a. When indirect actions due to restrained deformations are neglected, $E_{fi,d}$ can be considered as time-independent as assumed in Fig. 5b. $R_{fi,d}$ (t) decreases in time, due to material degradation under increasing temperatures of the cross-section considered. $R_{fi,d}(0)$ is higher than R_d due to the fact that in the accidental fire situation $\gamma_{M,fi}$ = 1 whereas for the persistent design situation γ_c = 1.5 and γ_s = 1.15. Generally $R_d > E_d$ due to the fact that for practical reasons more reinforcement is placed than strictly necessary from the design calculations, or $A_{s,prov} > A_{s,req}$.

The general verification format for each cross-section can be written as:

$$t_{fi,d} \geq t_{fi,req} \tag{11}$$

or

$$R_{fi,d}\ (t) \geq E_{fi,d}\ (t) \qquad \text{for} \qquad t \leq t_{fi,req} \tag{12}$$

As the action effects are obtained with reduced partial safety factors and also the material factors are reduced, the margin between the design values $R_{fi,d}(0)$ and $E_{fi,d}(0)$ at the onset of fire is higher than the margin between R_d an E_d for ambient temperature design. This makes it possible to satisfy the basic design equation (12) during at least $t_{fi,req}$ despite the fact that

$R_{fi,d}$ (t) is a monotonic decreasing function of the exposure time.

The reduction of the partial safety factors in the accidental fire situation is related to the fact that, although the conditional probability of failure in case of fire is relatively high, the occurrence probability of a fire during the design service life is quite low. Hence, the product of both probabilities should be comparable to the target failure probability specified for the persistent design situation.

Design procedures

In order to satisfy Eq. (12) the following design methods are permitted:

- detailing according to recognised design solutions e.g. tabulated data
- simplified calculation methods for specific types of members
- advanced calculation methods for simulating the behaviour of structural members, parts of the structure or the entire structure.

For beams and slabs, neglecting the effect of axial restraint generally leads to conservative results[3]. For frames, the axial compression forces in beams and slabs, resulting from longitudinal restraint, may result in shear failures in the columns, even if they are in bays not exposed to the fire.

DESIGN PROCEDURE BASED ON TABULATED DATA

Field of application

The method is applicable for the verification of separate members and gives recognized design solutions for standard fire exposures up to 240 minutes. In the tables, minimum values are given for the cross-sectional dimensions and the axis distance of the main reinforcement. Additional detailing rules are specified for each type of structural member. An advantage of the tabulated values is that a designer can quickly verify whether the dimensions which follow from a normal temperature design, are acceptable under fire conditions. The following aspects should be considered:

- The given values are based on the standard fire exposure according to ISO 834
- The tables have been developed on an empirical basis and confirmed by experience and theoretical evaluations of tests. The values have been derived under rather conservative assumptions.
- The values apply to normal weight concrete made with siliceous aggregates. If calcareous or lightweight aggregates are used in beams or slabs, the minimum dimensions of the cross-section may be reduced by 10 %
- When using tabulated values, no further checks are required concerning explosive spalling, shear and torsion capacity and anchorage details.

General rules

1. The minimum requirements concerning section sizes and axis distance of the steel have been determined in order to satisfy criterion R (load bearing function) during the specified standard fire resistance according to Eq. (12). The tabulated data are based on a reference load level corresponding to $\eta_{fi} = 0.7$ with η_{fi} determined in accordance with Eq. (7), unless otherwise specified.
2. In the tables, minimum concrete cover is expressed as the distance "a" from the axis of the main reinforcement to the closest concrete surface (see Fig. 6). The stated axis distances are nominal values. Allowance for tolerance need not be added. In part 1-1 of Eurocode 2, valid for normal temperature design, the concrete cover "c" is defined as the distance from the edge of a reinforcing bar to the closest concrete surface. Hence, for a longitudinal rebar (main reinforcement) with diameter ϕ_{bar}, the relation between "a" and "c" typically can be written as $a = c + \phi_{stirrup} + \phi_{bar}/2$ with $\phi_{stirrup}$ the stirrup diameter.
3. The minimum axis distance of reinforcement located in tensile zones of simply supported beams and slabs, was calculated on the basis of a critical steel temperature $\theta_{cr} = 500$ °C. The critical temperature is that temperature for which the steel yields under the steel stress $\sigma_{s,fi}$ occurring in the fire situation. For prestressing tendons, the critical temperature for bars is assumed to be 400 °C and for strands and wires to be 350 °C.

Columns

For assessing the fire resistance of columns, two methods (A and B), are provided, which are applicable within certain limits of the influencing parameters. As an example, Table 2 gives values of "b_{min}"and "a" for columns with rectangular or circular cross-section according to method A. In this method, the load level in the fire situation is characterized by the parameter

$$\mu_{fi} = \frac{N_{Ed,fi}}{N_{Rd}} = \frac{\eta_{fi}\ N_{Ed}}{N_{Rd}} \tag{13}$$

Where $N_{Ed,fi}$ is the design value of the applied axial load which follows from Eq. (6) and $N_{Rd} = A_s\ f_{yd} + A_c\ f_{cd}$ is the design value of the resisting axial force under normal temperature conditions. Fig. 5b shows the relative positions of $R_d = N_{Rd}$, $E_d = N_{Ed}$ and $E_{fi,d} = N_{Ed,fi}$.

In method A, also a formula is given which allows to calculate the standard fire resistance (in minutes) for other parameter values than those mentioned in Table 2.

Other members

In EN 1992-1-2, similar tables are given for load-bearing and non load-bearing walls, tensile members, simply supported and continuous beams and solid slabs, flat and ribbed slabs. By way of example, table 3 is given which is valid for simply supported reinforced or prestressed slabs.

In continuous beams and slabs, thermal moments M_θ develop during heating due to restraint of the thermal rotations. This phenomenon is shown schematically in Fig. 7 for a beam clamped at both ends. Mainly during the first 30 minutes, the moment line shifts towards more negative values (line 1). In some cases, the zone with positive moments can disappear completely (line 2). If no upper reinforcement is provided in that zone, a hinge can occur in the middle section. For this reason it is required that in continuous beams and slabs at least 25 % of the upper reinforcement is placed at the intermediate supports, continuous over a distance of at least 0.3 l_{eff} towards the adjacent spans.

SIMPLIFIED CALCULATION METHODS

General

Simplified calculation methods may be used to determine the ultimate load-bearing capacity of a heated cross section and to compare the capacity with the action effect resulting from Eq. (6). In 1992-1-2 the following methods are mentioned :

- Informative annex B provides two alternative methods, B.1 "500°C isotherm method" and B.2 "Zone method" for calculating the resistance to bending moments and axial forces. Second order effects may be included with both methods.
- Informative annex C provides a zone method for analyzing column sections with significant second order effects.
- Informative annex D provides a simplified calculation method for shear, torsion and anchorage.
- Informative annex E provides a simplified calculation method for the design of beams and slabs.

Temperature profiles

The basis of all simplified methods are the temperature profiles in the considered cross section. In Annex A, several temperature profiles are given for typical cross-sections with siliceous aggregates for 30, 60, …, 240 minutes exposure to a standard fire. The temperature profiles are conservative for most other aggregates. By way of example, fig. 8 shows the temperature profiles in a slab (thickness 200 mm) for one-sided exposure.

500°C isotherm method

The method is based on the hypothesis that concrete at a temperature above 500°C is neglected ($k_c = 0$) in the calculation of the load-bearing capacity, while concrete at a temperature below 500°C is assumed to retain its full strength ($k_c = 1$). The characteristics of the reinforcing and prestressing steel are determined according to the local temperature. The method is applicable to reinforced and prestressed concrete sections with respect to axial load, bending moment and their combination.

The following steps are followed (fig. 9) :

- determine the 500°C isotherm by mean of the information provided in annex A for the specified fire exposure.
- determine the reduced width b_{fi} and the reduced effective depth d_{fi} of the cross-section by excluding the concrete outside the 500°C isotherm. The rounded shape of the isotherms is approximated by rectangles.
- determine the temperature of the reinforcing bars in the tension and compression zones by means of the available temperature profiles. Also the bars outside the reduced cross section are considered.
- calculate the reduced value $f_{yk}(\theta)$ for each bar from the $k_s(\theta)$-temperature curves.

- calculate the resisting moment $M_{Rd,fi}$ according to the methods for normal temperature design, taking into account the reduced cross section and the modified material properties.
- verify whether $M_{Rd,fi} \geq M_{Ed,fi}$ for the exposure time considered.

Continuous beams and slabs

The verification is based on plastic limit analysis, assuming that sufficient rotation capacity is available at the intermediate supports. The following condition has to be verified (fig. 10) :

$$\frac{|M_{Rd1,fi}| + |M_{Rd2,fi}|}{2} + M_{Rd,fi,span} \geq M_{Ed,fi} \tag{14}$$

with

$$M_{Ed,fi} = \frac{w_{Ed,fi}\ \ell^2}{8} \tag{15}$$

where $w_{Ed,fi}$ follows from Eq. (6). Eq. (14) expresses that the statically determined parabolic bending moment line "1" is located within the values $M_{Rd1,fi}$, $M_{Rd2,fi}$ and $M_{Rd,fi,span}$.

CONCLUSIONS

- Common rules regarding general verification formats and load combinations for the accidental fire situation are given in Part 1-2 of Eurocode 1 (EN 1991-1-2). Thus a unified approach is available with the advantage of being independent of the construction material used.
- Models giving the influence of high temperatures on concrete properties are given in Part 1-2 of Eurocode 2 (EN 1992-1-2) which offer a valuable basis for detailed calculations.
- The tabulated values for minimum cross-section size and axis distance of the main reinforcement given in EN 1992-1-2, allow a fast verification of the cross-section for the required fire resistance time. The values are based on a synthesis of experimental data, numerical simulations and field experience.
- Simplified design methods (e.g. the 500°C isotherm method), which are an extension of those valid for normal temperature design, are given in EN 1992-1-2. Thus a fast verification of the fire resistance of concrete structures can be performed.

REFERENCES

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2. EN 1992-1-2 : Eurocode 2 : Design of concrete structures – Part 1-2 : General Rules – Structural Fire design, CEN, December 2004.

3. Riva P., Nonlinear and plastic analysis of reinforced concrete beams, Proceedings of the Workshop "Fire Design of Concrete Structures: What now? What next?", Starrylink Editrice, 2005, Brescia, Italy.

Table 1–Values of ψ_0, ψ_1, and ψ_2 according to EN 1990

Occupancy	ψ_0	ψ_1	ψ_2
Residential	0.7	0.5	0.3
Office	0.7	0.5	0.3
Storage	1.0	0.9	0.8

Table 2– Combinations b_{min}/a for columns with rectangular or circular cross-section (dimensions in mm)

Standard fire resistance	Fire exposure on more than one side			One-sided exposure
	$\mu_{fi} = 0.2$	$\mu_{fi} = 0.5$	$\mu_{fi} = 0.7$	$\mu_{fi} = 0.7$
R 30	200/25	200/25	200/32 300/27	155/25
R 60	200/25	200/36 300/31	250/46 350/40	155/25
R 90	200/31 300/25	300/45 400/38	350/53 450/40	155/25
R 120	250/40 350/35	350/45 450/40	350/57 450/51	175/35
R 180	350/45	350/63	450/70	230/55
R 240	350/61	450/75	-	295/70

Table 3 – Minimum thickness and axis distance for reinforced and prestressed simply supported one-way and two-way solid concrete slabs

Standard fire resistance	Slab thickness h_s (mm)	Axis distance (mm)		
		One-way	Two-way	
			$\ell_y/\ell_x \leq 1.5$	$1,5 < \ell_y/\ell_x \leq 2$
REI 30	60	10	10	10
REI 60	80	20	10	15
REI 90	100	30	15	20
REI 120	120	40	20	25
REI 180	150	55	30	40
REI 240	175	65	40	50

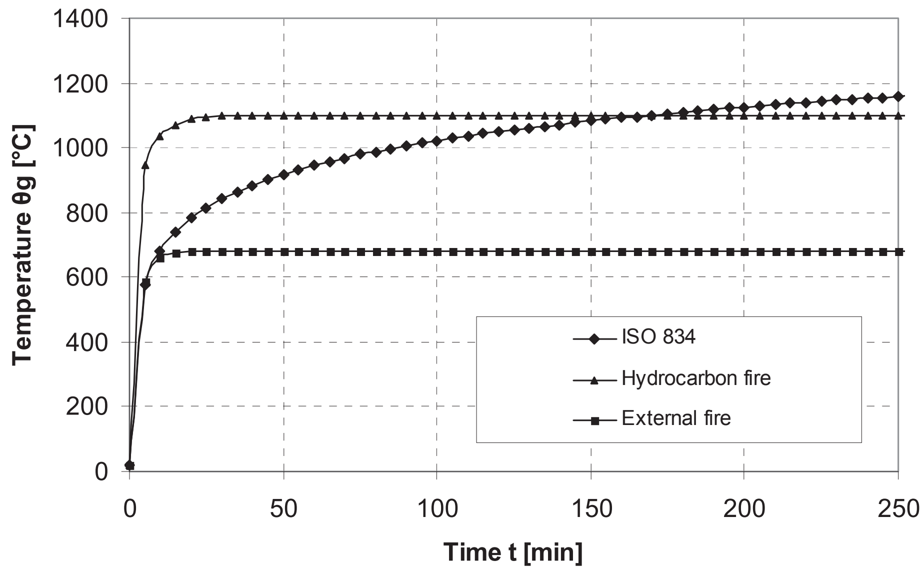


Fig. 1 – Standardized temperature-time curves

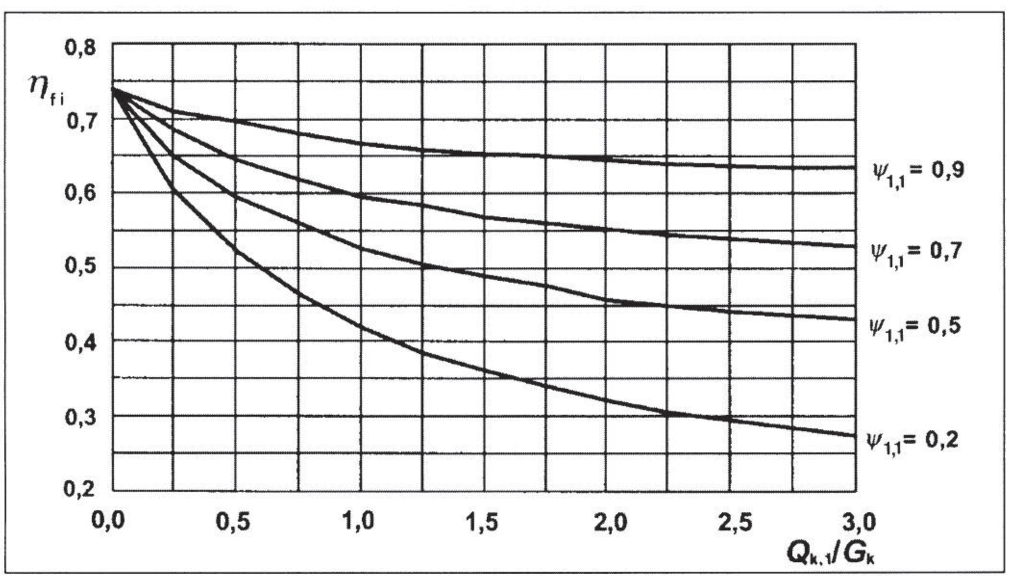


Fig. 2 - Variation of the reduction factor $\zeta = Q_{k1}/G_k$

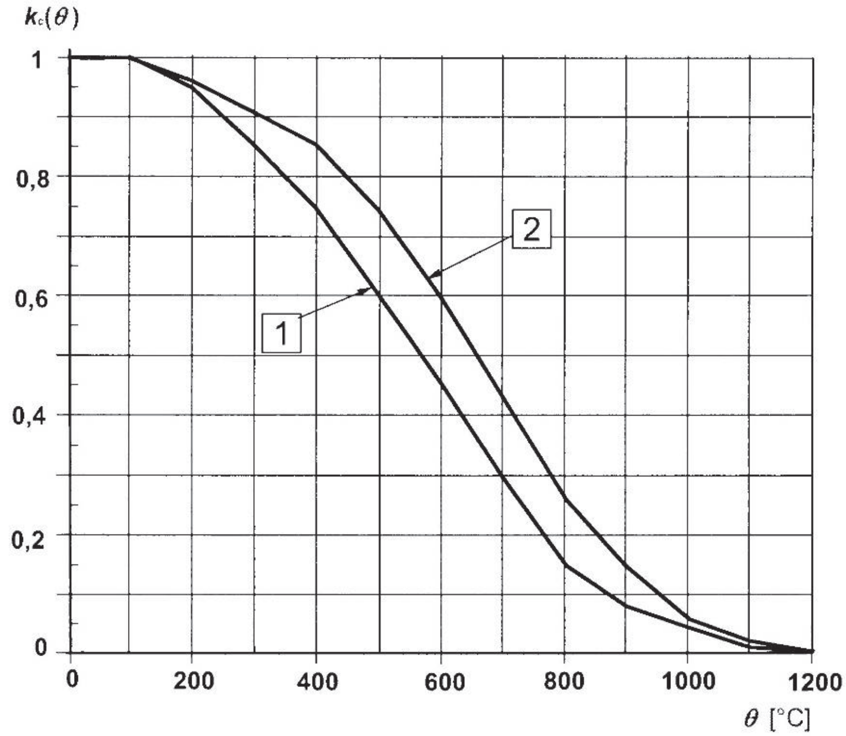


Fig. 3 - Variation of the Coefficient $k_c(\theta)$ for normalweight concrete with siliceous aggregates (curve 1) and calcareous aggregates (curve 2)

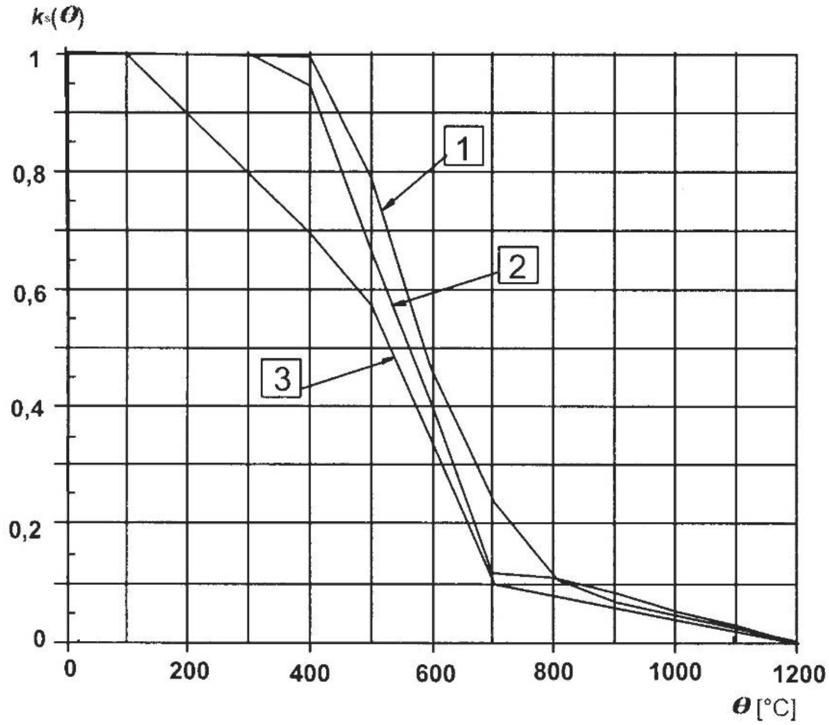
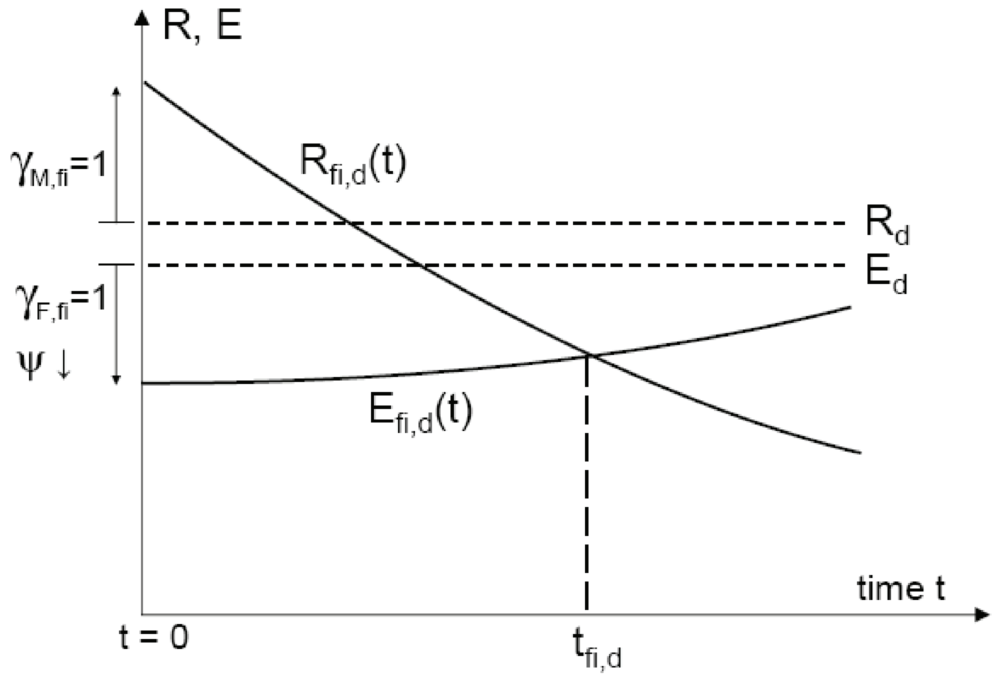


Fig. 4 - Variation of the coefficient $k_s(\theta)$

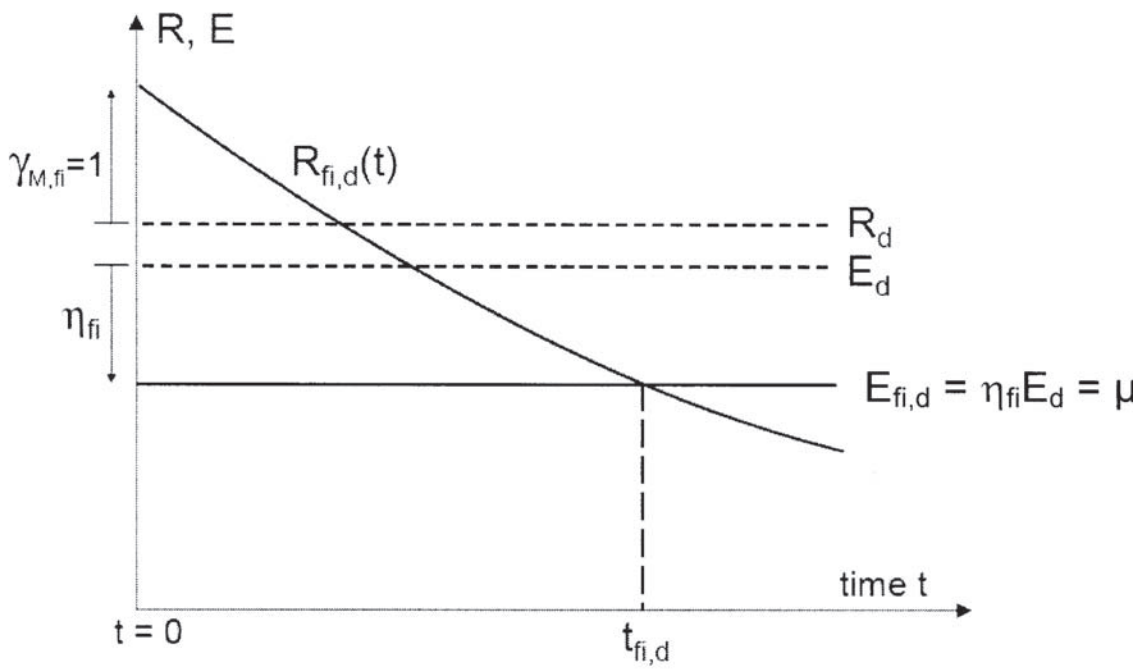
Curve 1 : Tension reinforcement (hot rolled) for strains $\varepsilon_{s,fi} \geq 2$ %

Curve 2 : Tension reinforcement (cold worked) for strains $\varepsilon_{s,fi} \geq 2$ %

Curve 3 : Compression reinforcement and tension reinforcement for strains $\varepsilon_{s,fi} < 2$ %



(a)



(b)

Fig. 5 – Design values of action effects and corresponding resistances

(a) including indirect actions and (b) neglecting indirect actions

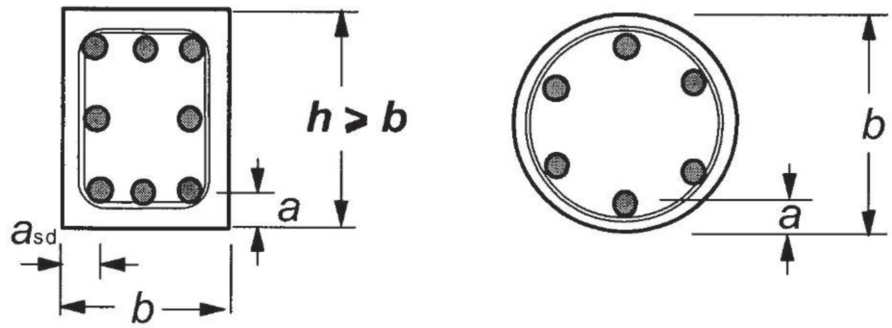


Fig. 6 - Nominal axis distance and other typical cross-sectional dimensions

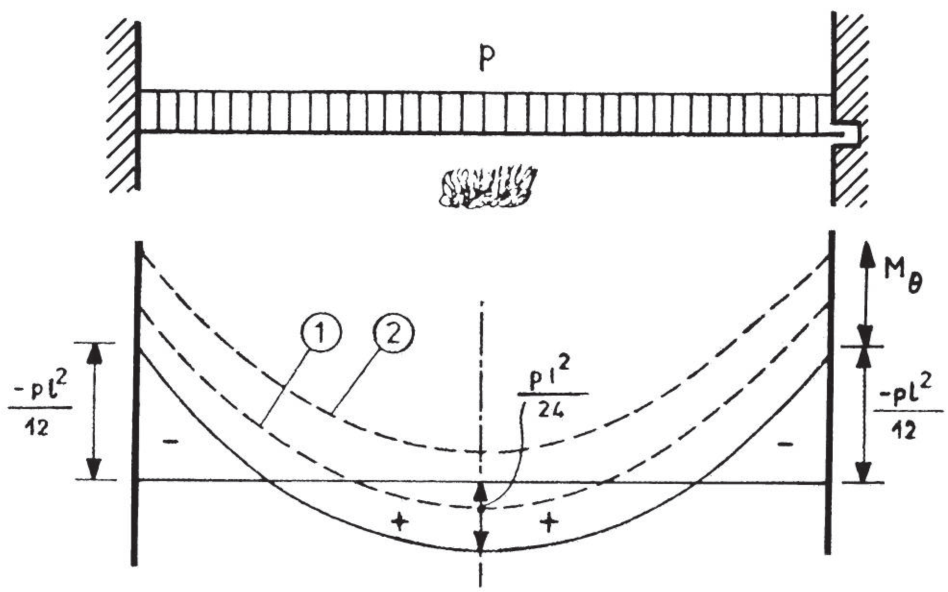


Fig. 7 - Shift of the bending moment line in a clamped beam during fire exposure

at the bottom side

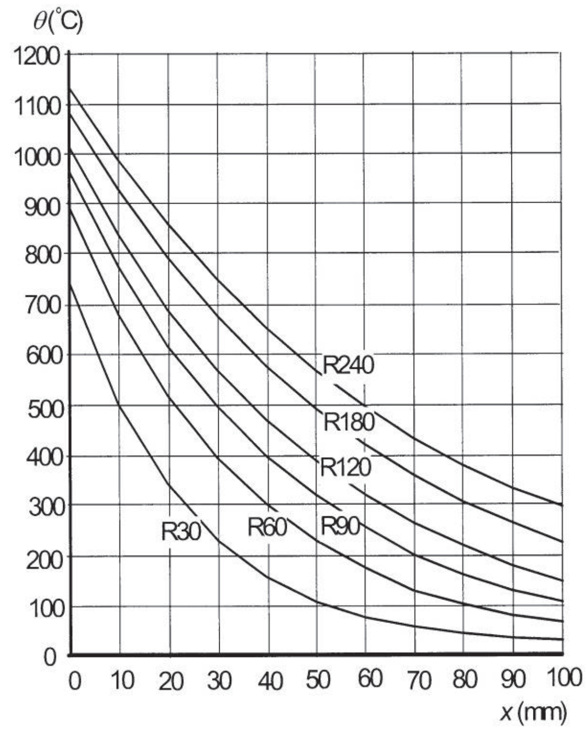


Fig. 8 - Temperature profiles for slabs (thickness 200 mm) for R30, R60, …, R240

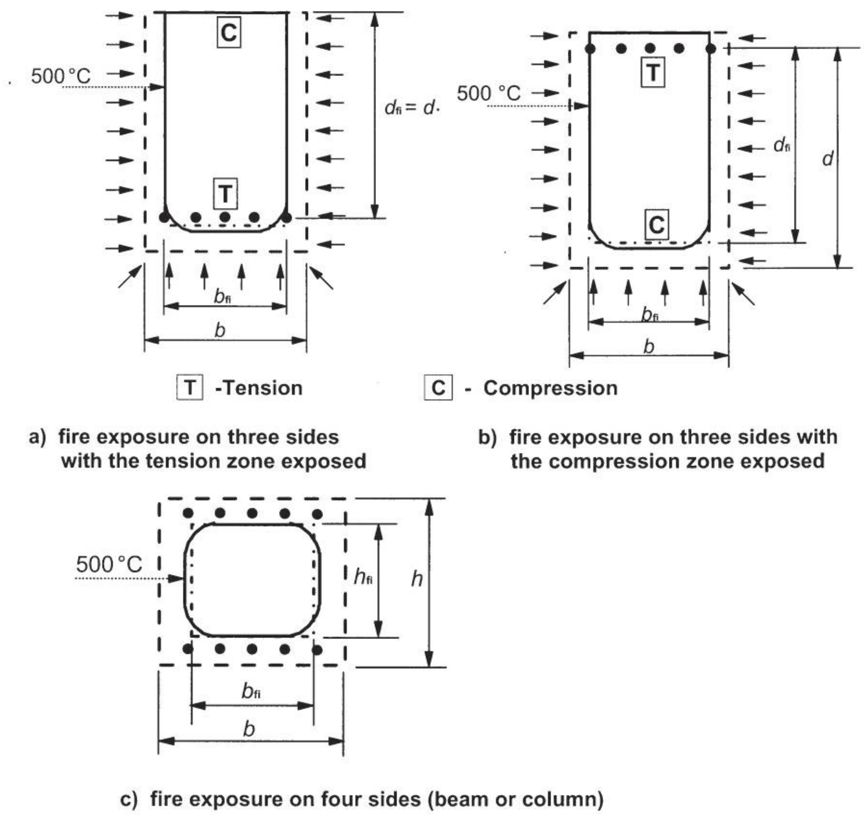


Fig. 9 - Reduced cross section of reinforced concrete beams and column

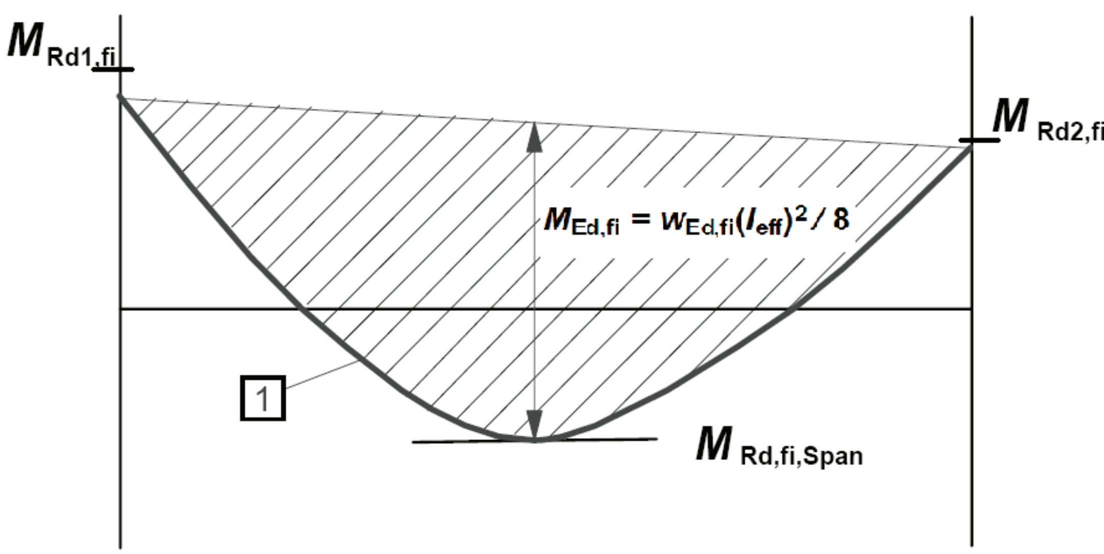


Fig. 10 - Positioning of the free bending moment diagram $M_{Ed,fi}$ to establish equilibrium