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EC 4: Structural Fire Design

EC 4: Calcul de la résistance au feu

EC 4: Brandbemessung

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SUMMARY

The scope of Eurocode 4 Part 10, now Part 1.2, is to give principles and rules to carry out a structural fire design for composite steel and concrete structures. Three different levels of structural fire design are described, covering member analysis and analysis of fire performance of total building frameworks. The last procedure may be used for the standard fire and for general time-temperature regimes. Concerning the relationship with Parts 10 of EC 2, concrete structures and of EC 3, steel structures, it should be emphasized that Part 10 of EC 4 is in full compliance with the thermo-mechanical material properties at elevated temperatures for concrete and steel. Strainhardening of steel may be activated for composite structures as local instability is less critical than for purely steel structures.

RESUME

L'Eurocode 4: partie 10, maintenant la partie 1.2, traite les méthodes de calcul pour la résistance au feu des structures mixtes acier-béton. On décrit les trois types de calcul pour l'analyse des éléments et de l'ossature du bâtiment. Le troisième type peut être utilisé pour un incendie normalisé et des courbes température-temps générales. Les parties 10 des Eurocodes 2 et 3 sont compatibles avec la partie 10 de l'Eurocode 4 quant aux caractéristiques thermo-mécaniques de l'acier et du béton à température élevée. On explique comment on peut profiter de l'écrouissage de l'acier puisque le flambement local est moins critique pour les structures mixtes que pour les structures en acier.

ZUSAMMENFASSUNG

Eurocode 4, Teil 10 (nun Teil 1.2) umfasst Prinzipien und Regeln zur Brandbemessung für Verbundtragwerke aus Stahl und Beton. Es werden drei unterschiedliche Niveaus für die Brandbemessung von Tragelementen und die Ermittlung der Feuerbeständigkeit ganzer Geschossrahmenbauten beschrieben. Letztere kann für Standardbrandkurven oder allgemeine Temperatur-Zeitverläufe verwendet werden. Die Werkstoffeigenschaften von Beton und Stahl bei höheren Temperaturen stimmen mit denen der Teile 10 von EC 2 (Betontragwerke) und EC 3 (Stahltragwerke) überein. Allerdings darf in Verbundtragwerken die Verfestigung des Stahls berücksichtigt werden, da örtliche Instabilität weniger kritisch ist als in reinen Stahltragwerken.

1. INTRODUCTION

Eurocode 4: Part 10, Structural Fire Design, was issued for national comments in 1990 [1], and is now being revised [2]. It should be issued as a pr ENV in 1993, for approval by CEN before publication as ENV 1994: Part 1.2.

This Part 1.2 of Eurocode 4 deals with the design of composite steel and concrete structures for the accidental situation of fire exposure and shall be used in conjunction with Part 1.1 of Eurocode 4 and Part 10 of Eurocode 1. This Part 1.2 only identifies differences or supplements to the design for normal conditions of use.

Typical composite cross-section types [3, 4] for slabs, beams and columns, partially developed in view of fire resistance requirements, are given in Fig. 1.

2. BASIC PRINCIPLES

Shear connection. For all composite cross-sections longitudinal shear connection between steel and concrete shall be assured according to the principles of Part 1.1 of EC4. Nevertheless there shall be no shear connection between the steel components directly heated of a composite cross-section and the encased concrete.

Performance requirements. For structural fire design the main failure criterion to be fulfilled is CRITERION "R" which corresponds to the requirement that structures shall maintain their load bearing function during the relevant fire exposure. It shall be verified that $R_{f,d} \geqslant E_{f,d}$, which means that the design load bearing resistance of the structure exposed to a fire shall be larger than the design effect of actions during the required fire exposure time. Structural failure will correspond to the loss of equilibrium and may be due to rupture of sections, buckling, plastic hing formation or structural collapse mechanism [5, 6]. Composite structural elements easily comply with criterion "R", due to an adequate concept of the cross-section. Therefore, it is normally not required to apply additional insulation.

Partial safety factors. For design values of the thermal and mechanical properties of steel and concrete, a partial safety factor of $\gamma_{M,f} = 1{,}0$ shall be adopted when considering the accidental situation of fire exposure.

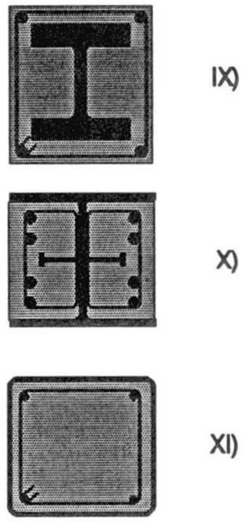
Assessment methods. The structural system adopted shall reflect the performance of the entire structure exposed to plan any fire [6]. This structural GENERAL ANALYSIS shall take into account the relevant failure mode in fire exposure, the temperature dependent material properties and stiffnesses and effects of thermal expansions. The design effect of actions $E_{f,d}$ shall be based on the fundamental factored load combination in the fire situation given in § 5.3.1 of Eurocode 1: Part 10 by f.i. 1,0G + 0,5W + 0,3Q.
As an alternative to the general analysis, an ANALYSIS OF PARTS OF THE STRUCTURE or a MEMBER ANALYSIS may be performed (see Table I). In this case, the design effect of actions in the fire situation $E_{f,d}$ may be deduced from E_d, the design effect of actions resulting from the fundamental factored load combination for normal conditions of use given in Eurocode 1, by $E_{f,d} = E_d/\gamma_F = 0{,}7E_d$. This alternative foreseen in § 5.3.2 of Eurocode 1: Part 10, considers a weighted mean partial safety factor for actions included in E_d of $\gamma_F = (1{,}35 + 1{,}5)/2 \cong 1{,}43$.

3. MECHANICAL AND MATERIAL PROPERTIES [1, 2, 5]

Strength and deformation properties. The strength and deformation properties of structural steel at elevated temperatures are - for heating rates between 2 and 50°C/min - characterized by a set of stress-strain relationships with a linear-elliptical shape for strains up to $\varepsilon_{a,\theta} \leq 2$ %. According to Fig. 2 this material law may be extended by the strain-hardening option for temperatures below 400°C, provided local instability is prevented and the ratio between the tensile strength at high tempature $f_{at,\theta}$ and the yield point at normal conditions of use $f_{ay,20°C}$ is limited to 1.25.
The strength and deformation properties of uniaxially stressed concrete at elevated temperatures are characterized by a set of stress-strain relationships as specified in Fig. 3. Whereas the main parameters are the compressive strength $f_{c,\theta}$ and the corresponding strain $\varepsilon_{c1,\theta}$, a descending branch should be adopted when a general calculation model is used (see Table II).

Thermal properties. The thermal conductivity λ and the specific heat C of steel and concrete are given in Fig. 4 as a function of temperature.

Fig.1/a: Typical cross-sections of composite slabs with profiled steel sheets.



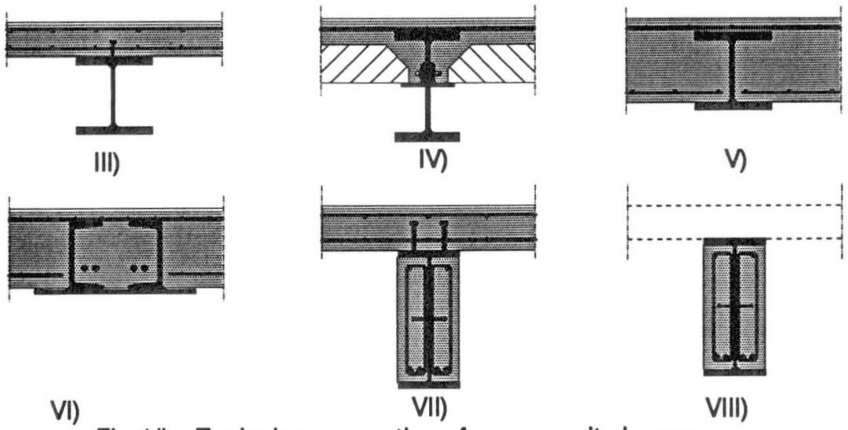


Fig.1/b: Typical cross-sections for composite beams.

Fig.1/c: Typical cross-sections for composite columns.

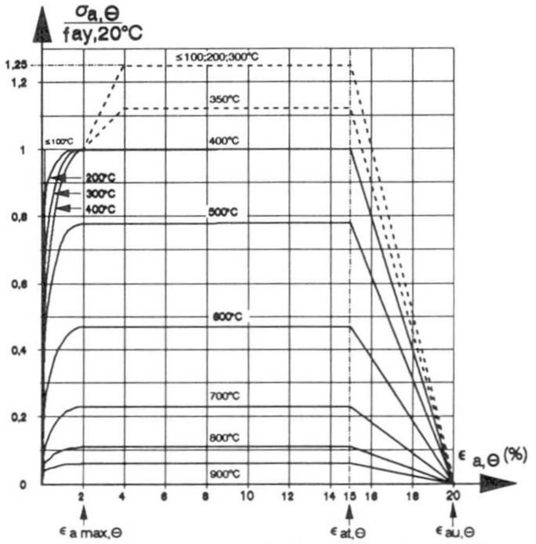


Fig. 2 : Stress-strain relationships for steel at high rising temperatures.

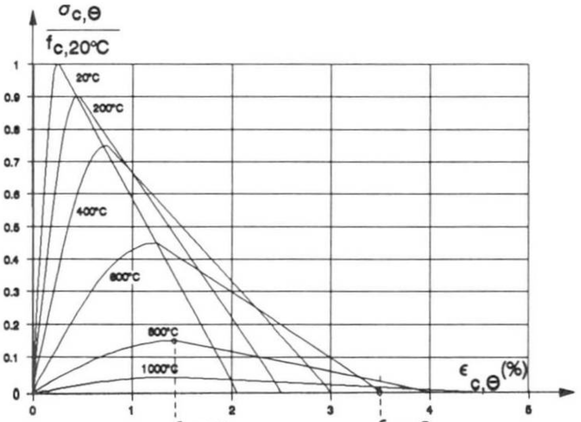


Fig.3: Stress-strain relationships for concrete at high rising temperatures.

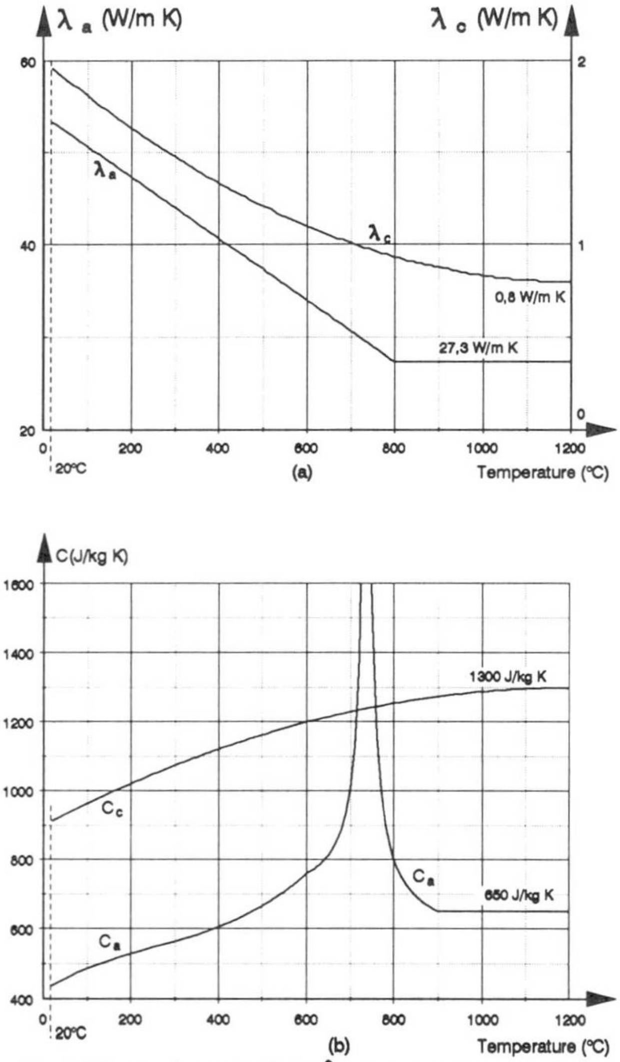
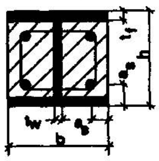


Fig.4: Thermal conductivity λ and specific heat C of steel (a) and concrete (c) given as a function of temperature.



Line		Fire Resistance Class R30	R60	R90	R120
1	Minimum cross-sectional dimensions for load level $\eta_f = 0.3$				
1.1	minimum dimensions h and b (mm)	160	260	300	300
1.2	minimum axis distance of reinforcing bars a_s (mm)	40	40	50	60
1.3	minimum ratio of web/flange-thickness t_w / t_f	0.6	0.5	0.5	0.7
2	Minimum cross-sectional dimensions for load level $\eta_f = 0.5$				
2.1	minimum dimensions h and b (mm)	200	300	300	-
2.2	minimum axis distance of reinforcing bars a_s (mm)	35	40	50	-
2.3	minimum ratio of web/flange-thickness t_w / t_f	0.6	0.6	0.7	-
3	Minimum cross-sectional dimensions for load level $\eta_f = 0.7$				
3.1	minimum dimensions h and b (mm)	250	300	-	-
3.2	minimum axis distance of reinforcing bars a_s (mm)	30	40	-	-
3.3	minimum ratio of web/flange-thickness t_w / t_f	0.6	0.7	-	-

Fig. 5 : Minimum cross sectional dimensions, minimum axis distance and minimum ratio of web/flange-thickness t_w / t_f of I-sections concreted between the flanges (LEVEL 1).

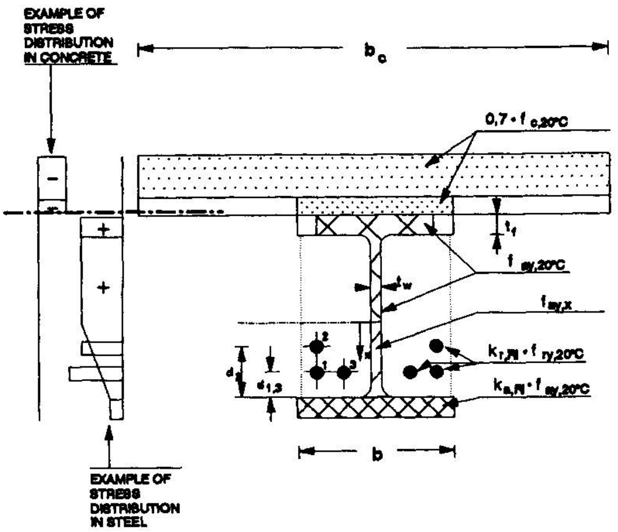


Fig. 6 : Evaluation of the positive plastic bending moment resistance of a composite beam, for different ISO-fire classes R30 to R180 (LEVEL 2).

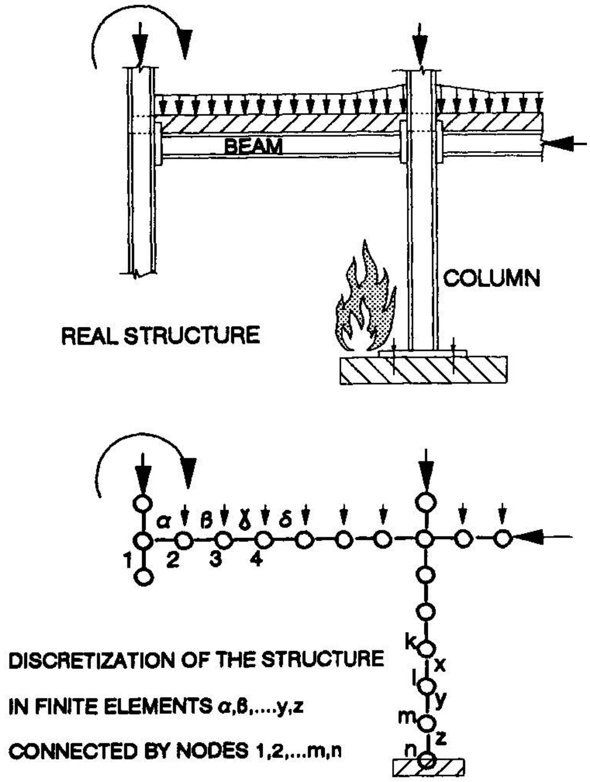


Fig.7 : Real structure with discretization in finite elements.

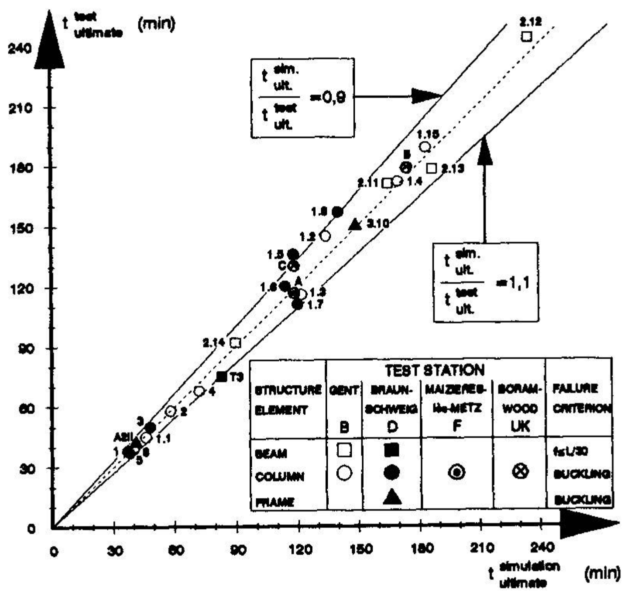


Fig. 8 : Fire resistance times measured and calculated by numerical model for columns, beams or frames of any cross section types (bare steel, protected steel, composite), [9,10,11,12].

4. STRUCTURAL FIRE DESIGN MODELS

Design shall be performed using any of the three fire design models, Level 1, 2 or 3, given in Table II. The relationship between these models and the three assessment methods related to the adopted structural system are shown in Table I.

Tabulated data / Level 1. Application of tabulated data is confined to individual member analysis. The structural member is considered as directly exposed to fire over its full length, and the thermal action corresponds to ISO-fire conditions. Extrapolation outside the range of experimental evidence is not allowed. This fire design model, as shown f.i. in Fig. 5, allows to determine the maximum load level η_f for fire design. This permits the evaluation of the design load bearing resistance of the structural member in the fire situation by $R_{f,d} = \eta_f \cdot R_d$, where R_d represents the design load bearing resistance at 20°C according to EC4: Part 1.1. It shall be verified that $R_{f,d} \geq E_{f,d} = 0{,}7 \cdot E_d$.

Simple calculation models / Level 2. Application of simple calculation models is normally confined to member analysis, but may be used for the analysis of parts of the structure (f.i. continuous beam or continuous column). The structural member is considered as exposed to ISO-fire conditions. Extrapolation outside the range of experimental evidence is not allowed. These fire design models [7 ,8], as shown in Fig. 6, permit the direct evaluation of the design load bearing resistance of the structural member in the fire situation $R_{f,d}$.
It shall, of course be verified that $R_{f,d} \geq E_{f,d} = 0{,}7\ E_d$.
General calculation models / Level 3. Application of general calculation models deals with the response to fire of structural members, of parts of the structure or entire structures. They permit the assessment of the interaction between parts of the structure which are directly exposed to any fire and those which are not exposed. Extrapolation outside the range of experimental evidence is allowed. These fire design models [9, 10, 11, 12] are based on a complete description of the physical processes involved (see Fig. 7). Their use is subject to a validation consisting in a verification of the numerical simulation results on basis of the corresponding test results (see Fig. 8). General calculation models permit a detailed investigation of the structural behaviour during and even after fire [13, 14]. They always lead to the design fire resistance time $t_{f,d}$ to be compared to the required fire resistance time $t_{f,r}$.

5. SOME SIGNIFICANT COMPARISONS

Members analysis by Level 1, 2 and 3 Design Models. As described in Table II the structural fire design model with a higher level, needs a larger calculation amount but corresponds also to a higher design accuracy. Therefore in order to obtain similar safety margins with these different design models, the corresponding design load bearing resistances for a given ISO-fire class should fulfill the following relation:

$$(R_{f,d})^{\text{LEVEL 1}} < (R_{f,d})^{\text{LEVEL 2}} < (R_{f,d})^{\text{LEVEL 3}}$$

Some representative calculation results for a partially encased beam connected to a slab and a partially encased profile column are shown in Table III.

Level 3 Design Model applied to different structural systems. Only general calculation models allow to determine the influence of the adopted structural systems. The general analysis of the entire structure leads of course to the largest calculation amount, but brings also the highest design accuracy. A general analysis permits the ACTIVATION OF HIDDEN LOAD BEARING RESISTANCES and leads to a HIGHER FIRE RESISTANCE TIME than that obtained with the member analysis of the weakest structural member of the structure. Some representative calculation results performed on an unprotected steel beam connected to a concrete slab are shown in Table IV [12, 15]. The permanent beam deflection after a local natural fire application, obtained when considering this heated beam as a part of the entire structure, is such that this composite beam may even not need to be replaced.

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TABLE I

STRUCTURAL FIRE DESIGN MODELS	ASSESSMENT METHODS $R_{f,d} \geq E_{f,d}$		
	MEMBER ANALYSIS	ANALYSIS WITH PARTS OF STRUCTURE	GENERAL ANALYSIS
LEVEL 1 TABULATED DATA	DESIGN EFFECT OF ACTIONS IN THE FIRE SITUATION GIVEN BY		
LEVEL 2 SIMPLE CALCULATION MODELS	$E_{f,d} = 0{,}7 \cdot E_d$		
LEVEL 3 GENERAL CALCULATION MODELS	EUROCODE1/PART10 / §5.3.2		LOAD COMBINATION IN THE FIRE SITUATION ↓ EUROCODE1/ PART10 / §5.3.1

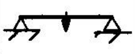
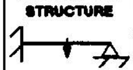
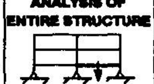
TABLE II

STRUCTURAL FIRE DESIGN MODELS	AIM	DESIGN RESULTS	DESIGN ACCURACY	CALCULATION AMOUNT
LEVEL 1 TABULATED DATA	INVESTIGATION OF MAIN PARAMETERS	ISO FIRE CLASSIFICATION	LOW	SMALL
LEVEL 2 SIMPLE CALCULATION MODELS	CALCULATION OF DESIGN LOAD BEARING RESISTANCE	$R_{f,d} \geq E_{f,d}$ FOR ISO FIRE	MEDIUM	MEDIUM
LEVEL 3 GENERAL CALCULATION MODELS	INVESTIGATION OF STRUCTURAL BEHAVIOUR DURING AND AFTER FIRE	$t_{f,d} \geq t_{f,r}$ FOR ANY FIRE	HIGH	LARGE

TABLE III

ANALYSED MEMBER	DESIGN LOAD BEARING RESISTANCE FOR ISO R90			
	TYPE	LEVEL 1	LEVEL 2	LEVEL 3
PARTIALLY ENCASED BEAM CONNECTED TO SLAB	BENDING MOMENT RESISTANCE	[1]	[8]	[12]
HE 900 AA / Fe 510 4φ28 / S500 SLAB 500 cm x 20 cm / C30	$M_{f,d}^{R90}$ in kNm	3548	3586	5105
PARTIALLY ENCASED PROFILE COLUMN	AXIAL LOAD BUCKLING RESISTANCE	[1]	[7]	[12]
HP 360x410x176 / Fe 510 8φ20 / S500 CONCRETE / C40	$N_{f,d}^{R90}$ in kN FOR $L_{cr} = 3$ m	3792	3851	4664

TABLE IV

COMPOSITE BEAM SPAN 15,5m	LEVEL 3/GENERAL CALCULATION MODEL [12, 15]		
IPE 600 / Fe 510 SLAB 240 cm x 15 cm / C35	MEMBER ANALYSIS 	ANALYSIS WITH PARTS OF THE STRUCTURE 	GENERAL ANALYSIS OF ENTIRE STRUCTURE 
PERMANENT BEAM DEFLECTION AFTER LOCAL NATURAL FIRE	24 cm	14 cm	10 cm