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

Steel Design 2

Fire

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Colophon

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Fire is the second textbook in the Steel Design series. Previously published is *Structual basics* (Steel Design 1).

Illustrations

All unnamed photographs and all drawings come from the archive of Bouwen met Staal.
L = left, R = right.

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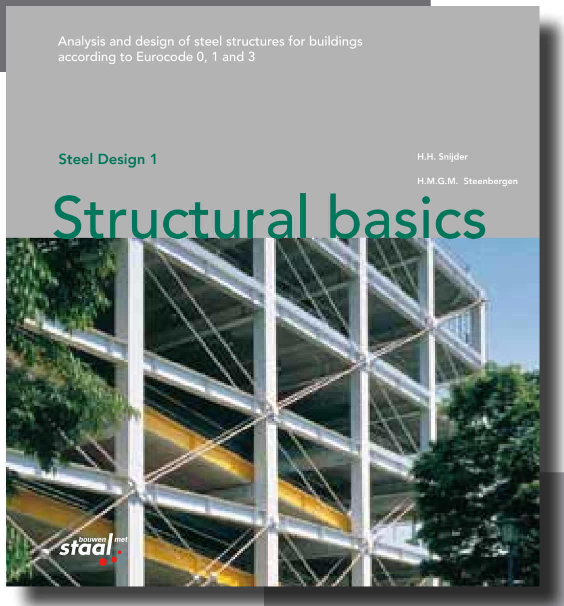
Structural basics

Steel Design 1



Structural basics

This textbook covers the design and analysis of steel structures for buildings according to EN 1990 (Eurocode 0), EN 1991 (Eurocode 1) and EN 1993 (Eurocode 3). It is effective as a textbook for students and as a reference guide to the Eurocodes 0, 1 and 3 for practising structural engineers.

The text is based on the (English) EN version of the Eurocodes using default and/or recommended values. Where a country can make a national choice – or when non-contradictory complementary information may be used – this is indicated by a symbol (black square). Separate annexes contain (for now) the national choices for Belgium, Luxembourg, The Netherlands and Switzerland. These annexes can be downloaded free of charge from the websites of the (national) organisations as well as any errata, corrections and additions to this textbook.

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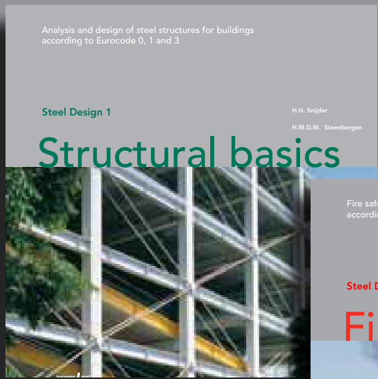


Steel Design 1

Steel Design 2

Steel Design series

Education and high quality textbooks are crucial to developing an interest in steel structures and their benefits for clients, architects and designers. However, despite the need to inspire the industry's next generation, many textbooks on steel structures are commissioned on a low budget, resulting in material that lacks imagination and tends to feature, at best, moderate illustrations. These textbooks are usually intended for high school and university level students, as well as designers who are not yet specialised in steel and steel construction. Therefore, it is vital that lecturers have access to up-to-date books that offer clear and concise explanations, while inspiring readers about the possibilities of steel through beautiful graphics and images.

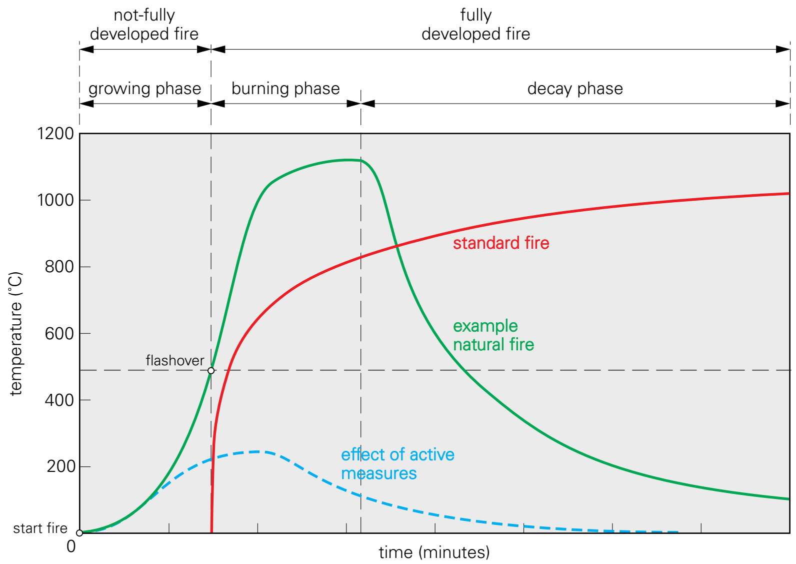
Steel Design is a set of English textbooks translated from the original Dutch that are based on the EN version of Eurocode with differences in nationally defined parameters included in an annex. These textbooks are intended for high-school and university level students. The content is applicable to designers who are not specialised in steel and steel construction.

World Steel Association

worldsteel has supported the development of study material related to steel in construction since 2018. This allows future architects and designers to take advantage of steel products and their features that support designs that meet the circular economy principles.

A separate opt-in programme has been developed called 'constructsteel.org' and is able to be joined by steel producers and construction industry related organisations upon application. This programme focusses on the construction market sector exclusively to promote steel and steel products.

Please see www.worldsteel.org and www.constructsteel.org for further details about the steel industry and specifically the construction market.



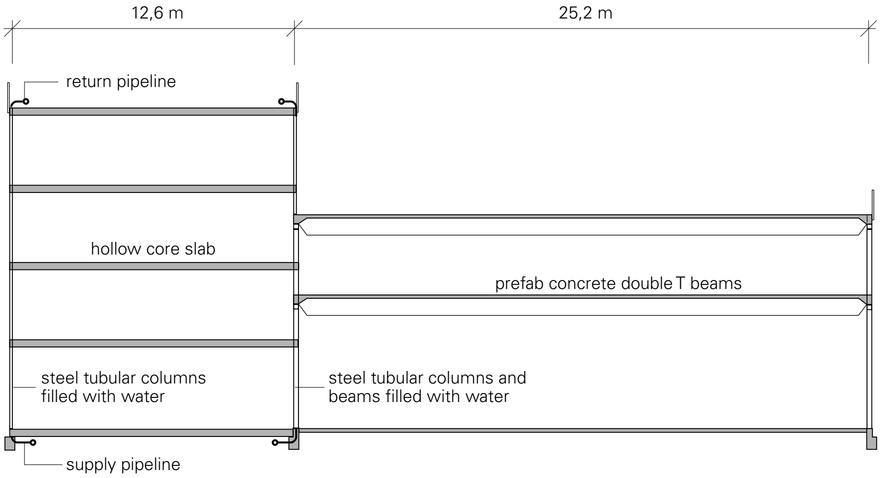
1.6 Development of a natural fire with the growing, burning and decay phase compared to the standard fire.

These factors differ for each situation. Therefore, the temperature development of the fire also varies from case to case. In figure 1.6 the possible temperature development over time for a real situation is sketched; this is called a natural or physical fire. The standard fire curve is also shown; this is the (assumed) standardized relationship between temperature and time. Based on either a natural fire or the standard fire, the behaviour of structures can be determined, both experimentally (through fire tests) and computationally.

Traditionally, individual structural members such as beams and columns are assessed based on the standard fire curve, which was defined in the 1920s. The fire resistance is in this case the time (in minutes) that a member can resist this standard fire under the design level of loading. It is assumed that the standard fire starts at flashover and that in the earlier period (growing phase) all occupants can escape from the building.

Modern design methods based on a natural fire – so called 'fire safety engineering' – offer the possibility of a more realistic approach. The structure is considered as a whole (or a part of one) in case of a fire. In reality, the temperature development over time in a natural fire determines to what extent the performance, which is expressed in minutes, corresponds to the fire resistance associated with a standard fire. The number of minutes resistance when subject to a standard fire provides only an indication of the real fire resistance. It is primarily a means of classification, and not much value in terms of identifying real minutes of fire fighters.

Finally it should be noted that the (standard) fire resistance of a structural member should not be interpreted as a measure of the available escape time, or time for intervention of the fire fighters. The development of a real (natural) fire and the resulting structural behaviour can differ significantly from that suggested by considering a single structural member in a standard fire test. As a result, the actual fire resistance in minutes can be much longer, or indeed shorter, than that determined in a standard fire test.

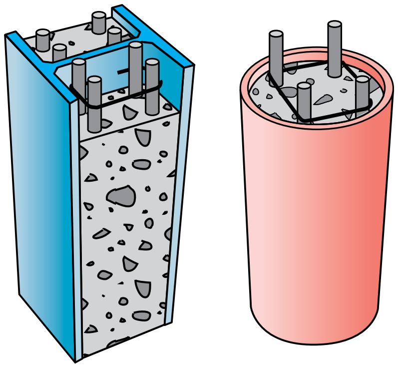


1.11 Fire station in Breda (The Netherlands): circular hollow sections filled with water. A water pump takes away the heated water (forced flow).

With natural flow the physical principle that hot water is lighter than cold water is relied upon, leading to an upward flow of the hot water. A pump is used to achieve forced flow (fig. 1.11). Whichever flow method is used the temperature of the steel structure can be designed to remain below 200 °C; a temperature at which the steel retains its full strength. This makes all durations of fire resistance achievable as long as the water flows, requiring a reliable water supply.
Previous experience is desirable before going down the route of a water-filled steel structure. Using water filled columns is particularly interesting for buildings with more than four storeys.

Filling or covering with concrete

1.12 Steel sections filled with, or encased in, concrete. The mass of the concrete delays the heating of the steel. Some of the force can be transferred from the steel into the concrete at elevated temperatures.



The heat capacity of a steel member increases strongly when hollow sections are filled with concrete, or when rolled sections are encased in concrete (fig. 1.12). The heating of the steel is delayed. In addition, loads can be partly transferred from the steel section to the cooler concrete cross-section.
Filling of hollow sections is a conventional way of increasing their fire resistance while maintaining the architectural expression of the steel. Partial or total encasement of I and H sections is also common in several European countries. Concrete filled hollow section columns have a minimum fire resistance of 30 minutes, even when the concrete is unreinforced and the column is relatively slender.
When the steel section and the concrete work together structurally, this is known as steel-concrete composite construction (fig. 1.13). A fire resistance of 120 minutes is achievable. For (the fire resistance of) steel-concrete composite structures, see reference [5].

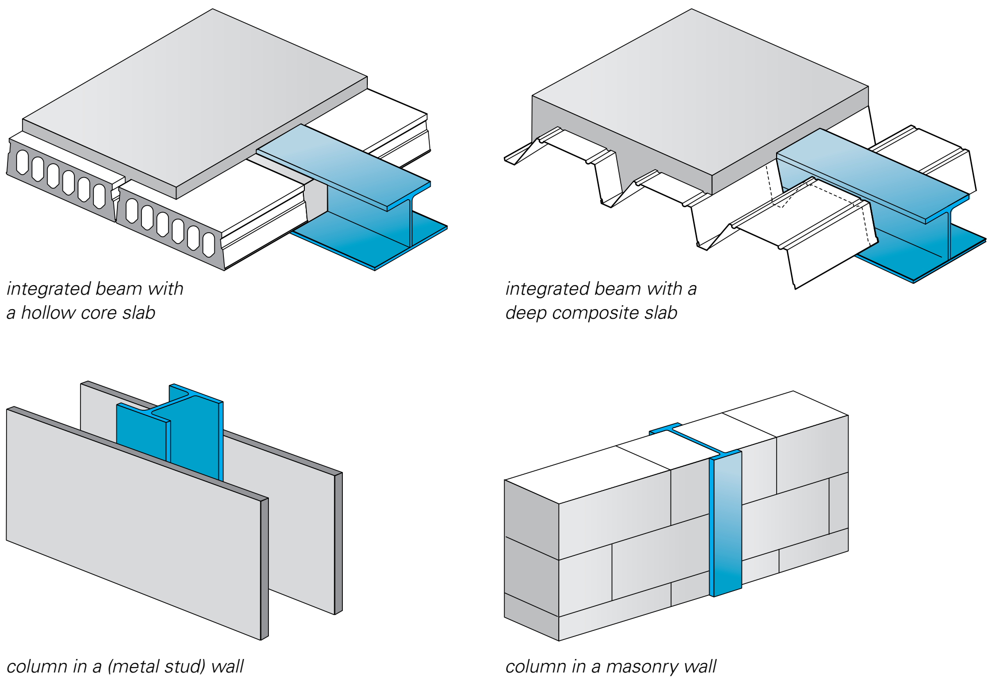
NA The free computer program Potfire[3] can be used for the analysis of both reinforced and unreinforced concrete filled hollow section columns. A3C[1] is another free program for the design of steel and composite columns (partially or totally encased with concrete).

The coating can be applied either off-site, in the fabrication shop, or on the construction site. With off-site application any damage that occurs during transportation will need to be repaired, and there will need to be on-site treatment of the joints. Also, attention to the thickness of the coating must be paid, because it affects drying time in the fabrication shop.

Most intumescent coatings are suitable for use in a non-aggressive internal environment. Some are suitable for outdoor applications. Approved test reports based on a standardized testing method[11] play a very important role in the approval by regulatory authorities and the fire department. Also, quality assurance during the application of the coating, including controlling the application conditions and checking the applied layer thickness, is important. Specific guidance on
NA quality assurance is available, see [2].

Structural integration

The designer can choose to integrate steel columns partially or totally within the depth of a façade or separating wall (fig. 1.16). This is also true for bracing members and other means of providing stability. The floor structure can also be designed such that the steel floor beams are located within the depth of the floor.



1.16 Integrated steel in floor and wall structures.

1.17 Integration of a steel column in a timber framed façade (covered with gypsum board on the inner side). Without additional measures, the fire resistance of the column is at least 30 minutes.

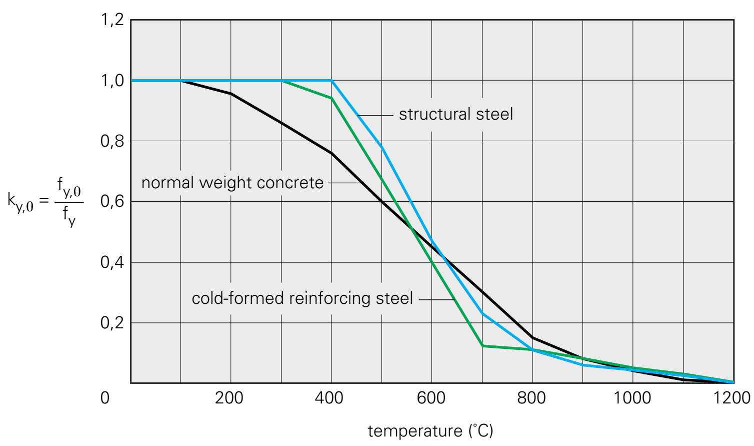
NA Every country specifies in its National Annex whether the frequent combination value ψ_1 has to be used in the accidental load combination fire (according to EN 1990, cl. 4.3.1), or the quasi-permanent combination value ψ_2. For wind loads $\psi_1 = 0{,}2$ and $\psi_2 = 0$. The combination values for variable actions depend on the occupancy of the building (category), see EN 1990, table A1.1:

$\psi_1 = 0{,}5$; $\psi_2 = 0{,}3$	floors of residential buildings, hotels and offices (category A and B);
$\psi_1 = 0{,}7$; $\psi_2 = 0{,}6$	floors of meeting rooms and stores (category C and D);
$\psi_1 = 0{,}9$; $\psi_2 = 0{,}8$	floors for storage (category E);
$\psi_1 = 0$; $\psi_2 = 0$	roofs.

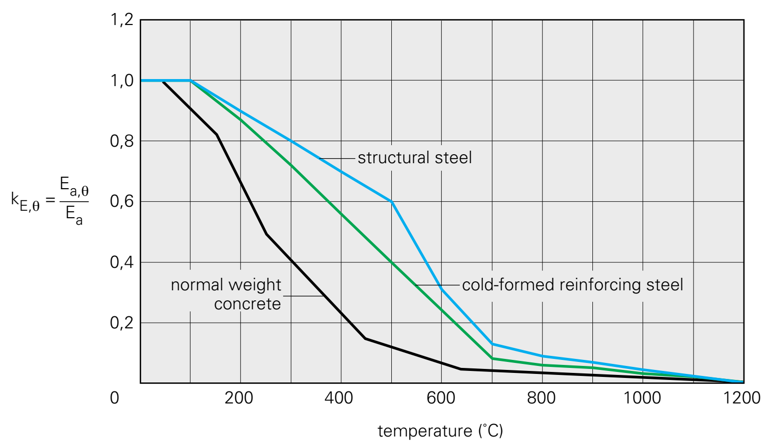
So, under fire conditions a roof structure is analysed for permanent action only ($\psi_1 = \psi_2 = 0$), unless the wind has to be taken into account (countries in which ψ_1 is mandatory) or the snow action on the roof (e.g. the Alpine and Scandinavian countries).
See *Fire 4* (Design tables), table 4.3 for the reduction factor for the design load level in the fire situation as a function of the occupancy and the national choice whether ψ_1 or ψ_2 has to be applied.

1.6 Behaviour of steel sections during fire

The temperature of a steel structure rises when exposed to fire. This causes a decrease in both the strength and stiffness of the steel. These decreases can be observed in the relationship between temperature and the effective yield strength (fig. 1.28), and in the relationship between temperature and the Young's modulus (fig. 1.29). The decreases in strength and stiffness of concrete and cold formed reinforcing steel are also shown for comparison. The full strength of the



1.28 Decrease of strength of structural steel, reinforcing steel and normal weight concrete with temperature.



1.29 Decrease of stiffness of structural steel, reinforcing steel and normal weight concrete with temperature.

Where:

A_m/v section factor of an unprotected the steel member, see section 2.1.4;

c_a specific heat of steel (in J/kgK), being temperature dependent (see EN 1993-1-2, cl. 3.4.1.2) and showing a strong peak due to changes in the crystallic structure at 735 °C;

ρ_a unit mass of steel (7850 kg/m^3);

k_{sh} shadow factor explained below.

Equation (2.7) is better understood if rewritten to the equation of equilibrium of energy between the energy transferred to the steel section and the energy stored in the section (causing an incease of temperature in the steel section):

$$\dot{h}_{net} k_{sh} A_m \Delta t = \Delta\theta_{a,t} c_a \rho_a V \qquad (2.8)$$

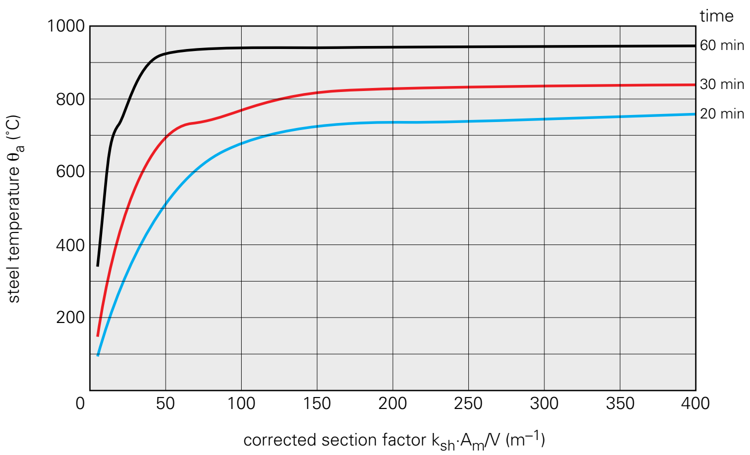
In the case of I shaped sections, the so-called 'shadow effect' plays a role. The shadow effect means that the radiative heat to the web and to the inner side of the flanges is partially shielded by the flanges. This effect is taken into account by the correction factor k_{sh} according to EN 1993-1-2, cl. 4.2.5.1:

$$k_{sh} = 0{,}9 \frac{\left(\dfrac{A_m}{V}\right)_b}{\dfrac{A_m}{V}} \qquad (2.9)$$

Where $(A_m/V)_b$ is the box value of the section factor for an imaginary rectangle that fits around the I shaped section.

Figure 2.10 shows the relationship between the (corrected) section factor and the steel temperature of an unprotected steel section after 20, 30 and 60 minutes of fire following a computation with the 'simple' calculation model of EN 1993-1-2. For a fire resistance of 30 minutes and a critical temperature of 600 ˚C – corresponding to a degree of utilization of $\mu_0 = k_{y,\theta} = 0{,}47$ (see fig. 2.2) – a section factor (were relevant corrected with the shadow factor k_{sh}) of up to 35 m^{-1} is required. The section factor is usually greater than 60 m^{-1} for conventional rolled and hollow sections and even greater than 140 m^{-1} for IPE sections (see table 2.6). As a result, a fire resistance of 30 minutes is generally not feasible for an unprotected steel structure. Usually, a fire protection measure – an insulating covering or the application of a composite steel-concrete construction – is necessary to obtain a fire resistance of 30 minutes (see section 2.2.4). A requirement of 30 minutes fire resistance can only be met with an unprotected steel structure at a low degree of utilization in combination with a low section factor. The choice of the section plays an important role here.

2.10 Relationship between the corrected section factor $k_{sh}{\cdot}A_m/V$ and the steel temperature θ_a of an unprotected I-section after 20, 30 and 60 minutes of fire. The graphs are given in tabular form in table 4.4 of *Fire 4*).



2.2.4 Heating of protected steel sections

An insulating plate material, a sprayed mortar or an intumescent coating slows down the heat flow to the steel profile, enlarging the time to reach the critical steel temperature.
For protected steel sections the increase of the steel temperature $\Delta\theta_{a,t}$ during a time interval Δt can be determined from EN 1993-1-2, cl. 4.2.5.2, equation (4.27):

$$\Delta\theta_{a,t} = \frac{\lambda_p \frac{A_p}{V}}{d_p c_a \rho_a} \cdot \frac{\theta_{g,t} - \theta_{a,t}}{1 + \frac{\phi}{3}} \Delta t - \left(e^{\phi/10} - 1\right)\Delta\theta_{g,t}$$

with (2.10)

$$\phi = \frac{c_p \rho_p}{c_a \rho_a} d_p \frac{A_p}{V}$$

Where:
A_p/V section factor of a protected steel member, see section 2.1.4;
λ_p thermal conductivity of the fire protection system (in W/mK);
d_p thickness of the fire protection material (in m);
c_p specific heat of the fire protection system (in J/kgK), being temperature dependent;
ρ_p unit mass of the fire protection material (in kg/m^3);
$\theta_{g,t}$ gas temperature (of the fire) at time t;
$\theta_{a,t}$ steel temperature (of the fire) at time t;
$\Delta\theta_{g,t}$ increase of gas temperature (of the fire) during the time interval Δt.

In protected sections there is no shadow effect.

The insulating properties of fire protection material depend not only on its thickness d_p, but also on the thermal properties of the protection material and in particular on the thermal conductivity λ_p. This coefficient generally depends strongly on the temperature, so that its value changes during the fire. For this reason, it is not permitted to use the value of the thermal conductivity at normal temperature in calculations.
The structural element to which the protection is attached may deform or deflect in the event of a fire. Therefore, the mechanical behaviour of the protection material is also important to prevent the protection from falling off, detaching and/or serious cracking. The constructional details – such as the number and type of mechanical fasteners and the shielding of the seams in case of plate protection – are important in this respect. The effects of the method of application cannot be theoretically assessed in advance. Therefore, EN 13381-4 and EN 13381-8 prescribe tests with protection materials on both loaded and unloaded steel sections. This enables the determination of the governing thermal properties of the protection during fire, depending on the temperature development and the thermal and mechanical deformations (the so-called ‘stickability’).

Example 2.1

- *Given.* A beam splice connection in which both members HEA 400 are provided with intumescent coating (fig. 2.14). The required fire resistance is 60 minutes. Both beams have welded end plates 300x360x20 mm with intumescent coating (except for the connection surface and the bolt holes) and 6 bolts M30. The moment in the fire design situation is $M_{E,\theta}$ = 115 kNm. The moment resistance at normal temperature is $M_{R,20}$ = 253 kNm.
- *Question.* Is protection of the bolts and nuts needed to reach the required fire resistance of 60 minutes?
- *Answer.* The section factor A/V of the unprotected part of the connection is calculated in three steps.

Step 1. The fire exposed area of the bolts is $A = 6 \cdot 2\pi(2r)^2 = 6 \cdot 2\pi \cdot (2 \cdot 15)^2 = 33929$ mm^2 (assuming the radius r of the fire exosed nut and the bolt heads are twice that of the bolt and the sides are protected after intumescing of the coating on the end plates, leaving only the upper surface of the bolt and nut unprotected);

Step 2. The volume of the bolted connection is $V = 2L_{end\ plate}b_{end\ plate}t_{end\ plate} + 6\pi r 2L_{bolt} = 2 \cdot 300 \cdot 360 \cdot 20 + 6 \cdot \pi \cdot 15^2 \cdot (2 \cdot 20 + 2 \cdot 30) = 4744115$ mm^3 (two end plates and the bolt extending 30 mm from the plates on both sides and neglecting the extra steel mass in the nut and bolt head outside the bolt diameter);

Step 3. De section factor is $A/V = (33929 \cdot 10^{-6})/(4744115 \cdot 10^{-9}) = 7{,}2$ m^{-1}.

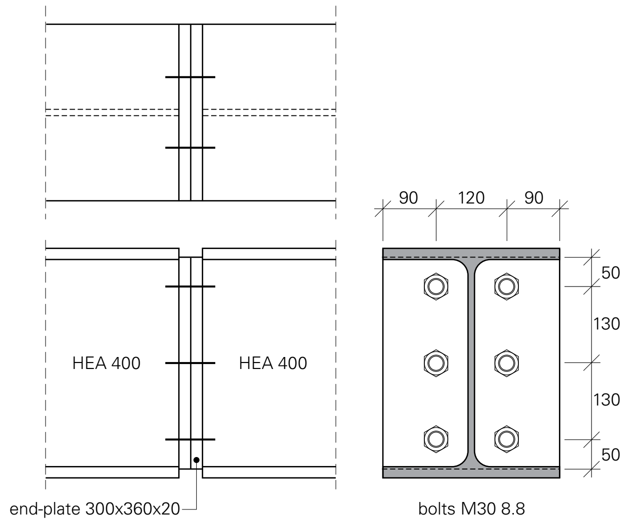
From table 4.4 of *Fire 4* it follows that after 60 minutes the connection temperature for A/V = 7,2 m^{-1} is $\theta_{a,con,60}$ = 433 °C.

Linear interpolation from EN 1993-1-2, table D.1(table 2.16) provides the strength reduction factor for bolts (tension and shear) for $\theta_{a,con,60}$ = 433 °C: $k_{b,\theta} = 0{,}700$.

The moment resistance in the fire design situation $M_{R,\theta}$ is checked:

$$M_{R,\theta} = \frac{k_{b,\theta} M_{R,20} \gamma_{M,20}}{\gamma_{M,\theta}} = \frac{0{,}7 \cdot 253 \cdot 1{,}25}{1{,}0} = 221 \text{ kNm}$$

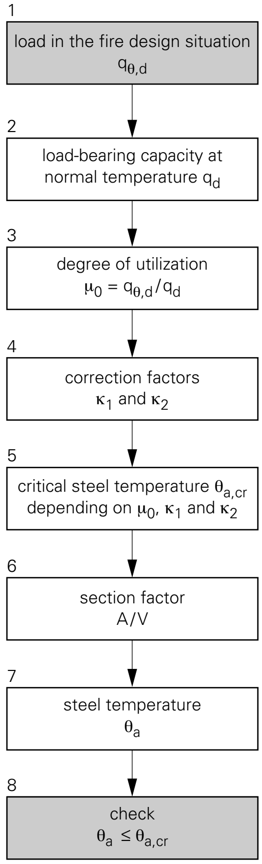
Since $M_{R,\theta}$ = 221 kNm > $M_{E,\theta}$ = 115 kNm, the connection complies without protecting the bolts and nuts.



2.15 Beam splice connection of example 2.1. Both HEA 400 sections have welded end plates which are fire protected by an intumescent coating. The nuts and bolts can be left unprotected for 60 minutes fire resistance.

temperature θ_a (°C)	bolts $k_{b,\theta}$ (tension and shear)	welds $k_{w,\theta}$	carbon steel $k_{y,\theta}$
20	1,000	1,000	1,000
100	0,968	1,000	1,000
200	0,952	1,000	1,000
300	0,903	1,000	1,000
400	0,775	0,876	1,000
500	0,550	0,627	0,780
600	0,220	0,378	0,470
700	0,100	0,130	0,230
800	0,067	0,074	0,110
900	0,033	0.018	0,060
1000	0,000	0,000	0,040
1100	0,000	0,000	0,020
1200	0,000	0,000	0,000

2.16 Strength reduction factors for bolts and welds, compared to that of the effective yield strength of carbon steel as a function of the steel temperature.



2.18 Flow chart for the assessment of the fire resistance of tension members and beams that are restrained against lateral torsional buckling.

In practice, it is usually not necessary to determine the exact fire resistance in minutes, but to check whether the fire resistance requirement of 30, 60, 90 or 120 minutes has been met. In principle, the approach consists of the same three parts as given in section 2.4, with a total of eight steps (fig. 2.18):

– calculation of the critical steel temperature $\theta_{a,cr}$ as a function of the (corrected) degree of utilization (steps 1-5);
– calculation of the steel temperature θ_a after 30, 60, 90 or 120 minutes (steps 6-7);
– check whether $\theta_a \leq \theta_{a,cr}$ (step 8).

- *Step 1.* Determine the load on the structure in the event of fire $q_{\theta,d}$ ($N_{\theta,d}$ for tension members).
- *Step 2.* Determine the load bearing capacity at normal temperature q_d (N_d for tension members).
- *Step 3.* Determine the degree of utilization:

$$\mu_0 = \frac{q_{\theta,d}}{q_d} \tag{2.14}$$

The equations (2.3) and (2.4) can also be used as a safe approach of the degree of utilization of, for example, office buildings.

- *Step 4.* Determine the adaptation factors κ_1 and κ_2, depending on the heating conditions of the beam. For beams that are fire-exposed form three sides under a composite or concrete floor, κ_1 = 0,7 (unprotected beams) or κ_1 = 0,85 (protected beams) applies. For statically undetermined (continuous) beams, κ_2 = 0,85 applies. In all other situations, $\kappa_1 = \kappa_2$ = 1,0.
- *Step 5.* Determine the critical steel temperature $\theta_{a,cr}$, represented by the red curves in figure 2.2 and figure 2.17 (righthand side):

$$\theta_{a,cr} = 39{,}19 \ln\left(\frac{1}{0{,}9674(\kappa_1\kappa_2\mu_0)^{3{,}833}} - 1\right) + 482\ ^\circ\mathrm{C} \tag{2.15}$$

Equation (2.15) can be derived from equation (2.2) with $\kappa_1\kappa_2\mu_0 = f_{y,\theta}/f_y = k_{y,\theta}$ and $\theta_{a,cr} = \theta_a$.

- *Step 6.* Determine the section factor depending on the type of section, the way of heating (from three or four sides) and the fire protective covering, if any (see fig. 2.4 and 2.5). The section factor is given in table 2.6 for the most common I sections. For unprotected hollow sections $A_m/V = 1/t$, where t is the wall thickness.
- *Step 7.* Based on the standard fire curve, determine the steel temperature θ_a after the fire-exposure time corresponding to the fire resistance requirement. In the case of unprotected sections, the development of the temperature in the steel section depends on the section factor, the thermal properties of steel (the steel surface emissivity ε_m, the specific heat c_a and the unit mass ρ_a) and on the heat transfer characteristics of the fire compartment (the emissivity of the flames ε_f and the convective heat transfer coefficient α_c). In the case of protected steel sections, the thickness and material properties of the protective material also play an important role.

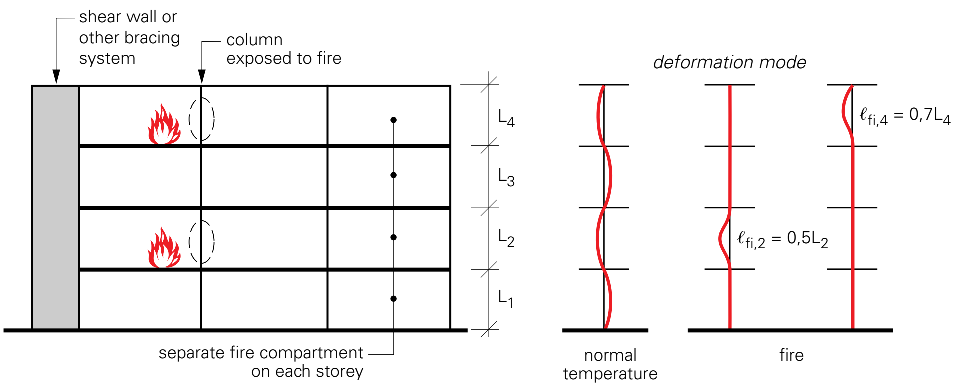
The calculation of the steel temperature θ_a is based on a time step method and is not possible manually. However, with a suitable computer program or available design tools, it is easy to calculate the steel temperature.

- *Step 8.* Check that $\theta_a \leq \theta_{a,cr}$.

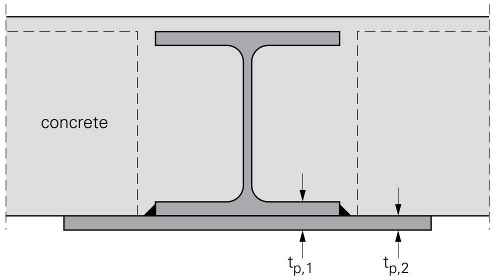
$$\chi_{fi} = \frac{1}{\varphi_\theta + \sqrt{\varphi_\theta^2 - \bar{\lambda}_\theta^2}} \quad \text{(buckling curve)} \tag{2.22}$$

$$N_{b,fi,t,Rd} = \chi_{fi} A k_{y,\theta} f_y \tag{2.23}$$

In a braced frame, the buckling length ℓ_{fi} of columns may, under certain conditions, be reduced in the fire design situation compared to the buckling length at normal temperature. The reduction applies to continuous columns or to columns that are connected rigidly (moment connection) or semi-rigidly (flexible connection). The fire must be confined to a single storey. In addition, the floor below and/or above the fire compartment shall form a separation between two fire compartments and shall also have a fire resistance at least equal to that of the column under consideration. Under these conditions, the column in the fire compartment can be considered to be fixed by the adjacent column in the upper and/or lower fire compartment. Figure 2.20 shows that the buckling length may then be reduced to 0,7 times the system length for the top column and 0,5 times the system length for the other columns, see EN 1993-1-2, cl. 4.2.3.2(5). The stiffness of the connecting, non-heated column(s) is high in relation to the column in the fire compartment, which has become weaker due to the heat.
For the same reason, for example, a base plate connection of a column on the ground floor can also be regarded as a rigid connection under certain circumstances. The condition for this is that the eccentricity of the load is not too great; this is usually the case for columns in a braced frame. The rotation of the base plate is then negligible compared to the column deformations that occur in the case of fire.
For centrically loaded columns with a relative slenderness of $0{,}5 \leq \bar{\lambda} \leq 1{,}4$, the reduction of the buckling length in the fire design situation by 50% means that the critical steel temperature increases by 50-150 °C (see table 2.19). This is the case with conventional wide flange sections up to a height of 300 mm and square hollow sections up to 200 mm with a yield strength of 235-355 N/mm^2 and a storey height of 3,5 m.



2.20 Reduction of the buckling length of columns in braced frames.



2.24 Definition of plate thicknesses to be used in the calculation of temperatures for an SFB beam.

time (min)	A_w	B_w	C_w	D_w
30	–140,7	832,4	0,00317	–0,0230
60	–103,8	968,6	0,00232	–0,0182
90	–108,6	1146,7	0,00198	–0,0154
120	–70,4	1124,4	0,00158	–0,0134

2.25 Parameters for calculating the web temperatures in equation (2.30).

For SFB beams two zones have to be considered: a zone with plate thickness $t_{p,1}$ (bottom plate and bottom flange together) and a zone with plate thickness $t_{p,2}$ (only the bottom plate), see figure 2.24. This method can be used when at least 85% of the bottom plate is covered by the concrete or composite floor or when the cannelures of the composite floor are filled with mineral wool. If not an alternative method has to be used to calculate the heating of the cross-section.

The temperatures in the web θ_w are much lower and can be determined from equation (2.30) as a function of the distance z to the top of the bottom plate or bottom flange (in case of a SFB beam).

$$\theta_w = k_1 e^{k_2 z}$$

with

$$k_1 = A_w \cdot \ln t_p + B_w$$
$$k_2 = C_w \cdot \ln t_p + D_w \qquad (2.30)$$

The parameters A_w, B_w, C_w and D_w are given in table 2.25. From equation (2.30), the point in the web where the temperature of 400 ˚C is reached, can be calculated. Above this point no strength reduction is necessary; below this point the strength is reduced.

The top flange temperature is always below 400 ˚C (for every type of beam and even for the smallest section after 120 minutes), and no strength reduction is necessary.

Verification of the bottom plate

The purpose of assessing the bottom plate is to determine whether the bottom plate (and in the case of an SFB beam, also the bottom flange) is sufficiently strong in the transverse direction to transfer the floor loading to the beam web. This is because the force transmitted from the support of the hollow core slab via the bottom plate to the web causes stresses due to bending and shear in the transverse direction in the bottom plate (and, in the case of an SFB beam, also in the bottom flange).

Example 2.6

- *Given*. An office building with integrated beams SFB 200-HEB 200-400x15 (fig. 2.31) in S355 with W_{pl} = 822·10^3 mm^3 and hollow core slabs with a support length of 80 mm. The weight of the integrated beams is G_k = 1,1 kN/m; they have a centre-to-centre distance a = 7,2 m spanning L = 4,5 m. The weight of the hollow core slabs is G_k = 5,5 kN/m^2. They are designed for a variable load Q_k = 4 kN/m^2 with ψ_2 = 0,3. NA
- *Question*. Check whether a fire resistance of 60 minutes is achieved.
- *Answer*. The load on the beam in the event of fire is $q_{\theta,d} = (G_{k,floor} + \psi_2 Q_k)a + G_{k,beam}$ = (5,5 + 0,3·4)·7,2 + 1,1 = 49,3 kN/m and the corresponding bending moment $M_{\theta,d} = q_{\theta,d}L^2/8$ = 49,3·4,5^2/8 = 125 kNm. The load from the floor on the beams on one side is $q_{l\theta,max} = 0{,}5q_{\theta,d}$ = $0{,}5(G_{k,floor} + \psi_2 Q_k)a$ = 0,5·(5,5 + 0,3·4)·7,2 = 24,1 kN/m.

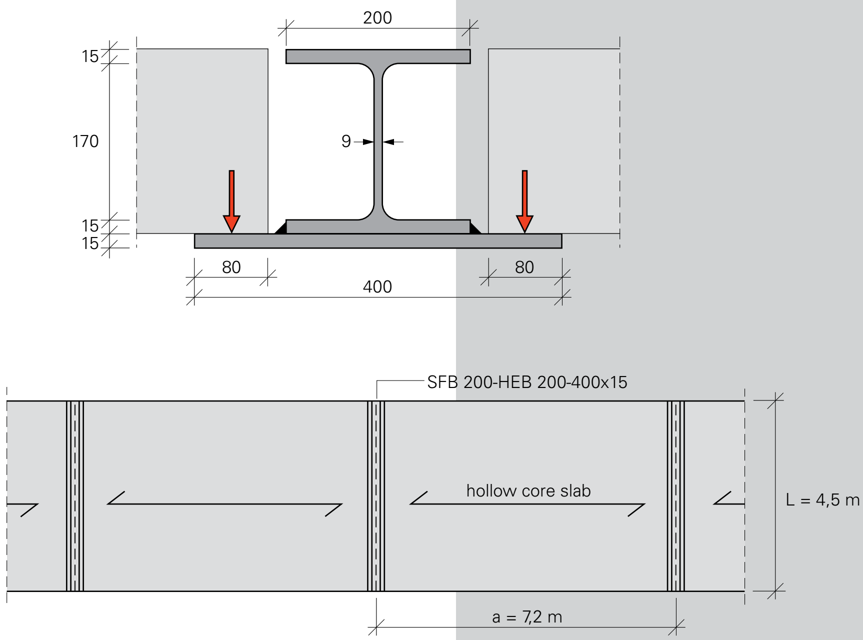
For a fire resistance of 60 minutes, the temperatures of the bottom parts are calculated with equation (2.29), see figure 2.32:

bottom flange above bottom plate: $\theta_1 = At_p^2 + Bt_p + C$

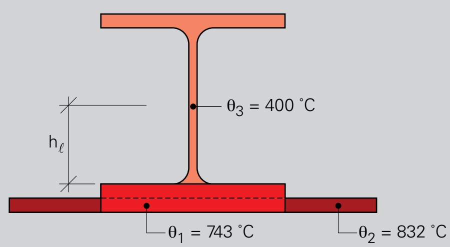
$$= 0{,}13 \cdot 30^2 + 11{,}8 \cdot 30 + 980 = 743\ ^\circ\text{C}$$

extended bottom plate: $\theta_2 = At_p^2 + Bt_p + C$

$$= 0{,}13 \cdot 15^2 + 11{,}8 \cdot 15 + 980 = 832\ ^\circ\text{C}$$



2.31 Integrated beam SFB 200-HEB 200-400x15 supporting hollow core slabs.



2.32 Temperature in three characteristic points: θ_1 = 743 °C, θ_2 = 832 °C and θ_3 = 400 °C.

3

Fire safety engineering

Fire safety engineering – often referred to as FSE – is a field in which physical models are used to describe the behaviour of a fire and its effect on a structure and its occupants. The actual behaviour of a structure in the event of fire sometimes differs considerably from the behaviour observed and measured in a standard fire test, or predicted by a simple calculation. Sometimes the actual behaviour is (much) more favourable, but sometimes it is (much) more unfavourable. The latter in particular calls for a reconsideration of the current classification system, whereby the behaviour of a structure in the event of a fire is assessed by considering the behaviour of individual components exposed to a standard fire.

It is possible, and indeed practical, to carry out an advanced calculation of the behaviour of a (steel) structure in the event of fire (fig. 3.1). Of particular relevance are EN 1991-1-2, which describes the mechanical and thermal actions during fire, and EN 1993-1-2, with which the thermal and mechanical response of the steel structure subject to these actions can be determined.

This chapter first describes what is meant by fire safety engineering, see section 3.1. Then four situations are discussed, each of which can be considered with FSE in practice, namely:

- steel structures subject to a natural fire without flashover (local fires, e.g. in car parks), see section 3.2;
- steel structures subject to a natural fire, where flashover does occur (compartment fires), see section 3.3;
- steel structures located outside a building in the open air, see section 3.4;
- system behaviour of steel structures with composite slabs subject to a standard fire, see section 3.5.

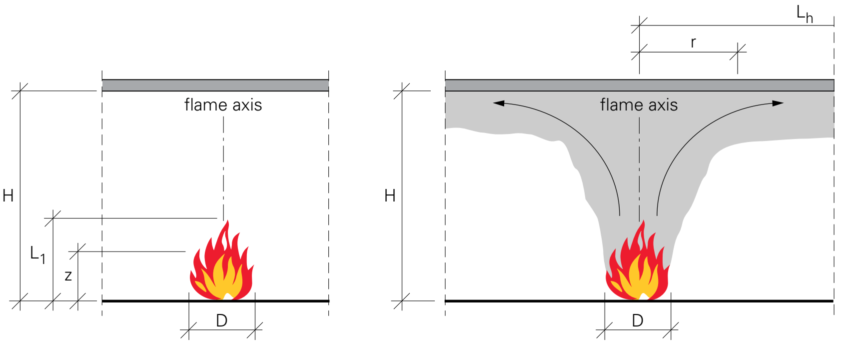
3.1 With fire safety engineering attractive (fire) safe buildings can be achieved, like this sports hall with an indoor athletics track and a free span of 80 m in Louvain-la-Neuve (Belgium).

However, in order to assess the behaviour of a structural element just above the fire, assuming an even temperature distribution in the hot top layer is unsafe and the two-zone model must be combined with a model for the local fire. The temperatures in the immediate vicinity of a floor beam, for example, are obtained by assuming the highest temperature at each point along the beam, as predicted by one of the two models.
The steel temperature can also be calculated using special software that takes into account local effects, such as whether or not the flames touch the ceiling, for example for car parks. See also EN 1991-1-2, cl. 3.3.1.3 and annex C.

In the 2010s, important knowledge was developed as part of the European LOCAFI project, in which the effect of local fires on the thermal actions and heating of a steel column were studied. Outcomes of this project have been presented in practical design rules and tools, and disseminated in publications and seminars[8,18]. Also, for most member states, the legal situation concerning application of fire safety engineering in the Eurocodes has been explained in 'legal context' documents.

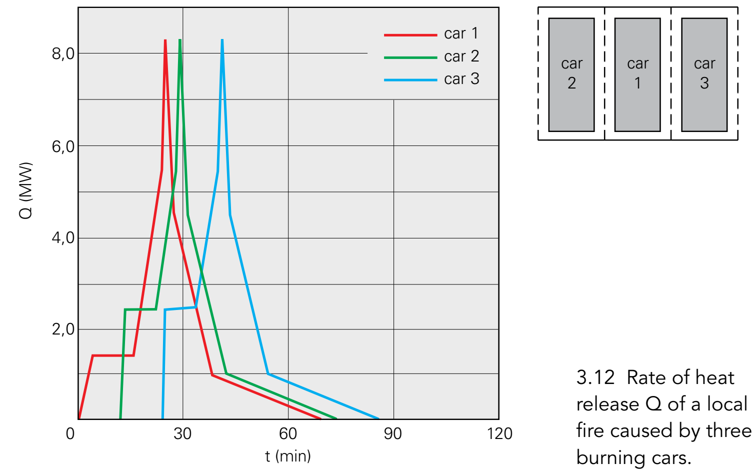
3.2.1 LOCAFI

The LOCAFI design guide[8] presents a design method for determining the temperature of a column subject to a local fire. The approach is aligned to the Eurocodes. The present annex C of EN 1991-1-2 gives a method for calculating the flame length and temperatures in the plume of a local fire. It is based on work by Heskestad and Hasemi, which provides a correlation between fire size (defined by the rate of heat release and diameter) and other parameters, including the flame height and the internal temperature of the fire. Fires that impact a ceiling tend to spread in a radial direction. The model accounts for this when calculating the temperature distribution within the compartment. Figure 3.5 shows both situations. As part of the LOCAFI project, the annex C method was extended to include an assessment of the temperature and the heat flux received by a member at a given distance from the fire source.

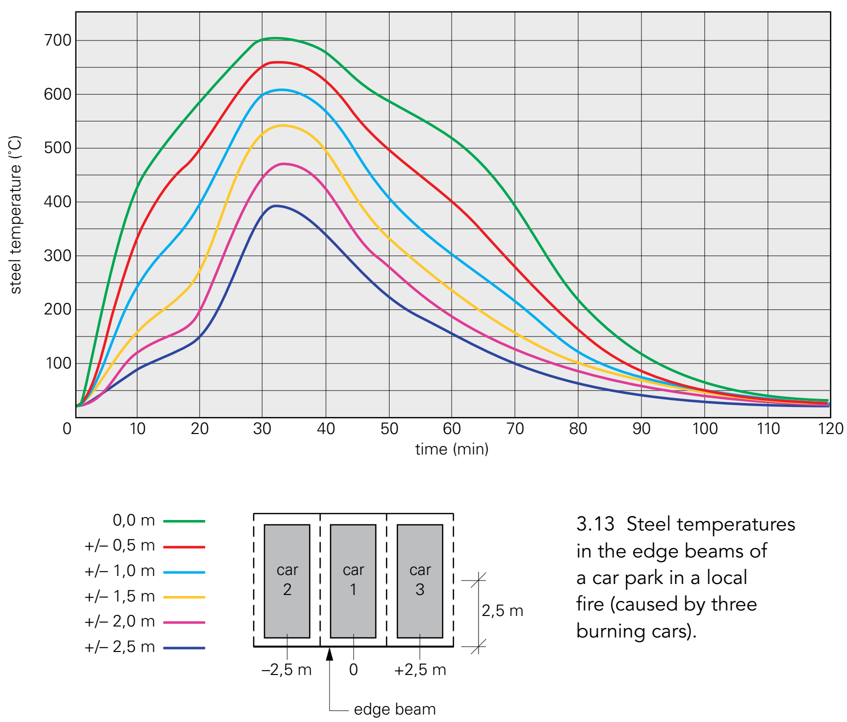


3.5 Key parameters describing a local fire
(left: fire does not impact the ceiling, right: fire impacts the ceiling).

How the parked cars are positioned in relation to the steel floor beams and the columns is also important. European research has shown that, in some circumstances, a fire in one car can spread to a car parked next to it after about twelve minutes. A fire in up to three cars is usually considered as a fire scenario (car 1 ignites at t = 0 min, car 2 at t = 12 min and car 3 at t = 24 min; see also fig. 3.12). Further spread of the fire is possible if no repressive action by the fire brigade – or other means like sprinklers – is possible. Fire spread, however, generally has no consequences for the structure, because the fire 'travels' through the car park, and by the time the fire has transferred to other cars the first burning cars are burnt out. New parts of the floor structure are then heated, while the structure where the fire started is already cooling down.
When a structure is able to withstand a fire from three cars – which are at a random, but most unfavourable, location – without collapsing the fire resistance requirement for car parks of 60 minutes or
NA more is then met. In some countries (for example France and the Netherlands) design recommendations specify the number of burning cars and the local fire load of the burning cars. In other countries (such as Germany) bare steel structures can be used without further analysis, based on the restricted, local fire scenario with a low risk of collapse of the steel structure.
The rate of heat release of a scenario of three burning cars, each with a fire load of 9500 MJ (class 3), is shown in figure 3.12. The area below the graphs for each car corresponds to a fire load of 9500 MJ multiplied by a reduction factor of around 0,7. This reduction factor was determined based on research - it appears that not all materials ignite, or that unburnt gases only ignite outside the car park. The real fire load to consider is therefore 6800 MJ. Based on this, the actual steel temperature occurring during a natural fire can be calculated (fig. 3.13), and it can be demonstrated that the structure can withstand the local car park fire. Figure 3.14 shows a project in which this fire engineering method was applied.

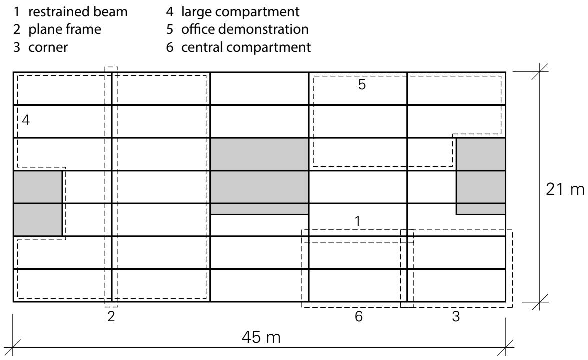


3.12 Rate of heat release Q of a local fire caused by three burning cars.

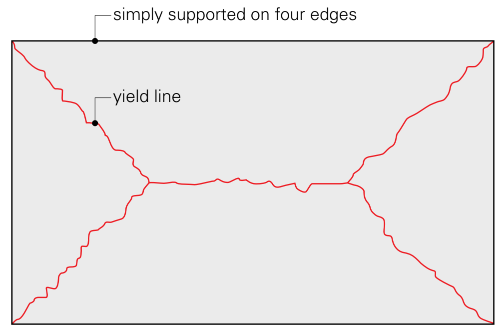


3.13 Steel temperatures in the edge beams of a car park in a local fire (caused by three burning cars).

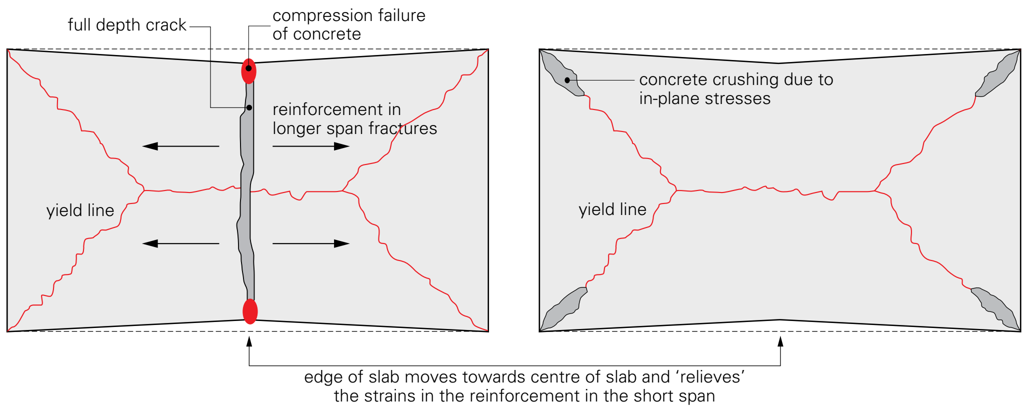
3.14 Medimall car pak in Rotterdam (Netherlands), in which the fire engineering method considering physical, local fires was applied.



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

4.3 (continued)

NA

Reduction factor for the design load level in the fire situation η_{fi} as a function of the ratio between the permanent load G_k and the variable load Q_k for different occupancies and load factors $\gamma_G = 1{,}2$ and $\gamma_Q = 1{,}5$, according to equation (2.3) of *Fire 2* (Calculation of the fire resistance). Every member state specifies in its National Annex whether Ψ_1 or Ψ_2 has to be applied.

G_k/Q_k	$\Psi_1 = 0{,}0$ (roofs)	$\Psi_1 = 0{,}2$ (wind actions)		$\Psi_1 = 0{,}5$ (office, residential)		$\Psi_1 = 0{,}6$ (shopping, congregations)		$\Psi_1 = 0{,}9$ (storage)
	$\Psi_2 = 0{,}0$ (roofs, wind actions)		$\Psi_2 = 0{,}3$ (office, residential)		$\Psi_2 = 0{,}6$ (shopping, congregations)		$\Psi_2 = 0{,}8$ (storage)	
1,35	0,433	0,497	0,529	0,593	0,625	0,657	0,689	0,721
1,40	0,440	0,503	0,535	0,597	0,629	0,660	0,692	0,723
1,45	0,448	0,509	0,540	0,602	0,633	0,664	0,694	0,725
1,50	0,455	0,515	0,545	0,606	0,636	0,667	0,697	0,727
1,55	0,461	0,521	0,551	0,610	0,640	0,670	0,699	0,729
1,60	0,468	0,526	0,556	0,614	0,643	0,673	0,702	0,731
1,65	0,474	0,532	0,560	0,618	0,647	0,675	0,704	0,733
1,70	0,480	0,537	0,565	0,621	0,650	0,678	0,706	0,734
1,75	0,486	0,542	0,569	0,625	0,653	0,681	0,708	0,736
1,80	0,492	0,546	0,574	0,628	0,656	0,683	0,710	0,738
1,85	0,497	0,551	0,578	0,632	0,659	0,685	0,712	0,739
1,90	0,503	0,556	0,582	0,635	0,661	0,688	0,714	0,741
1.95	0,508	0,560	0,586	0,638	0,664	0,690	0,716	0,742
2,00	0,513	0,564	0,590	0,641	0,667	0,692	0,718	0,744

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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