Fire safety and fire resistant design of steel structures for buildings according to Eurocode 3

Steel Design 2

Fire

A.F. Hamerlinck

bouwen met staal

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Colophon

text	dr.ir. A.F. Hamerlinck
editing	ir. C.H. van Eldik / Bouwen met Staal
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Fire is the second textbook in the Steel Design series. Previously published is *Structual basics* (Steel Design 1).

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Content

Fire

1 Fire safety

dr.ir. A.F. Hamerlinck

Bouwen met Staal and Adviesbureau Hamerlinck

1 Fire safety

The safety of buildings in case of fire is a subject, which is relatively unfamiliar to many structural engineers. This is strange because fire is one of the actions – along with e.g. self-weight, variable loads and wind – which structures have to withstand. By taking the fire load case into account early in the design phase, it is possible to assure sufficient fire safety of a structure at minimum cost.
This chapter first describes the objectives of fire safety and the many measures, which a designer can take in order to meet the requirements concerning fire safety. Next, how a fire can develop is discussed – depending on the chosen fire safety concept – and which fire safety requirements the national building regulations ask for. The terms 'main load bearing structure' and 'equivalent safety' for fire conditions are also discussed. Finally, the fire load case as an accidental action, and the behaviour of steel sections at elevated temperatures, are briefly discussed.

1.1 What is fire safety?

Fire is a chemical phenomenon, which usually involves a rapid chemical reaction (oxidation) of a combustible material (e.g. paper or oil) with oxygen. For this reaction to occur, a sufficiently high ignition temperature is required, caused for example by a cigarette, short circuit in an electrical device, or by arson. The largest threat of a fire to people and animals is not so much the flames themselves, but rather the smoke and hot gases.
Fire safety concerns measures to prevent the ignition of a fire as much as possible, and limit the risks associated with, and effects of, a fire. This section discusses the objectives of fire safety in general, and provides an overview of the possibilities a designer has to assure the fire safety of a building.

1.1.1 Goals of fire safety

Fire causes life-threatening situations for humans and animals (fig. 1.1). For this reason, the probability of occurrence and the possible consequences – casualties and damage – of a fire need to be limited by paying special attention to fire safety during the design, construction and use of the structure. Fire safety protection of buildings has two goals:
– preventing fatal accidents (casualties);
– reducing direct and indirect damage.

1.1 Fire causes life-threatening situations for humans and animals.

The safety of humans and animals is in most countries controlled by the government through building regulations. The rules focus on preventing the formation of fire, limiting the number of victims, and preventing the fire spreading to neighbouring buildings. Insurance companies focus more on the limitation of material damage, such as loss of furniture, and interruption to the production process (i.e. the use of the building). Fire safety is an important aspect in the design of buildings and affects the architecture, the building structure and the services. The different design strategies to create a fire-safe building usually consist of a 'package' of measures. The use of the building and the organizational aspects – such as evacuation of non-self-reliant people – play an important role.

The choice of the measures, which are taken – the fire safety concept – depends mainly on the layout and the use of the building. In public buildings, for example shops and libraries, the required fire safety is achieved through a combination of measures, such as smoke detection systems, smoke exhaust systems, and sprinklers. It is a misunderstanding to think that the fire resistance of a steel structure can only be achieved with protective measures such as fire-resistant coatings or coverings. Sophisticated computational methods are available which, for some cases, can demonstrate the acceptability of unprotected steel in a fire-safe building.

Fire safety focuses in general on the following three topics:

– safety of users;
– smoke control and escape routes;
– material damage.

The degree of utilization μ_0 is always less than or equal to the reduction factor for the the design load level in the fire situation $\eta_{fi} = E_{fi,d}/E_d$. The reduction factor η_{fi} is therefore an upper limit approach to the degree of utilization of μ_0. Take as an example a floor beam in an office building with combination factor $\psi_2 = 0{,}3$, loaded by a permanent load G_k and a variable load Q_k. If the beam has been designed on strength and meets the requirements at normal temperature (unity check of $E_d/R_{fi,d,0}$ less than or equal to 1,0), then $E_d = R_{fi,d,0}$ and therefore also $\mu_0 = \eta_{fi}$. This yields to the equation:

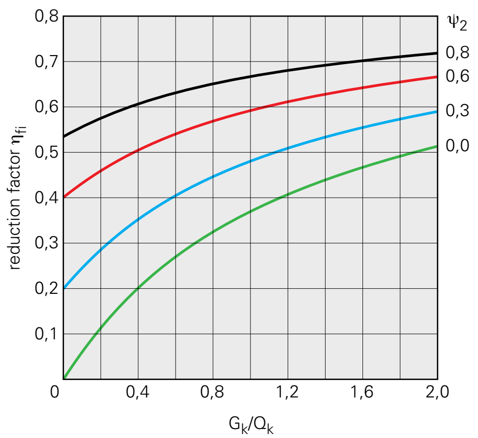
$$\eta_{fi} = \frac{G_k + \psi_2 Q_k}{\gamma_G G_k + \gamma_Q Q_k} = \frac{G_k + 0{,}3Q_k}{1{,}2G_k + 1{,}5Q_k} \geq \mu_0 \qquad (2.3)$$

The partial load factors are used according to EN 1990: $\gamma_G = 1{,}2$ for the permanent and $\gamma_Q = 1{,}5$ for the variable action (dependant on the National Annex). Depending on the loads, for offices the reduction factor for the the design load level in the fire situation varies usually between $\eta_{fi} = 0{,}48$ ($G_k = Q_k$) and $\eta_{fi} = 0{,}59$ ($G_k = 2Q_k$). Equation (2.3) is shown graphically in figure 2.3 for different occupancies (different values of ψ_2). See also table 4.3 in *Fire 4*. For countries in which the combination factor ψ_1 for frequent actions is mandatory instead of the combination factor ψ_2 for quasi-permanent actions, the equation has to be slightly modified.

NA

For centrically loaded columns in braced frames with n storeys, the upper limit for the degree of utilization in offices (with $\psi_2 = 0{,}3$ for fire and $\psi_0 = 0{,}5$ for normal load combinations (in which two storeys are fully loaded and n – 2 stories with combination factor ψ_0)) is set at:

$$\eta_{fi} = \frac{nG_k + n\psi_2 Q_k}{n1{,}2G_k + 2 \cdot 1{,}5Q_k + (n-2) \cdot 1{,}5\psi_0 Q_k} = \frac{G_k + 0{,}3Q_k}{1{,}2G_k + 0{,}75Q_k\left(1 + \frac{2}{n}\right)} \geq \mu_0 \qquad (2.4)$$



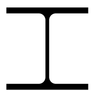
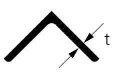
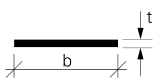
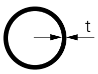
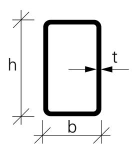
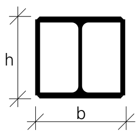
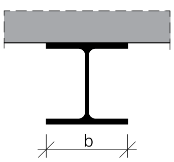
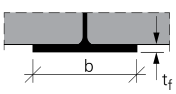
2.3 Relationship between the reduction factor for the design load level in a fire situation η_{fi} and the ratio between the permanent and the variable load G_k/Q_k for different values of the combination factor ψ_2.

For two storeys, the equations (2.3) and (2.4) are the same. For three or more storeys, the degree of utilization for columns is higher than that for beams. This is because the fundamental combination of actions for columns at normal temperature consists of two storeys fully loaded with a variable load, which is reduced on the other storeys by the combination factor ψ_0. The effect of the factor ψ therefore decreases with an increasing number of storeys and is less for columns than for beams.
In practice, the degree of utilization is usually in the order of 0,5. This value is smaller than follows from equation (2.3) or equation (2.4). This is because there is often extra loadbearing capacity available, for example in the case of beams because the deflection criterion is decisive, or in the case of columns because for practical reasons the same profile is used at several storeys. In the event of a fire, a reduced buckling length of the columns can often be applied (see section 2.6).

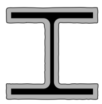
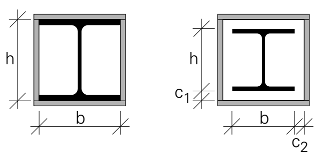
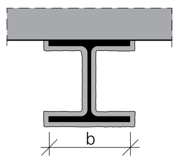
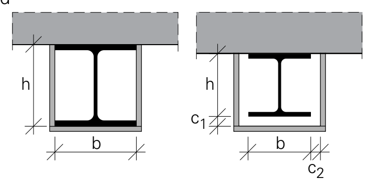
2.1.4 Section factor

The section factor takes into account the influence of the geometry of the steel section on the heating. The section factor is defined as the heated area A (in m^2 per metre of length) divided by the volume of the steel section V (in m^3 per metre of length), using m^{-1} as the unit. In practical terms, the section factor is therefore equal to the heat-exposed circumference (in m) divided by the area of the steel cross-section (in m^2).

- For *unprotected* sections, the section factor is referred to as A_m/V (index m for member), where A_m is the heated circumference of the profile and V is the area of the steel cross-section (fig. 2.4).
- In the case of *protected* sections, the section factor is referred to as A_p/V (index p for 'protected'), where A_p is the inner circumference of the *protection* and V is the area of the steel cross-section (fig. 2.5a). For *protection* that is not placed directly against the section – e.g. by means of spacers or a framework – no larger circumference is used and the smallest circumference around the heated section of the profile applies (fig. 2.5b). In the case of a *protected* steel girder below a concrete floor, the circumference of the steel section must be reduced by the width of the top flange. The concrete floor protects the upper flange against heating (fig. 2.5c and d).

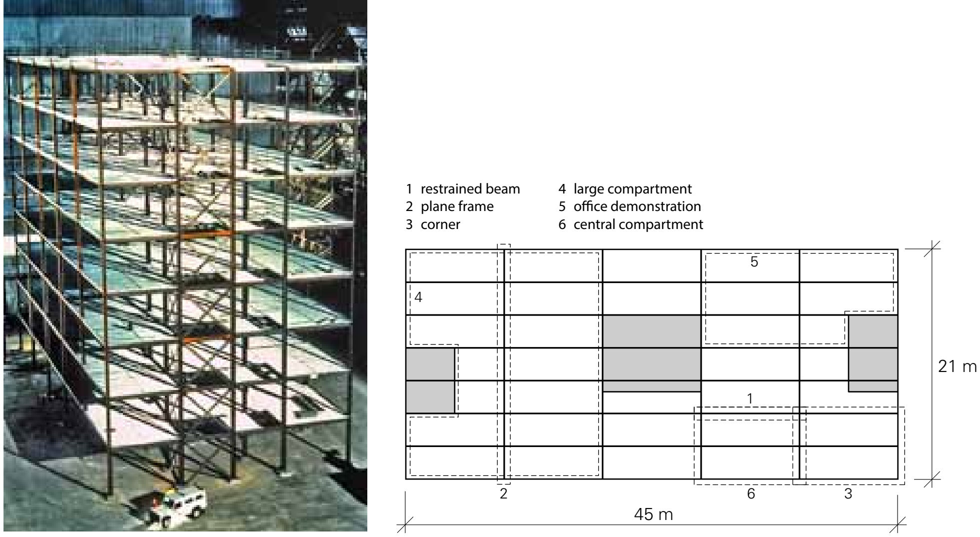
cross-section	description	section factor A_m/V
	open section exposed to fire on all sides	$\dfrac{\text{section perimeter}}{\text{cross-section area}}$
	angle exposed to fire on all sides	$\dfrac{2}{t}$
	flat plate exposed to fire on all sides	$\dfrac{2(b + t)}{bt} \approx \dfrac{1}{t}$ (for $t \ll b$)
	circular hollow section exposed to fire on all sides	$\dfrac{1}{t}$
	rectangular hollow section – or welded box of uniform thickness – exposed to fire on all sides	$\dfrac{2(b + h)}{\text{cross-section area}}$ $\approx \dfrac{1}{t}$ (for $t \ll b$)
	I-section with box reinforcement, exposed to fire on all sides	$\dfrac{2(b + h)}{\text{cross-section area}}$
	open section exposed to fire on three sides	$\dfrac{\text{section perimeter} - b}{\text{cross-section area}}$
	I-section flange (or a flat plate) exposed to fire on three sides	$\dfrac{b + 2t_f}{bt_f} \approx \dfrac{1}{t_f}$ (for $t_f \ll b$)

2.4 Section factor A_m/V (without 'shadow effect') for unprotected sections.

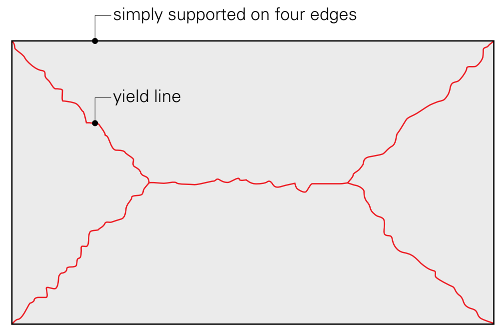
cross-section	description	section factor A_p/V
a 	contour encasement of uniform thickness, exposed to fire on all sides	$\dfrac{\text{section perimeter}}{\text{cross-section area}}$
b 	hollow encasement of uniform thickness[1], exposed to fire on all sides	$\dfrac{2(b + h)}{\text{cross-section area}}$
c 	contour encasement of uniform thickness, exposed to fire on three sides	$\dfrac{\text{section perimeter} - b}{\text{cross-section area}}$
d 	hollow encasement of uniform thickness[1], exposed to fire on three sides	$\dfrac{2h + b}{\text{cross-section area}}$

1. The clearance dimensions c_1 and c_2 should not normally exceed h/4.

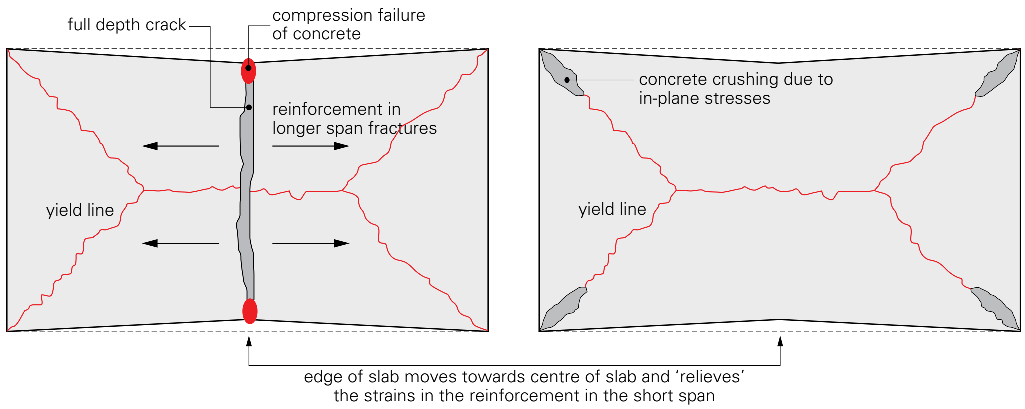
2.5 Section factor A_p/V for protected sections.



3.27 Cardington test building prior to concreting of the floors, and indication of test locations.



3.28 A typical yield line pattern for a rectangular slab simply supported on four sides.



3.29 Failure modes for composite steel-concrete slabs.

3.5.1 MACS

The MACS software is based on a simple design method developed by researchers at the Building Research Establishment (UK), following the experimental work at Cardington (fig. 3.27). The simple design method is principally based on yield line theory, being an ultimate load theory based on assumed collapse mechanisms and plastic properties. The collapse mechanism is defined by a pattern of yield lines along which the reinforcement yields and the slab undergoes plastic deformations (fig. 3.28). The areas bounded by the yield lines are assumed to remain rigid, with all rotation taking place at the yield lines.

In-plane forces occur in the slab in order to maintain equilibrium, thus inducing tensile stresses in the centre of the slab and a compressive ring around the perimeter. The resulting collapse mechanisms are illustrated in figure 3.29: fracture of the mesh across the short span, and compression failure of the concrete at the slab corners. The development of tensile and compressive in-plane forces influences the yield line moments developed in the slab, with reductions in bending resistance occurring in the tensile zone and enhancement of the bending resistance of the yield lines in the compression zone. In addition to this influence on bending resistance, there is additional load bearing capacity due to tensile membrane action developing in the slab at large displacements. Figure 3.30 shows a rectangular slab simply supported around its perimeter, and the expected lower bound yield line pattern that develops due to uniformly distributed loading. Derivation of the various formulae is given in section 6 of the MACS background document[9]. The level of enhancement of the bending resistance due to membrane forces is calculated by means of an enhancement factor for both elements, as shown in figure 3.30.

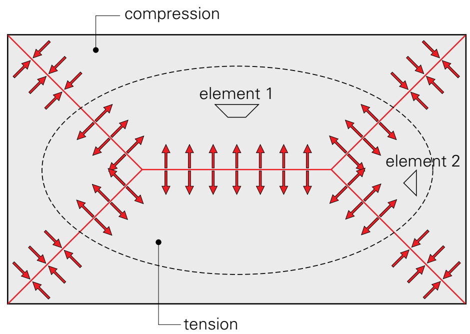
It is based on the equilibrium of the elements. The enhancement factor in the simple design method was derived by considering tensile failure of the mesh reinforcement. However, compressive failure of the concrete in the proximity of the slab corners must also be considered as a possible mode of failure, which in some cases (of high reinforcement ratios) may precede mesh fracture.

Tests have shown that load bearing capacity is enhanced by membrane forces provided that vertical support is maintained along the slab boundaries. Flat slabs which only have vertical support at their corners can not develop significant tensile membrane forces, and therefore receive little benefit from enhancement due to membrane action. For a composite slab supported on a grillage of steel beams in fire conditions, it is important to divide the slab into a number of rectangular areas where vertical support can be maintained on the perimeter of each area. These are referred to as floor design zones. The lines of vertical support are achieved by ensuring that the perimeter beams frame into column positions and are fire protected (fig. 3.31). Intermediate, secondary beams are left unprotected. Fire protection of the perimeter beams has to be specified, taking the slightly increased vertical loading into account.

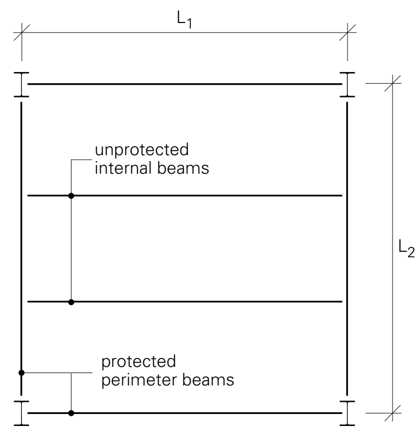
For ambient temperature design the floor is assumed to be continuous over the boundaries of each floor design zone. However, in fire conditions it is likely that cracks will form over the perimeter beams, due to the large thermal curvatures experienced by the slab. This may lead to fracture of the reinforcement, either due to the curvature or due to the combination of bending and membrane stresses. Fracture of the reinforcement in these hogging regions will occur before fracture of the reinforcement in the centre of the floor. Therefore, the floor design zones are considered to have no rotational or transverse restraint around their boundary.

The yield line pattern for a composite floor slab depends on the behaviour of the unprotected beams, which continually lose strength as the temperature increases. The load carrying mechanism of the floor therefore changes with increasing temperature. Initially, the composite slab acts as a one-way spanning element supported on the secondary beams. As these beams lose strength with increasing temperature the behaviour of the slab tends towards that of a simply supported two-way spanning element, resulting in the formation of the yield line pattern shown in figure 3.28. By assuming that this ultimate failure condition will occur when the beam strength is low relative to the slab, a conservative estimate of capacity is obtained relatively simply.

The load bearing capacity of the slab is calculated assuming that the composite beams have no strength, and is based on the yield line pattern which, whilst being compatible with the boundary conditions, provides the lowest load bearing capacity. This resistance is then enhanced by taking



3.30 Rectangular slab simply supported on all four edges showing in-plane forces across the yield lines due to tensile membrane action.



3.31 Typical floor design zone.

4.5 (continued)

Section factor A/V (m^{-1}) for IPE, HEA, HEB and HEM sections. Values given for unprotected sections include the correction factor to allow for the shadow effect k_{sh}.

section	unprotected		contour encasement		hollow encasement	
	four-sided heating	three-sided heating	four-sided heating	three-sided heating	four-sided heating	three-sided heating
HEB 600	60	50	86	75	67	56
HEB 650	60	50	85	74	66	56
HEB 700	59	50	82	72	65	55
HEB 800	59	51	81	72	66	57
HEB 900	58	51	78	70	65	57
HEB 1000	58	52	78	70	65	57
HEM 100	76	58	116	96	85	65
HEM 120	72	55	111	92	80	61
HEM 140	68	52	106	88	76	58
HEM 160	64	49	100	83	71	54
HEM 180	61	47	96	80	68	52
HEM 200	58	44	92	76	65	49
HEM 220	56	43	88	73	62	47
HEM 240	47	36	73	61	52	39
HEM 260	46	35	72	59	51	39
HEM 280	45	34	71	59	50	38
HEM 300	39	29	60	50	43	33
HEM 320	39	30	60	50	43	33
HEM 340	39	30	60	50	43	34
HEM 360	40	31	61	51	44	34
HEM 400	41	32	62	52	45	36
HEM 450	42	34	62	53	47	38
HEM 500	43	35	63	55	48	39
HEM 550	45	37	64	56	50	41
HEM 600	46	38	65	57	51	42
HEM 650	47	40	66	58	52	44
HEM 700	48	41	67	59	53	45
HEM 800	50	43	68	60	55	48
HEM 900	51	45	69	62	57	50
HEM 1000	53	47	70	64	59	52

4.6

Steel temperature θ_a (°C) of unprotected IPE and HE sections after 30 minutes exposure to the standard fire curve (based on values from table 4.5 and table 4.4), for both non-galvanized and galvanized steel.

section	non-galvanized steel		galvanized steel	
	four-sided heating	three-sided heating	four-sided heating	three-sided heating
IPE 80	835	833	835	833
IPE 100	834	831	834	831
IPE 120	833	830	833	829
IPE 140	832	828	832	827
IPE 160	831	825	831	824
IPE 180	829	823	829	821
IPE 200	827	819	827	816
IPE 220	825	814	824	810
IPE 240	822	809	820	802
IPE 270	819	804	817	797
IPE 300	816	798	812	788
IPE 330	811	791	805	779
IPE 360	804	782	795	767
IPE 400	798	774	787	757
IPE 450	790	767	778	750
IPE 500	781	759	765	743
IPE 550	771	751	754	737
IPE 600	761	744	744	734

section	non-galvanized steel		galvanized steel	
	four-sided heating	three-sided heating	four-sided heating	three-sided heating
HEA 100	822	798	820	787
HEA 120	822	798	820	787
HEA 140	818	789	815	776
HEA 160	813	779	808	764
HEA 180	810	774	804	757
HEA 200	803	764	794	747
HEA 220	794	753	782	739
HEA 240	782	744	767	734
HEA 260	777	741	760	730
HEA 280	771	738	754	726
HEA 300	760	735	744	712
HEA 320	752	732	738	700
HEA 340	747	730	736	692
HEA 360	744	728	734	684
HEA 400	740	724	729	675
HEA 450	737	722	724	667
HEA 500	736	718	718	659
HEA 550	735	719	716	661
HEA 600	735	720	714	662
HEA 650	735	720	712	662
HEA 700	733	718	707	657
HEA 800	734	721	707	664
HEA 900	732	719	701	659
HEA 1000	732	721	701	664

Steel Design 2

Fire

This book deals with the subject of fire safety and the design of fire resistant steel structures for buildings according to Eurocode 3.

- Chapter 1 describes the objectives of fire safety based on the behaviour of a fire and discusses the measures that a designer can take to meet the fire safety requirements found in building regulations.
- Chapter 2 deals with the calculation of the fire resistance of a steel structure. The simple calculation model is suitable for tension members, beams that are not sensitive to lateral torsional buckling, columns, and beams that are sensitive to lateral torsional buckling. The advanced calculation model is used for the calculation of the resistance of unprotected and protected integrated beams (as found in shallow floor construction).
- Chapter 3 deals with fire safety engineering. This is a relatively new field in which physical models are used to describe the behaviour of a fire and its effect on structures and users. Four situations are discussed for which fire safety engineering can already be applied in practice, namely: steel structures subject to a natural fire (local fires without flashover and compartment fires with flashover), steel structures located outside a building in the open air, and the system behaviour of a steel structure with a composite steel and concrete floor subject to a standard fire.
- Finally, chapter 4 contains fourteen design tables to allow easy determination of, among other things, the steel temperature, the reduction factor on the material strength, the cross-section class and the critical steel temperature.

The author – dr.ir. Ralph Hamerlinck – has extensive experience as a consultant, teacher and author in the field of fire safety. He is also closely involved in developments in regulations and standards on fire safety.

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