European Commission

Eurocodes

Background and Applications

Design of **Steel Buildings**

with worked examples

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Joint Research Centre
European Convention for Constructional Steelwork
European Committee for Standardization
CEN/TC250/SC3

EN 1993-1-2 Resistance of members and connections to fire

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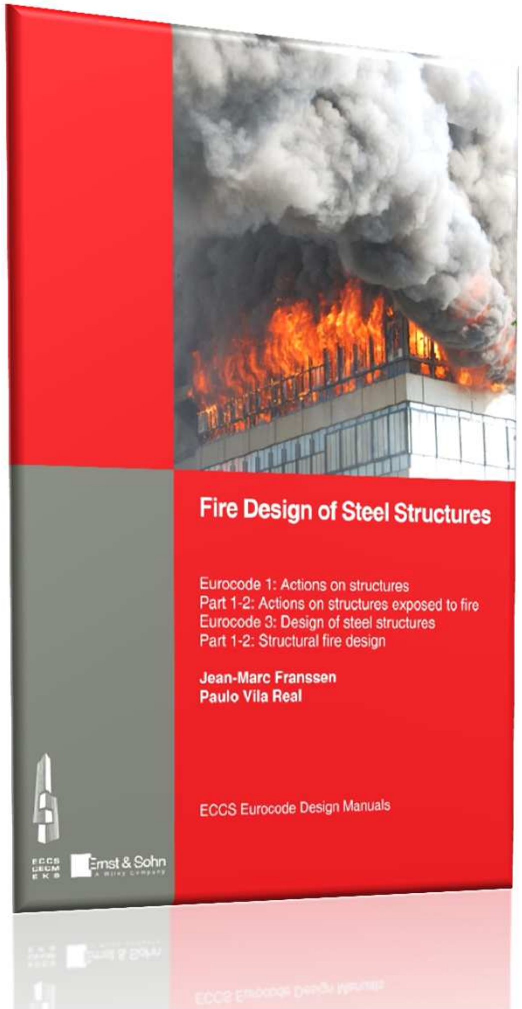
Joint Research Centre

Summary

- Introduction
- Thermal actions
- Mechanical actions
- Thermal analysis – temperature development in the members
- Mechanical analysis – fire resistance of members and connections



Introduction
Two type of regulations or standards

Each country has its own regulations for fire safety of buildings where the requirements for fire resistance are given

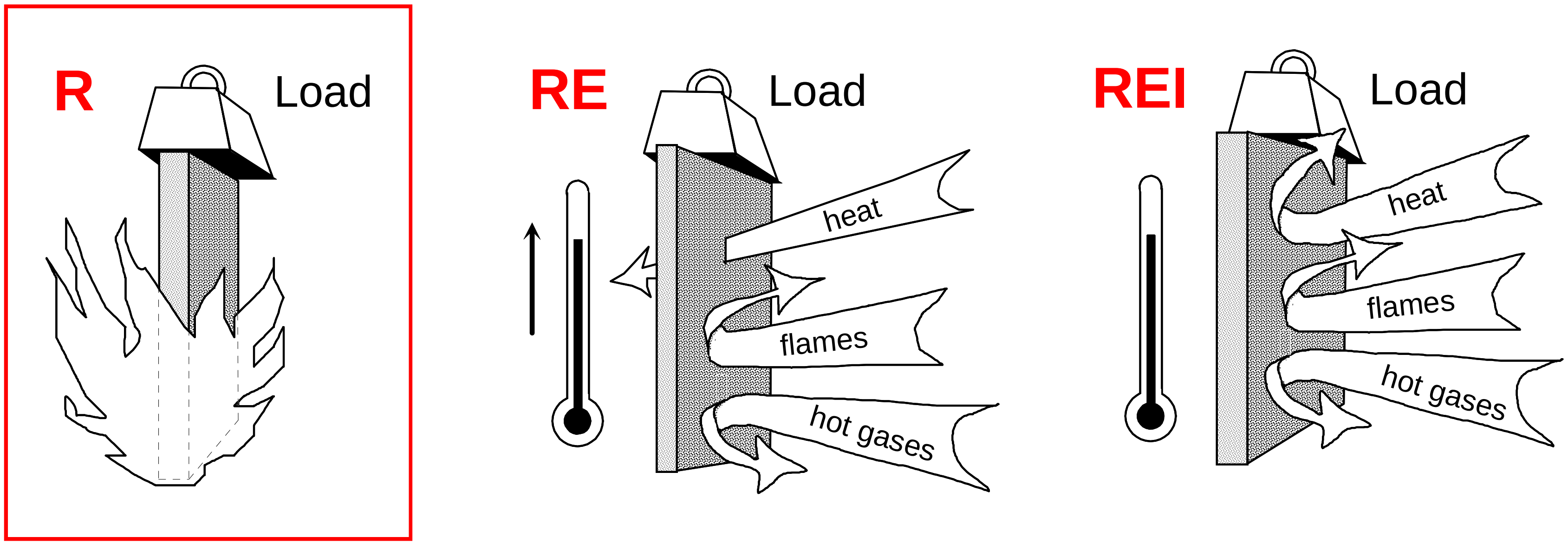
Standards for checking the structural fire resistance of buildings - in Europe the structural EUROCODES. For fire design of steel structures:

EN 1990	**Basis of structural design**
EN 1991-1-2	**Actions on structures exposed to fire**
EN 1993-1-2	**Design of steel structures – Structural fire design**

Introduction

Fire resistance - classification criteria

R – Load bearing criterion; E – Integrity criterion; I – Insulation criterion



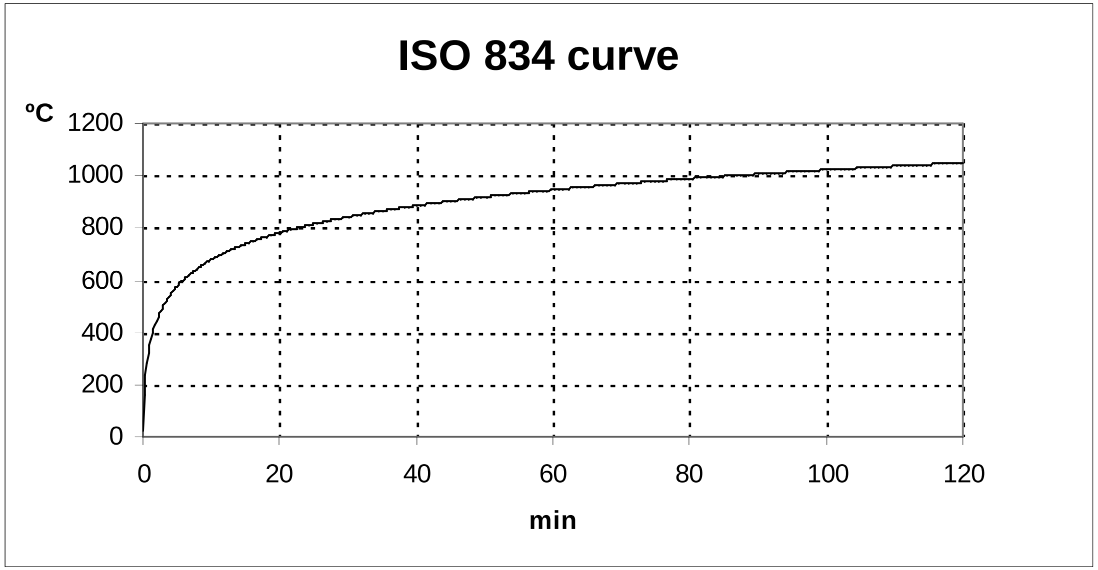
- **Load bearing only: mechanical resistance (criterion R)**
- **Load bearing and seprarting: criteria R, E and when requested, I**

Introduction

Standard Fire Resistance – Criteria R, E and I

Fire resistance is the time since the beginning of the standard fire curve ISO 834 until the moment that the element doesn't fulfil the functions for what it has been designed (Load bearing and/or separating functions)

$$T = 345\log_{10}(8t+1)+20$$

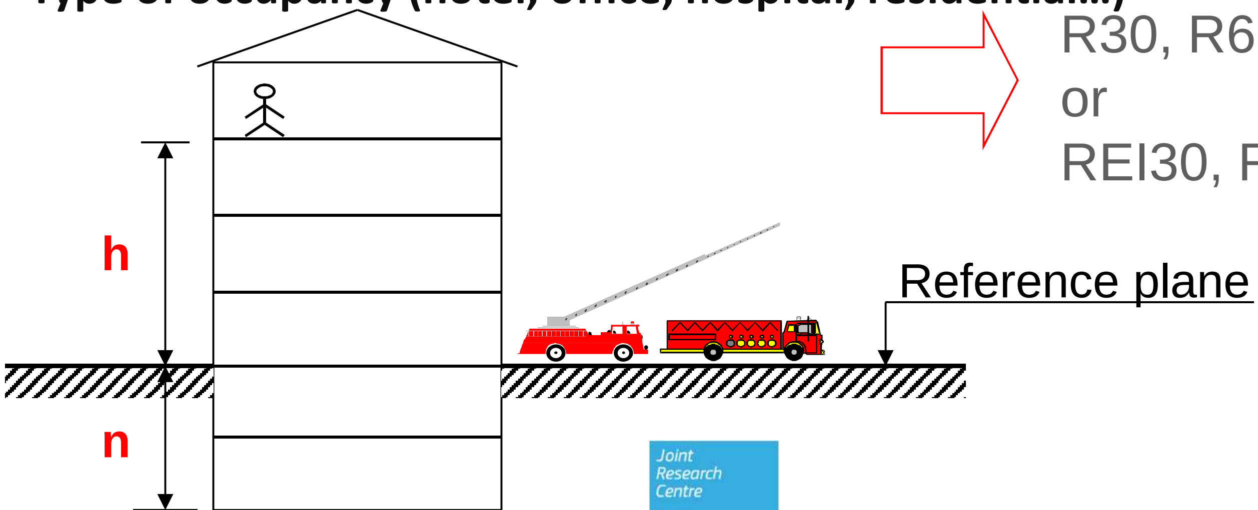


Introduction

Regulations for fire safety of buildings

- **Normally the risk factors are:**
  - **Height of the last occupied storey in the building (h) over the reference plane**
  - **Number of storeys below the reference plane (n)**
  - **Total gross floor area**
  - **Number of occupants (effective)**
  - **Type of occupancy (hotel, office, hospital, residential…)**

R30, R60, R90, ...
or
REI30, REI60. REI90, ...

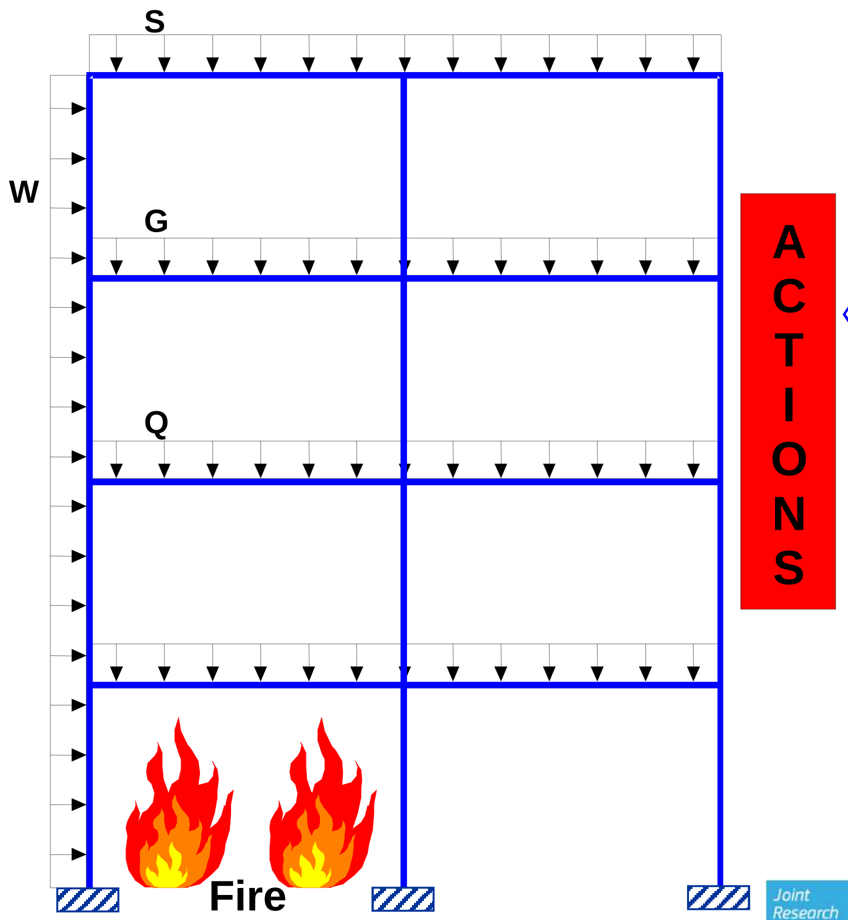


Fire Design of Steel Structures
Four steps

1. **Definition of the thermal loading - EC1**
2. **Definition of the mechanical loading - EC0 +EC1**
3. **Calculation of temperature evolution within the structural members - EC3**
4. **Calculation of the mechanical behaviour of the structure exposed to fire - EC3**

Eurocode 1

Actions on Structures



Actions for temperature analysis
Thermal Action

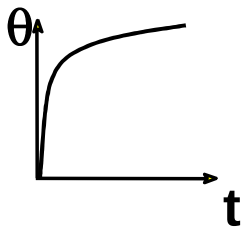
FIRE

Actions for structural analysis
Mechanical Action

Dead Load	G
Imposed Load	Q
Snow	S
Wind	W

EN 1991-1-2 - Actions on structures exposed to fire

Nominal temperature-time curves

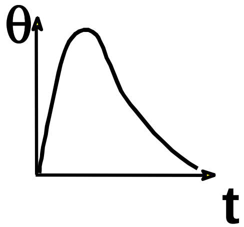


Standard temperature-time curve
External fire curve
Hydrocarbon curve

Natural fire models

Simplified fire models

Compartment fires - Parametric fire
Localised fires – Heskestad or Hasemi

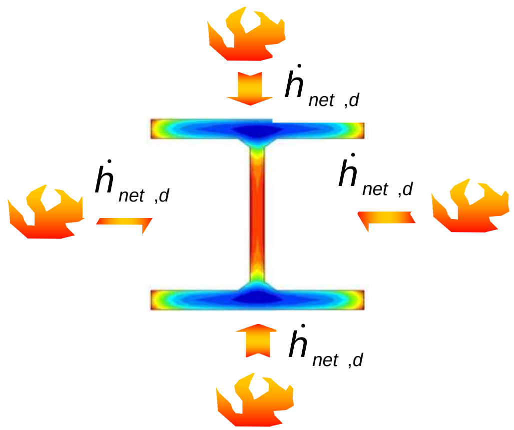


Advanced fire models

Two-Zones or One-Zone fire or a combination
CFD – Computational Fluid Dynamics

Thermal actions

Heat transfer at surface of building elements



$$\dot{h}_{net,d} = \dot{h}_{net,c} + \dot{h}_{net,r}$$

Total net heat flux

$$\dot{h}_{net,c} = \alpha_c(\theta_g - \theta_m)$$

Convective heat flux

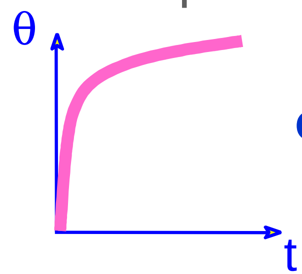
$$\dot{h}_{net,r} = \Phi \cdot \varepsilon_f \cdot \varepsilon_m \cdot \sigma \cdot [(\theta_r + 273)^4 - (\theta_m + 273)^4]$$

Radiative heat flux

$$\theta_g \approx \theta_r$$

Temperature of the fire compartment

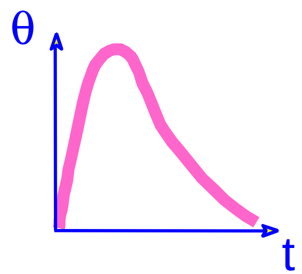
Prescriptive



Nominal fire

or

Performance-based



Natural fire

Fire Design of Steel Structures

Four steps

1. **Definition of the thermal loading - EC1**
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4. **Calculation of the mechanical behaviour of the structure exposed to fire - EC3**

Combination Rules for Mechanical Actions

EN 1990: Basis of Structural Design

- At room temperature (**20 ºC**)

$$\sum_{j\geq 1} \gamma_{G,j} G_{k,j} + \gamma_{Q,1} \cdot Q_{k,1} + \sum_{i>1} \gamma_{Q,1} \psi_{0,i} \cdot Q_{k,i}$$

- In fire situation (fire is na accidental action)

$$\sum_{j\geq 1} G_{k,1} + (\psi_{1,1} \text{ ou } \psi_{2,1}) \cdot Q_{k,1} + \sum_{i>1} \psi_{2,i} \cdot Q_{k,i} + A_d$$

$\psi_{1,1} Q_{k,1}$ – **Frequent value** of the representative value of the variable action Q_1

$\psi_{2,1} Q_{k,1}$ – **Quasi-permanent value** of the representative value of the variable action Q_1

A_d – Indirect thermal action due to fire induced by the restrained thermal expansion may be neglected for member analysis

Combination Rules for Mechanical Actions

$$\sum_{j\geq 1} G_{k,1} + (\psi_{1,1} \text{ ou } \psi_{2,1}) \cdot Q_{k,1} + \sum_{i>1} \psi_{2,i} \cdot Q_{k,i} + A_d$$

Action	Ψ_1	Ψ_2
Imposed loads in buildings, category (see EN 1991-1-1)	0.5	0.3
Imposed loads in congregation areas and shopping areas	0.7	0.6
Imposed loads in storage areas	0.9	0.8
vehicle weight $\leq$ 30 kN	0.7	0.6
30 kN $\leq$ vehicle weight $\leq$ 160 kN	0.5	0.3
Imposed loads in roofs	0.0	0.0
Snow (Norway, Sweden …)	0.2	0.0
Wind loads on buildings	0.2	0.0

In some countries the National Annex recommends Ψ_1,Q_1, so that wind is always considered and so horizontal actions are always taken into account

Member analysis - Simplified rule

Instead of using:

$$\sum_{j\geq 1} G_{k,1} + (\psi_{1,1} \text{ ou } \psi_{2,1}) \cdot Q_{k,1} + \sum_{i>1} \psi_{2,i} \cdot Q_{k,i} + A_d$$

The effect of actions can be obtained from:

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

Design value of the effect of actions at normal temperature

where

$$\eta_{fi} = \frac{\gamma_{GA} G_k + \psi_{1,1} Q_{k,1}}{\gamma_G G_k + \gamma_{Q,1} Q_{k,1}}$$

Load combination in fire situation

Load combination at 20 ºC

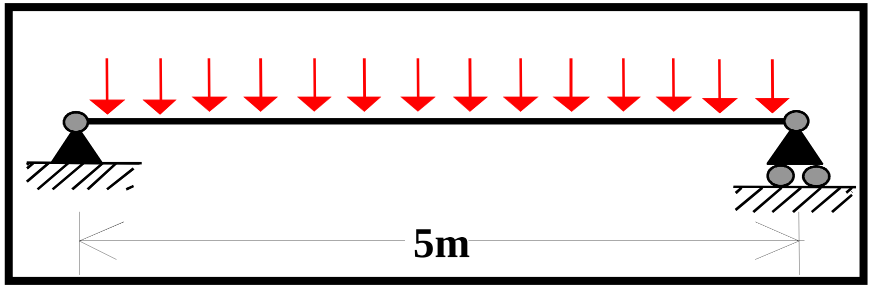
η_{fi}, as a simplification can be taken as 0.65

Member analysis - Simplified rule (example)

Characteristic loading (kN/m):

Permanent $G_k = 11.82$

Variablel $Q_{k,1} = 22.8$



- At normal temperature

$\gamma_G G_k + \gamma_{Q.1} Q_1 = 1.35x11.82 + 1.5x22.8 = 50.16$ kN/m

Design value of the moment at 20 ºC:

$M_{Ed} = 50.16x5^2/8$

$= 156.75$ kNm

- In fire situation

$G_k + \Psi_{1.1} Q_1 = 11.82 + 0.5x22.8 = 23.22$ kN/m

Design value of the moment in fire situation:

$M_{fi,Ed} = 23.22x5^2/8$

$= 72.6$ kNm

or

$$\eta_{fi} = \frac{\gamma_{GA} G_k + \psi_{1,1} Q_{k,1}}{\gamma_G G_k + \gamma_{Q,1} Q_{k,1}}$$

EC3

$\eta_{fi} = 0.65$

On the safe side but:

+ 40%

$\eta_{fi} = 23.22 / 50.16 = 0.463$

$M_{fi.Ed} = \eta_{fi} M_{Ed}$

$= 0.463 \times 156.75 = 72.6$ kNm

Joint Research Centre

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Thermal response

Heat conduction equation

$$\frac{\partial}{\partial x}\left(\lambda\frac{\partial\theta}{\partial x}\right)+\frac{\partial}{\partial y}\left(\lambda\frac{\partial\theta}{\partial y}\right)+\dot{Q}=\rho c_p\frac{\partial\theta}{\partial t}$$

Boundary conditions

$$q_c = h_c(\theta-\theta_\infty)$$ **convection**

$$q_\mathrm{r} = \sigma\varepsilon(\theta^4-\theta_a^4) = \underbrace{\sigma\varepsilon(\theta^2+\theta_a^2)(\theta+\theta_a)}_{h_r}(\theta-\theta_a) = h_r(\theta-\theta_a)$$ **radiation**

Note: this equation can be simplified for the case of current steel profiles

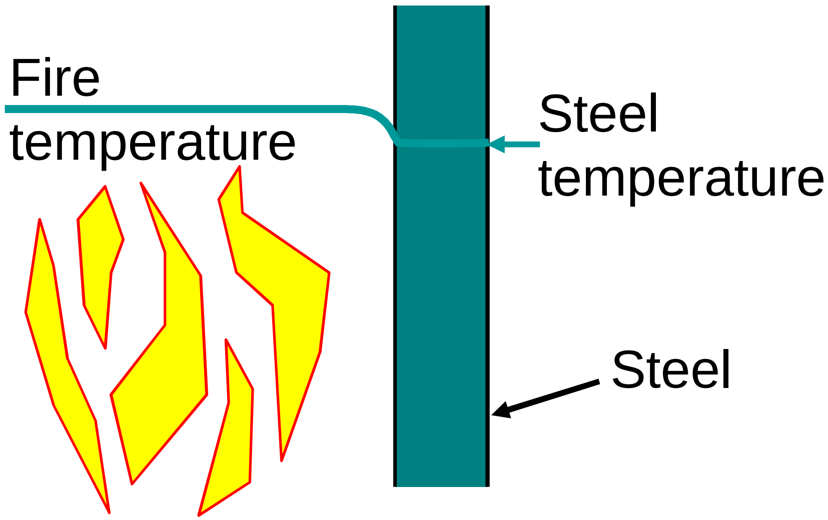
Temperature increase of unprotected steel

Simplified equation of EC3

Temperature increase in time step Δt:

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

Heat flux $\dot{h}_{net,d}$ has 2 parts:



Radiation:

$$\dot{h}_{net,r} = 5{,}67 \times 10^{-8} \Phi \varepsilon_f \varepsilon_m \left((\theta_r + 273)^4 - (\theta_m + 273)^4 \right)$$

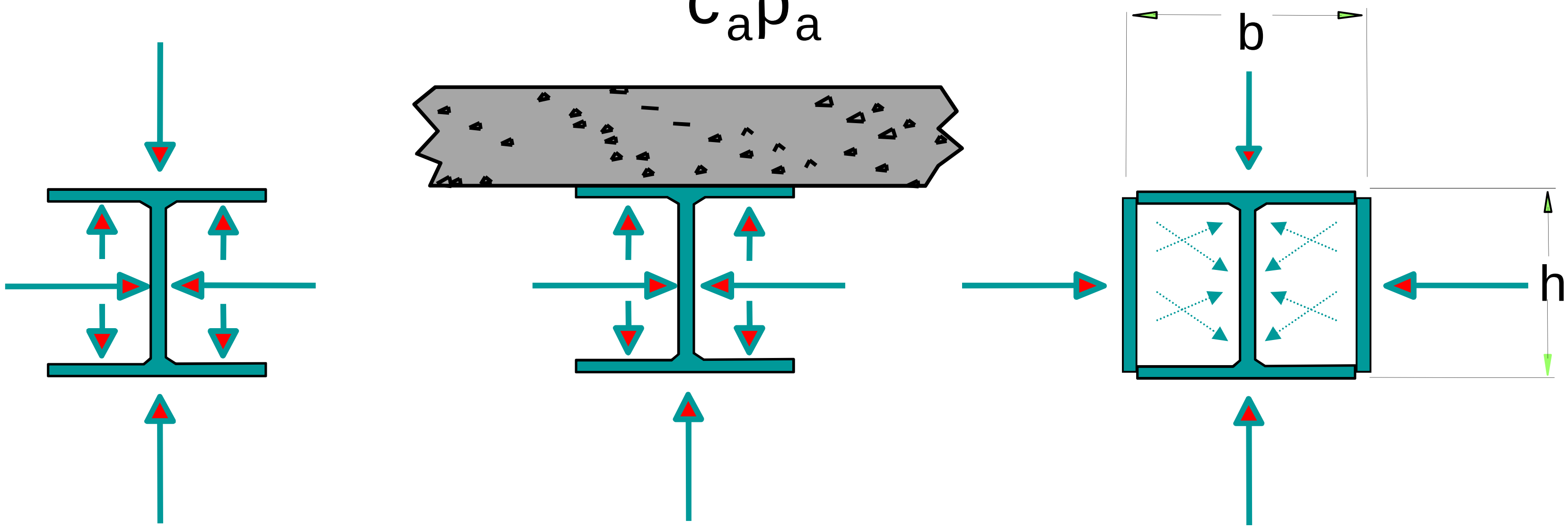
Convection:

$$\dot{h}_{net,c} = \alpha_c (\theta_g - \theta_m)$$

Section factor A_m/V

Unprotected steel members

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



$\frac{\text{perimeter}}{\text{c/s area}}$ $\frac{\text{exposed perimeter}}{\text{c/s area}}$ $\frac{2(b+h)}{\text{c/s area}}$

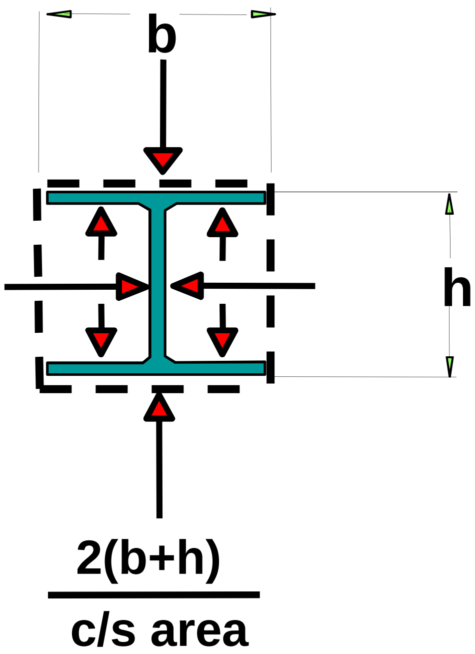
Correction factor for the shadow effect k_{sh}

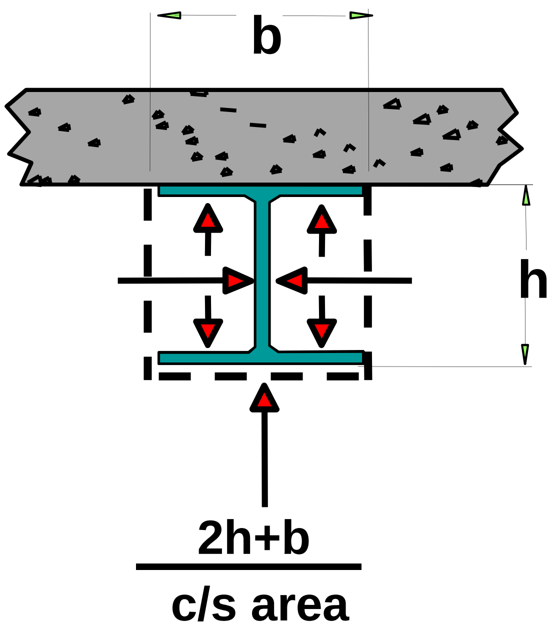
For I-sections under nominal fire: $\mathbf{k_{sh} = 0.9\ [A_m/V]_b/[A_m/V]}$

In all other cases: $\mathbf{k_{sh} = [A_m/V]_b/[A_m/V]}$

For cross-sections with convex shape: $\mathbf{k_{sh} = 1}$

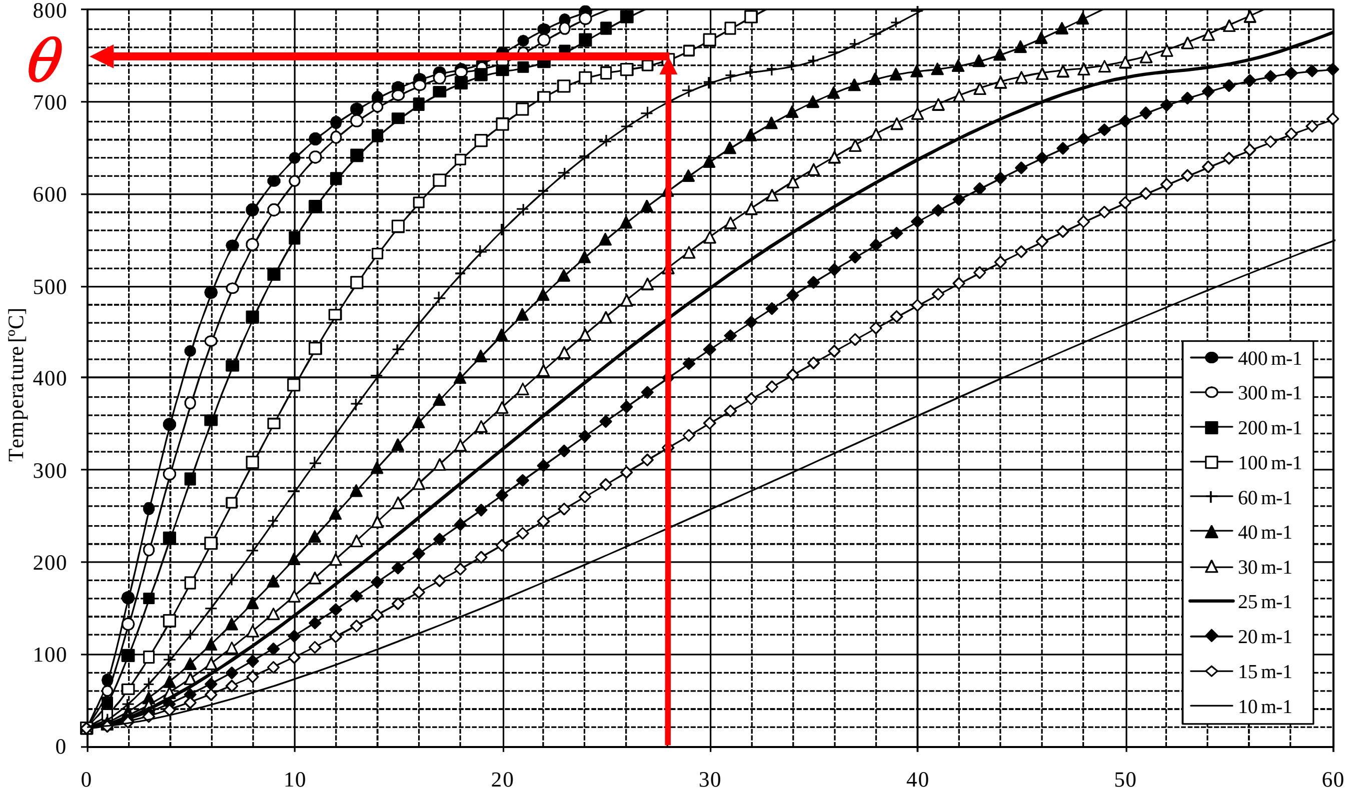
$[A_m/V]_b$ - Section factor as the profile has a hollow encasement fire protection





Nomogram for temperature of unprotected steel profiles

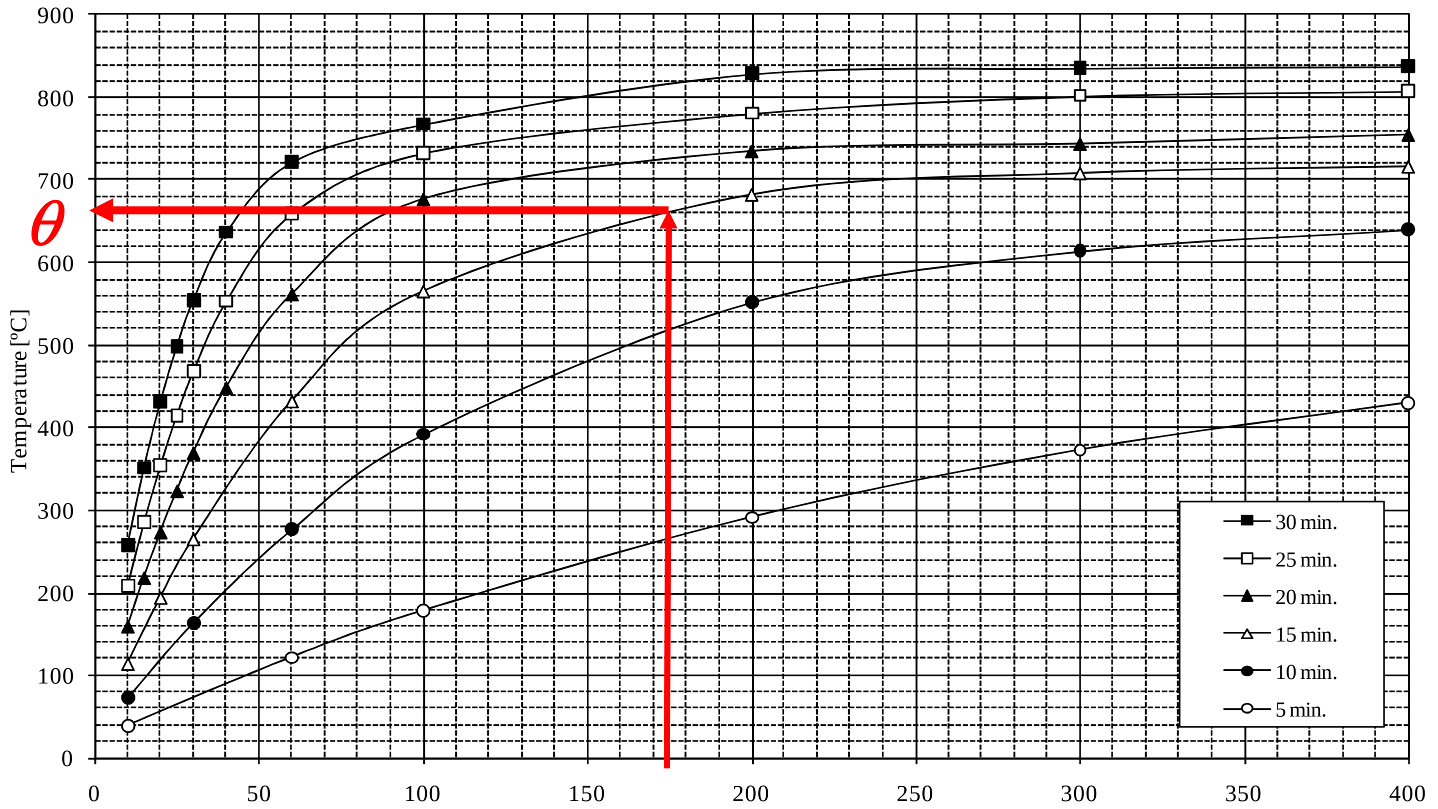
Nomogram for unprotected steel members subjected to the ISO 834 fire curve, for different values of $k_{sh} \cdot Am/V$ [m^{-1}]



JEAN-MARC FRANSSEN; PAULO VILA REAL – "Fire Design of Steel Structures", ECCS Press, ISBN 978-92-9-147-099-0 and Ernst & Sohn a Wiley Company, ISBN 978-3-433-02974-9, April 2010.

Nomogram for temperature of unprotected steel profiles

Nomogram for unprotected steel members subjected to the ISO 834 fire curve, for different time duration



Modified section factor $k_{sh} \cdot A_m/V$ [m^{-1}]

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ISBN 978-92-9-147-099-0 and Ernst & Sohn a Wiley Company, ISBN 978-3-433-02974-9, April 2010.

Table for evaluating the temperature

Unprotected steel members

Temperature of unprotected steel in ºC, exposed to the ISO 834 fire curve

for different values of $k_{sh} \dfrac{A_m}{V}$, $[\mathrm{m}^{-1}]$ (continued)

Time [min.]	10 m^{-1}	15 m^{-1}	20 m^{-1}	25 m^{-1}	30 m^{-1}	40 m^{-1}	60 m^{-1}	100 m^{-1}	200 m^{-1}	300 m^{-1}	400 m^{-1}
51	468	600	688	732	750	825	894	911	917	918	919
52	477	610	697	734	757	835	899	915	920	921	922
53	487	620	704	736	765	845	904	918	923	924	925
54	496	629	711	739	774	854	908	921	926	927	928
55	505	638	718	743	784	863	913	924	928	930	930
56	514	648	723	747	794	872	917	927	931	932	933
57	523	656	728	753	804	880	920	930	934	935	936
58	532	665	731	760	814	887	924	933	937	938	938
59	541	673	734	768	825	894	927	935	939	940	941
60	549	681	736	777	834	901	931	93[illegible]	942	943	944

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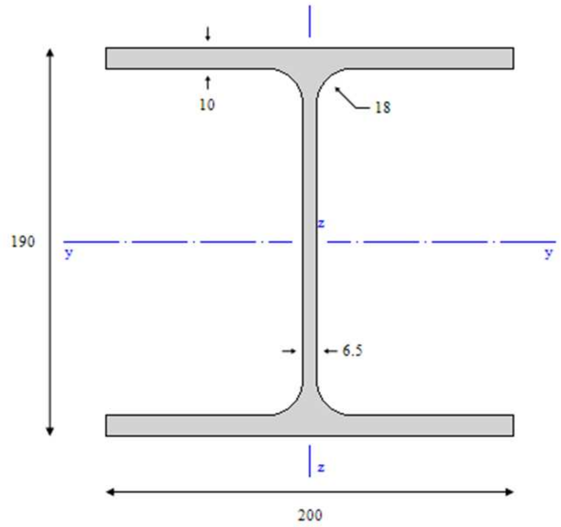
Temperature calculation in unprotected profiles - 1

Example 1

What is the temperature of an unprotected HE 200 A profile after 30 minutes of standard fire exposure (ISO 834) on four sides?

HE 200 A
Gross Section



The section factor for an HE 200 A is:

$$A_m / V = 211 \text{ m}^{-1}$$

The HE 200 A has the following geometric characteristics:

$$b = 200 \text{ mm}$$
$$h = 190 \text{ mm}$$
$$A = 53.83 \text{ cm}^2$$

and the box value of the section factor $[A_m / V]_b$ takes the value

$$[A_m / V]_b = \frac{2 \times (b+h)}{A} = \frac{2 \times (0.2 + 0.19)}{53.83 \times 10^{-4}} = 144.9 \approx 145 \text{ m}^{-1}$$

Temperature calculation in unprotected profiles - 2

Example 1

The shadow factor, k_{sh} is given by:

$$k_{sh} = 0.9[A_m / V]_b / [A_m / V] = 0.9 \cdot 144.9 / 211 = 0.618$$

Taking into account the shadow effect, the modified section factor has the value

$$k_{sh}[A_m / V] = 0.618 \cdot 211 = 130.5 \text{ m}^{-1}$$

This value should be obtained without evaluating k_{sh}.

$$k_{sh}[A_m / V] = 0.9[A_m / V]_b = 0.9 \cdot 145 = 130.5 \text{ m}^{-1}$$

$$\Delta\theta_{a,t} = k_{sh} \frac{A_m / V}{c_a \rho_a} \dot{h}_{net,d} \Delta t \quad \Rightarrow \quad \theta_a = 802^{o}\text{C}$$

Temperature calculation in unprotected profiles - 3

Example 1

$$k_{sh}[A_m/V] = 130.5\ \text{m}^{-1}$$

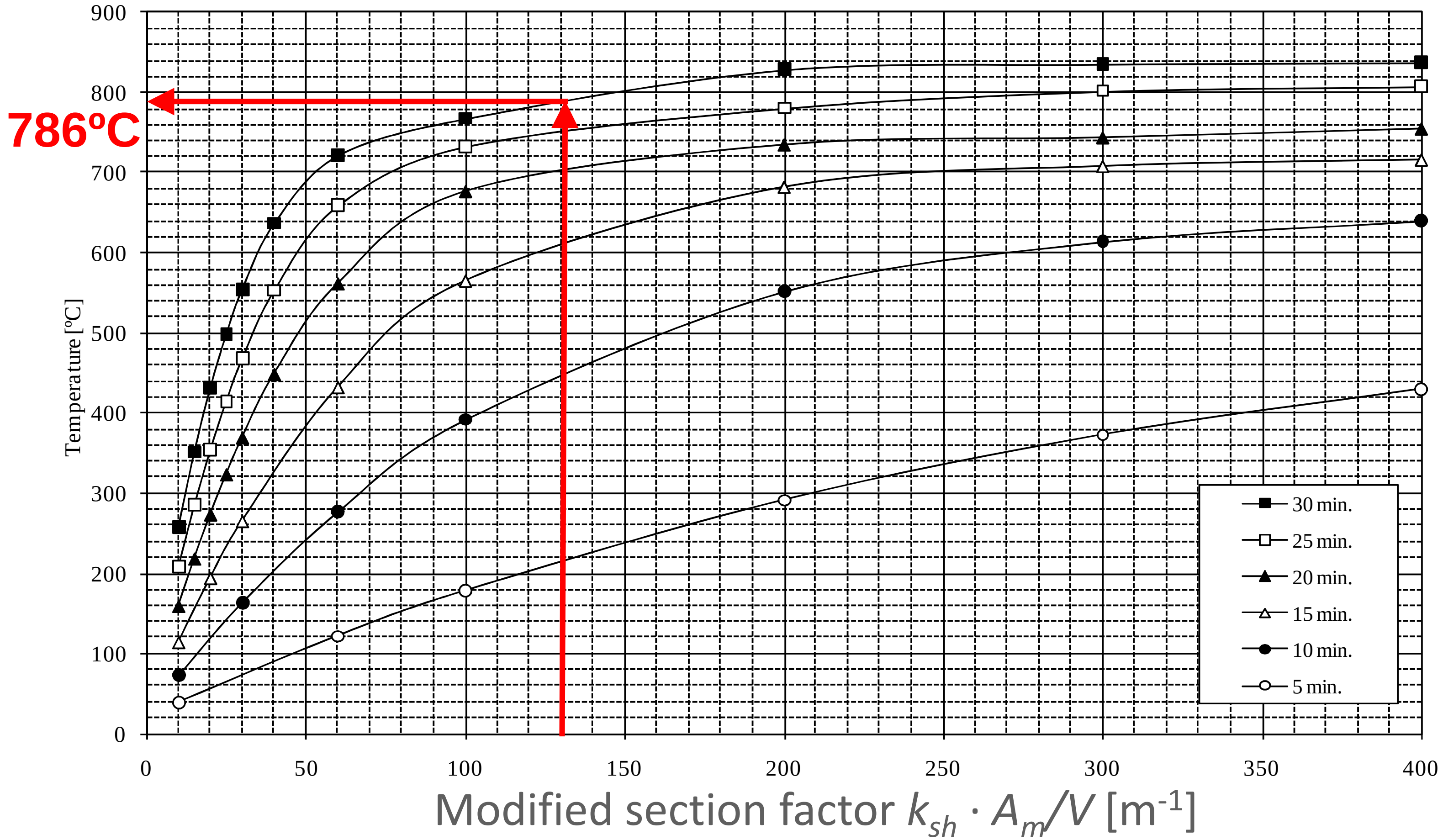
Temperature of unprotected steel in ºC, exposed to the ISO 834 fire curve for different values of $k_{sh}\dfrac{A_m}{V}$, $[\text{m}^{-1}]$ (continued)

Time [min.]	10 m^{-1}	15 m^{-1}	20 m^{-1}	25 m^{-1}	30 m^{-1}	40 m^{-1}	60 m^{-1}	100 m^{-1}	200 m^{-1}	300 m^{-1}	400 m^{-1}
24	197	271	337	396	448	532	641	726	767	791	799
25	207	284	353	414	467	552	658	732	780	801	807
26	217	298	369	432	485	570	674	735	792	809	813
27	227	311	385	449	503	588	688	739	803	816	820
28	237	324	401	466	521	604	701	746	813	823	826
29	247	338	416	482	538	621	712	756	821	829	831
30	257	351	431	498	554	636	721	767	828	835	837

786ºC

Temperature calculation in unprotected profiles - 4

Example 1



Structural passive fire protection

Insulating Board

Gypsum, Mineral fibre, Vermiculite.
Easy to apply, aesthetically acceptable.
Difficulties with complex details.

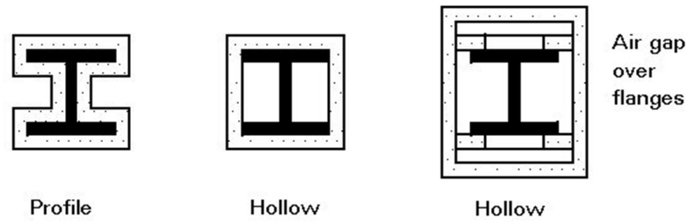
Cementitious Sprays

Mineral fibre or vermiculite in cement binder.
Cheap to apply, but messy; clean-up may be expensive.
Poor aesthetics; normally used behind suspended ceilings.

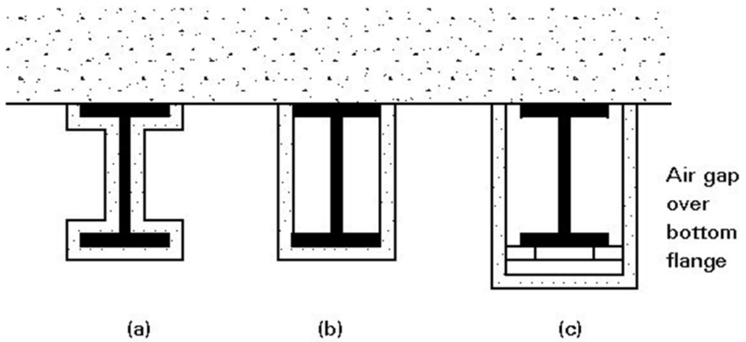
Intumescent Paints

Decorative finish under normal conditions.
Expands on heating to produce insulating layer.
Can be done off-site.

Columns:



Beams:



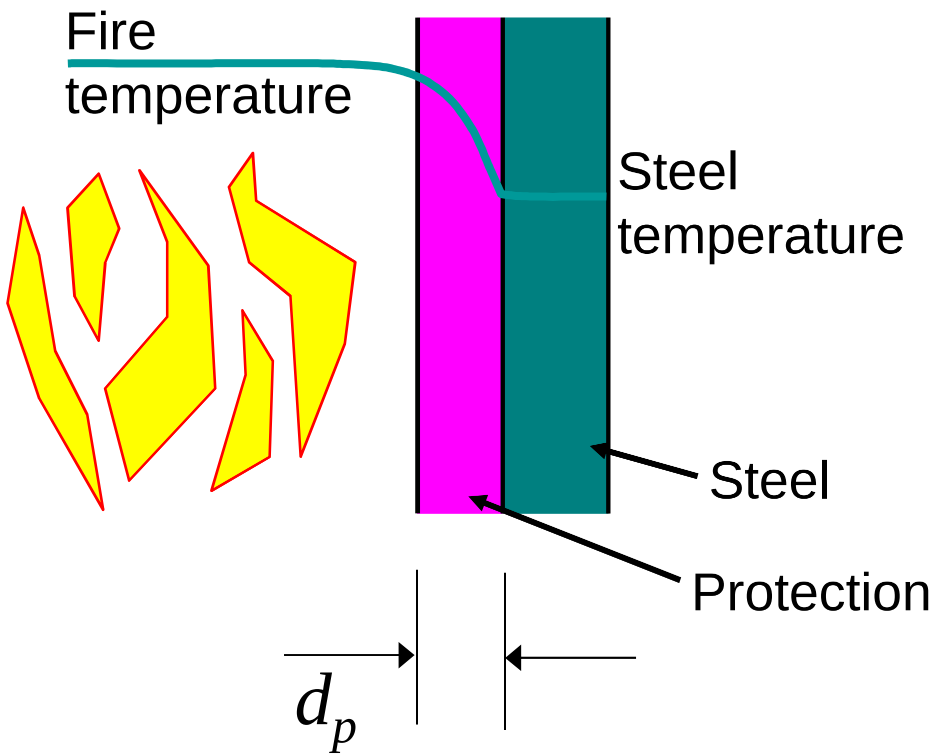
(a) - Spray or intumescent
(b) - Board
(c) - Board

Temperature increase of protected steel

Simplified equation of EC3

- **Heat stored in protection layer relative to heat stored in steel**

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



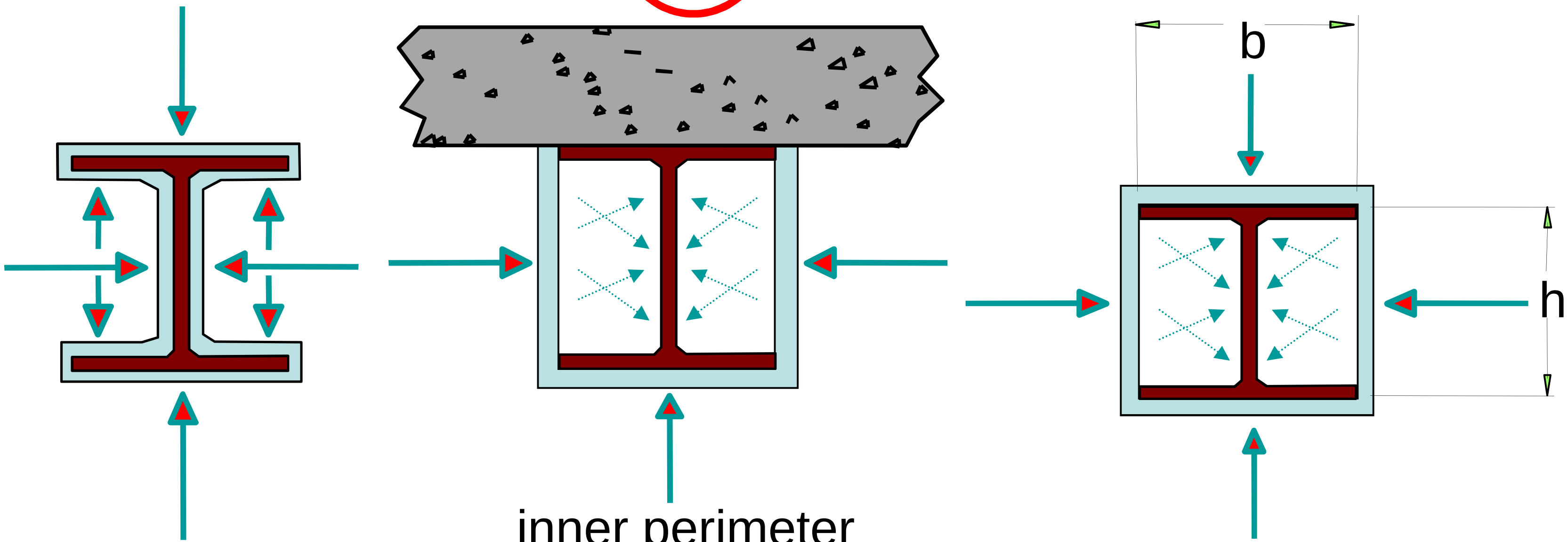
- **Temperature rise of steel in time increment Δt**

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

Section factor A_p/V

Protected steel members

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



Steel perimeter / steel c/s area

inner perimeter of board / steel c/s area

2(b+h) / c/s area

How to build nomograms

Tables for evaluating the temperature of protected steel profiles

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

This equation has too many variables to build a table. Making $\phi = 0.0$, Which correspons to a ligth protection material, it becomes:

$$\Delta\theta_{a.t} = \frac{\lambda_p / d_p}{c_a \rho_a} \frac{A_p}{V} (\theta_{g.t} - \theta_{a.t}) \Delta t$$

Now it is possible to build a two entries table. Two variables: time and the modified section factor:

$$\frac{\lambda_p}{d_p} \frac{A_p}{V}$$

Tables for evaluating the temperature

Protected steel profiles submitted to the ISO 834 curve

Temperature of protected steel in ºC, exposed to the ISO 834 fire curve

for different values of $\frac{A_p}{V}\frac{\lambda_p}{d_p}$, $[\mathrm{W/m^3K}]$

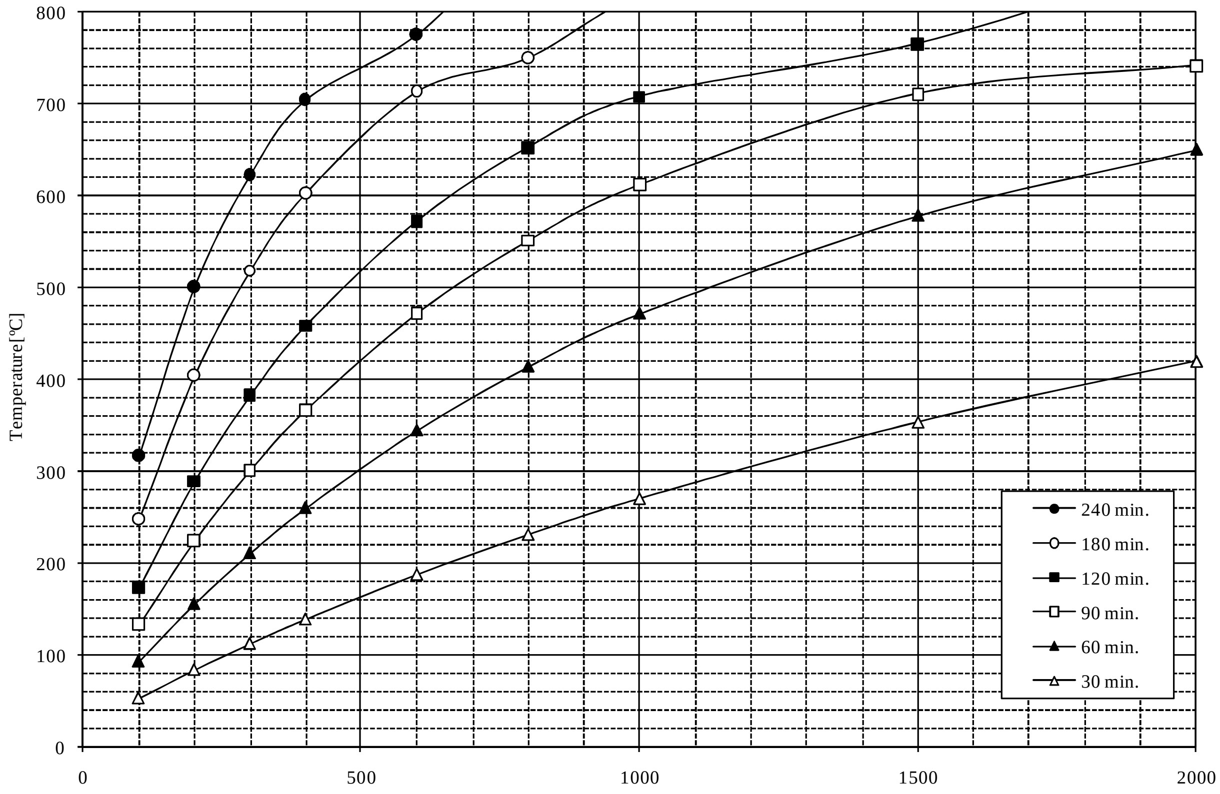
Time [min.]	100 W/m^3K	200 W/m^3K	300 W/m^3K	400 W/m^3K	600 W/m^3K	800 W/m^3K	1000 W/m^3K	1500 W/m^3K	2000 W/m^3K
0	20	20	20	20	20	20	20	20	20
5	24	27	31	35	41	48	55	71	86
10	29	38	46	54	70	85	100	133	164
15	35	49	62	75	100	123	145	194	237
20	41	61	79	97	130	160	189	251	305
25	47	72	96	118	159	197	231	305	366
30	54	84	113	140	188	232	271	354	421
35	60	97	130	161	216	266	309	400	470
40	67	109	147	181	244	298	346	442	514
45	74	121	163	202	270	329	380	481	554
50	80	133	179	222	296	359	413	516	589
55	87	145	196	241	321	387	443	549	621
60	94	156	211	261	345	414	472	578	650

Note: this table is only valid for light-weight protection material,

$$\phi \approx 0.0$$

Nomogram for temperature of protected steel profiles

Nomogram for protected steel members subjected to the ISO 834 fire curve, for different time duration



Modified section factor $[A_p/V][\lambda_p/d_p]$ [W/m³K]

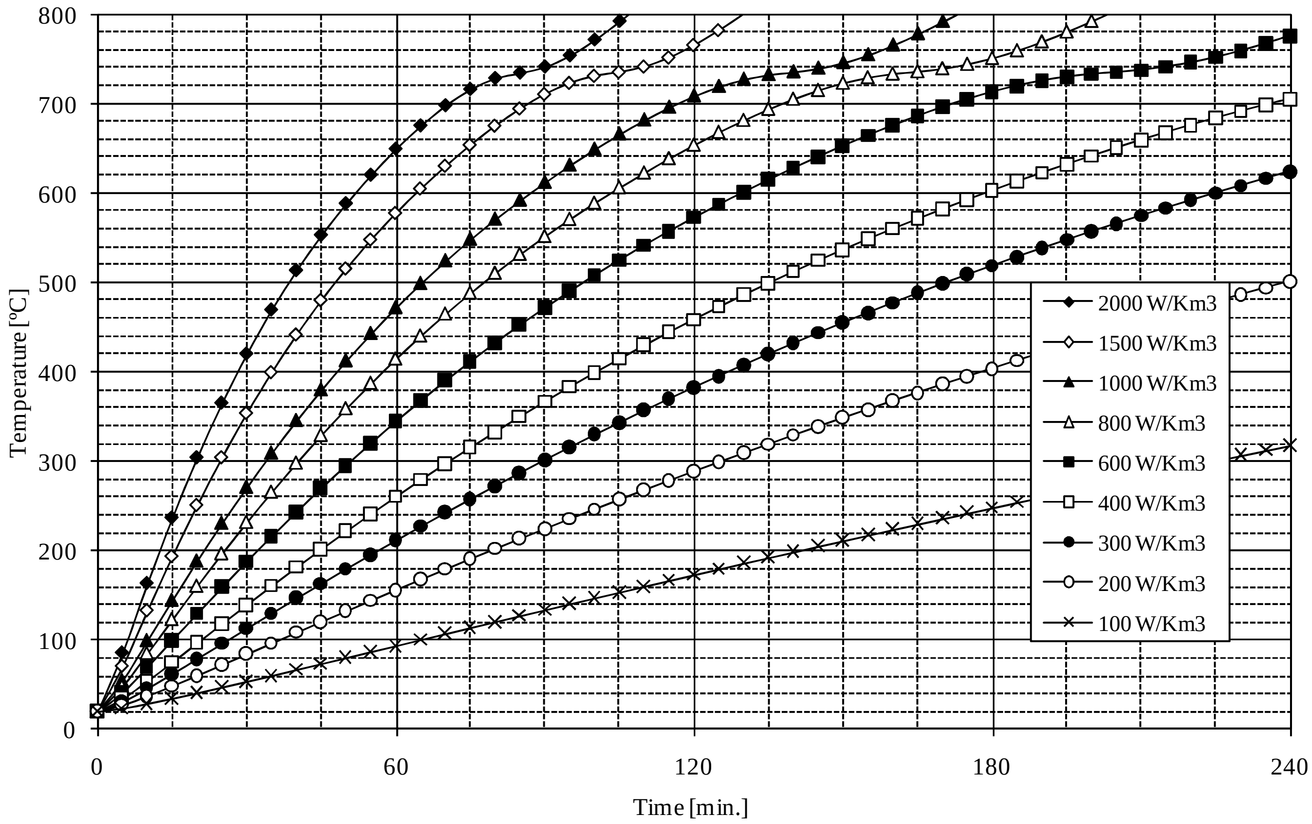
Note: this chart is only valid for light-weight protection material,

$$\phi \approx 0.0$$

For heavy-weight materials it should be used a corrected modified section factor

Nomogram for temperature of protected steel profiles

Nomogram for protected steel members subjected to the ISO 834 fire curve, for different values of $[A_p/V][\lambda_p/d_p]$ [W/Km³]



Note: this chart is only valid for light-weight protection material,

$$\phi \approx 0.0$$

For heavy-weight materials it should be used a corrected modified section factor

Light-weight protection materials

Light-weight materials are the ones with a thermal capacity $d_p A_p c_p \rho_p$ less than one half of the thermal capacity of the steel $c_a \rho_a V$

$$d_p A_p c_p \rho_p < c_a \rho_a V / 2 \quad \Rightarrow \quad \phi = \frac{c_p d_p \rho_p}{c_a \rho_a} \cdot \frac{A_p}{V} < 0.5$$

For these materials we can make $\phi = 0$

$$\phi = \frac{c_p d_p \rho_p}{c_a \rho_a} \cdot \frac{A_p}{V} \approx 0$$

Heavy-weight protection materials

Tables and nomograms for protected profiles where obtained for light-weight materials ($\phi = 0$), only using the modified section factor

$$\frac{A_p}{V} \cdot \frac{\lambda_p}{d_p}$$

For heavy-weight material the tables and the nomograms can be used but the modified section factor should be corrected using

$$\frac{A_p}{V} \cdot \frac{\lambda_p}{d_p} \cdot \left(\frac{1}{1+\phi/2} \right)$$

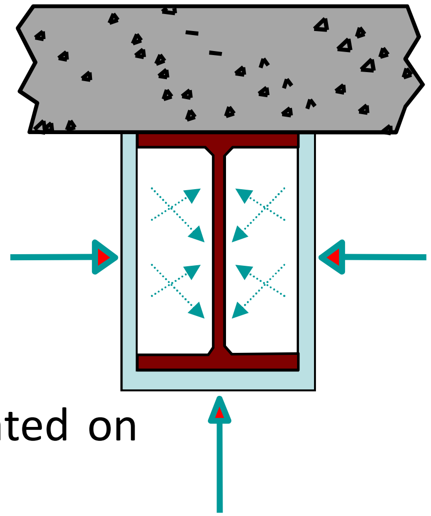
Thermal properties of fire protection materials

Material	Unit mass, ρ_p [kg / m^3]	Moisture content, p %	Thermal conductivity, λ_p [W / (mK)]	Specific heat, c_p [J/(kgK)]
Spays				
- mineral fibre	300	1	0.12	1200
- vermiculite cement	350	15	0.12	1200
- perlite	350	15	0.12	1200
High density sprays				
- vermiculite (or perlite) and cement	550	15	0.12	1100
- vermiculite (or perlite) and gypsum	650	15	0.12	1100
Boards				
- vermiculite (or perlite) and cement	800	15	0.20	1200
- fibre-silicate or fibre-calcium -silicate	600	3	0.15	1200
- fibre-cement	800	5	0.15	1200
- gypsum boards	800	20	0.20	1700
Compressed fiber boards				
- fibre silicate, mineral-wool, stone-wool	150	2	0.20	1200
Concrete	2300	4	1.60	1000
Light weight concrete	1600	5	0.80	840
Concrete bricks	2200	8	1.00	1200
Brick with holes	1000	-	0.40	1200
Solid bricks	2000	-	1.20	1200

Temperature calculation in protected profiles - 1

Example 2

What is the thickness of fibre-cement board encasement for an IPE 300 heated on three sides to be classified as R90 if the critical temperature is 654°C?



The fibre-cement has the following thermal properties:

$$\lambda_p = 0.15 \text{ W/(m·K)}$$

$$c_p = 1200 \text{ J/(kgK)}$$

$$\rho_p = 800 \text{ kg/m}^3$$

The massivity factor for the IPE 300 with hollow encasement heated on three sides is:

$$A_p / V = 139.4 \text{ m}^{-1}$$

Temperature calculation in protected profiles - 2

Example 2

By interpolation in Table for protected profiles or using a nomogram, for a temperature of 654ºC, at 90 minutes of standard fire exposure, the modified section factor is:

$$\frac{A_p}{V} \cdot \frac{\lambda_p}{d_p} \leq 1210 \text{ W/(m}^3 \cdot \text{K)}$$

Temperature calculation in protected profiles - 3

Example 2

Temperature of protected steel in °C, exposed to the ISO 834 fire curve

for different values of $\frac{A_p}{V}\frac{\lambda_p}{d_p}$, $[\mathrm{W/m^3K}]$

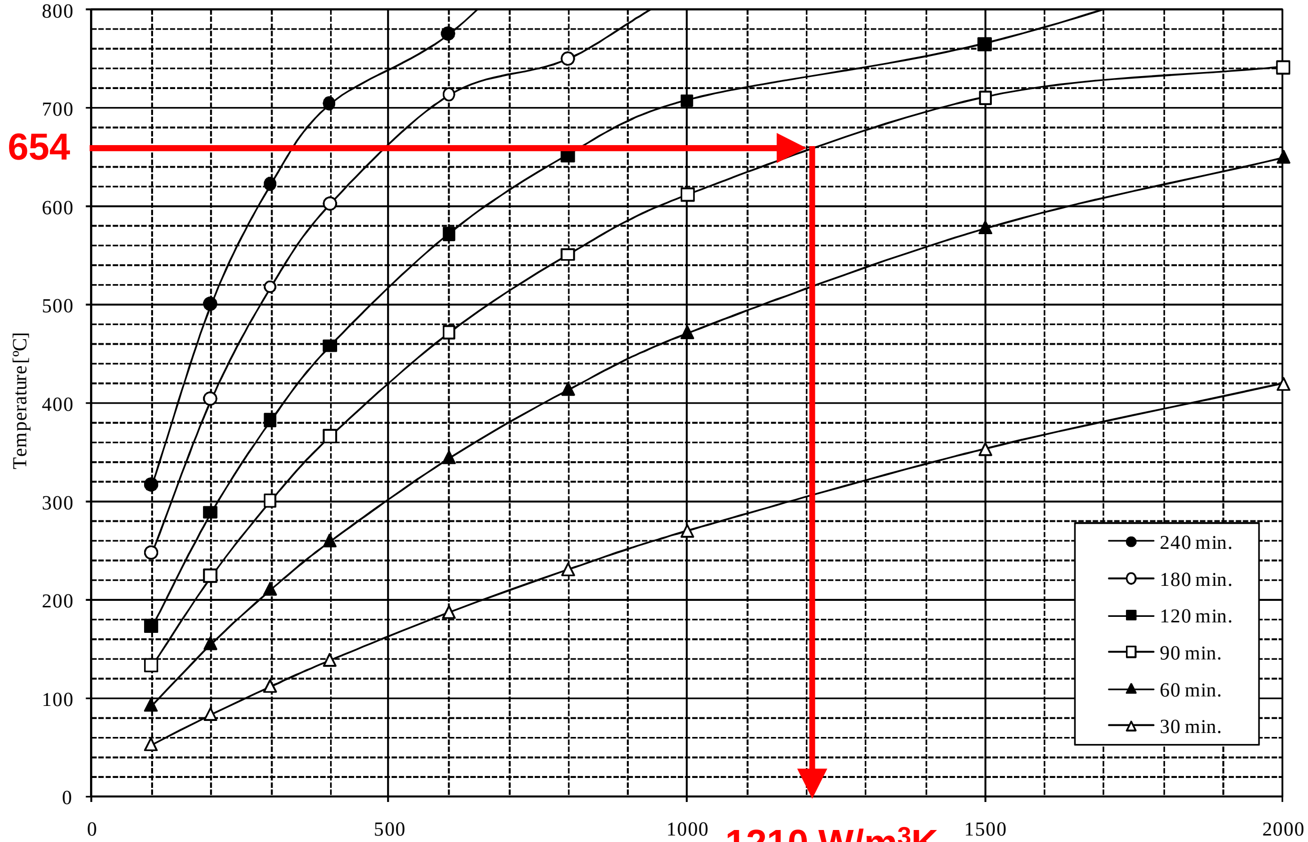
1210 W/m³K

Time [min.]	100 W/m³K	200 W/m³K	300 W/m³K	400 W/m³K	600 W/m³K	800 W/m³K	1000 W/m³K	1500 W/m³K	2000 W/m³K
0	20	20	20	20	20	20	20	20	20
5	24	27	31	35	41	48	55	71	86
10	29	38	46	54	70	85	100	133	164
15	35	49	62	75	100	123	145	194	237
20	41	61	79	97	130	160	189	251	305
25	47	72	96	118	159	197	231	305	366
30	54	84	113	140	188	232	271	354	421
35	60	97	130	161	216	266	309	400	470
40	67	109	147	181	244	298	346	442	514
45	74	121	163	202	270	329	380	481	554
50	80	133	179	222	296	359	413	516	589
55	87	145	196	241	321	387	443	549	621
60	94	156	211	261	345	414	472	578	650
65	100	168	227	279	368	440	499	606	676
70	107	180	242	298	391	465	525	631	699
75	114	191	258	316	412	488	549	655	717
80	120	202	273	333	433	510	571	676	729
85	127	214	287	350	453	531	592	695	735
90	134	225	302	367	472	552	612	712	742

654

Temperature calculation in protected profiles - 4

Example 2



Modified section factor $[A_p/V][\lambda_p/d_p]$ [W/m³K]

Temperature calculation in protected profiles - 5

Example 2

The thickness should satisfy

$$\frac{A_p}{V}\cdot\frac{\lambda_p}{d_p}\leq 1210 \qquad \Rightarrow \qquad d_p \geq \frac{A_p/V}{1210}\lambda_p = \frac{139.4}{1210}\cdot 0.15 = 0.017\,\text{m} = 17\,\text{mm}$$

This thickness can be corrected if the amount of heat stored in the protection, ϕ, is taken into account, according as it was shown and using the following expression to obtain the corrected thickness.

$$\frac{A_p}{V}\cdot\frac{\lambda_p}{d_p}\cdot\frac{1}{1+\phi/2}\leq 1210\ \text{W/(m}^3\cdot\text{K)}$$

The following iterative procedure is needed to evaluate the corrected thickness:

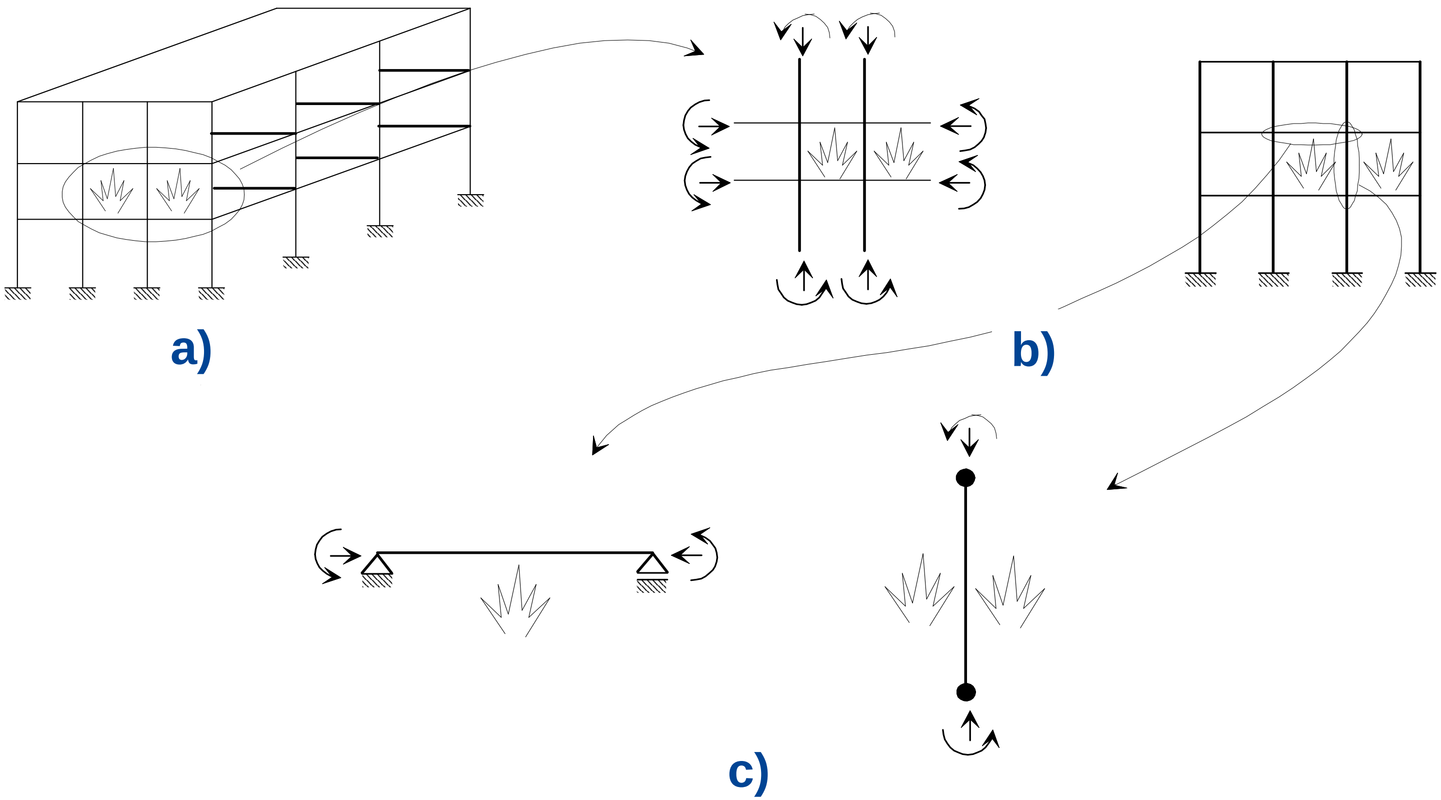
d_p	$\phi = \frac{c_p d_p \rho_p}{c_a \rho_a}\cdot\frac{A_p}{V}$	$d_p \geq \frac{A_p}{V}\cdot\frac{\lambda_p}{1210}\cdot\frac{1}{1+\phi/2}$
0.017	$\frac{1200\cdot 0.017\cdot 800}{600\cdot 7850}\cdot 139$	0.0139
0.0139	$\frac{1200\cdot 0.0139\cdot 800}{600\cdot 7850}\cdot 139$	0.0144
0.0144	$\frac{1200\cdot 0.0144\cdot 800}{600\cdot 7850}\cdot 139$	0.0143
0.0143	$\frac{1200\cdot 0.0143\cdot 800}{600\cdot 7850}\cdot 139$	0.0143

-15%

Fire Design of Steel Structures
Four steps

1. **Definition of the thermal loading - EC1**
2. **Definition of the mechanical loading - EC0 +EC1**
3. **Calculation of temperature evolution within the structural members - EC3**
4. **Calculation of the mechanical behaviour of the structure exposed to fire - EC3**

Degree of simplification of the structure

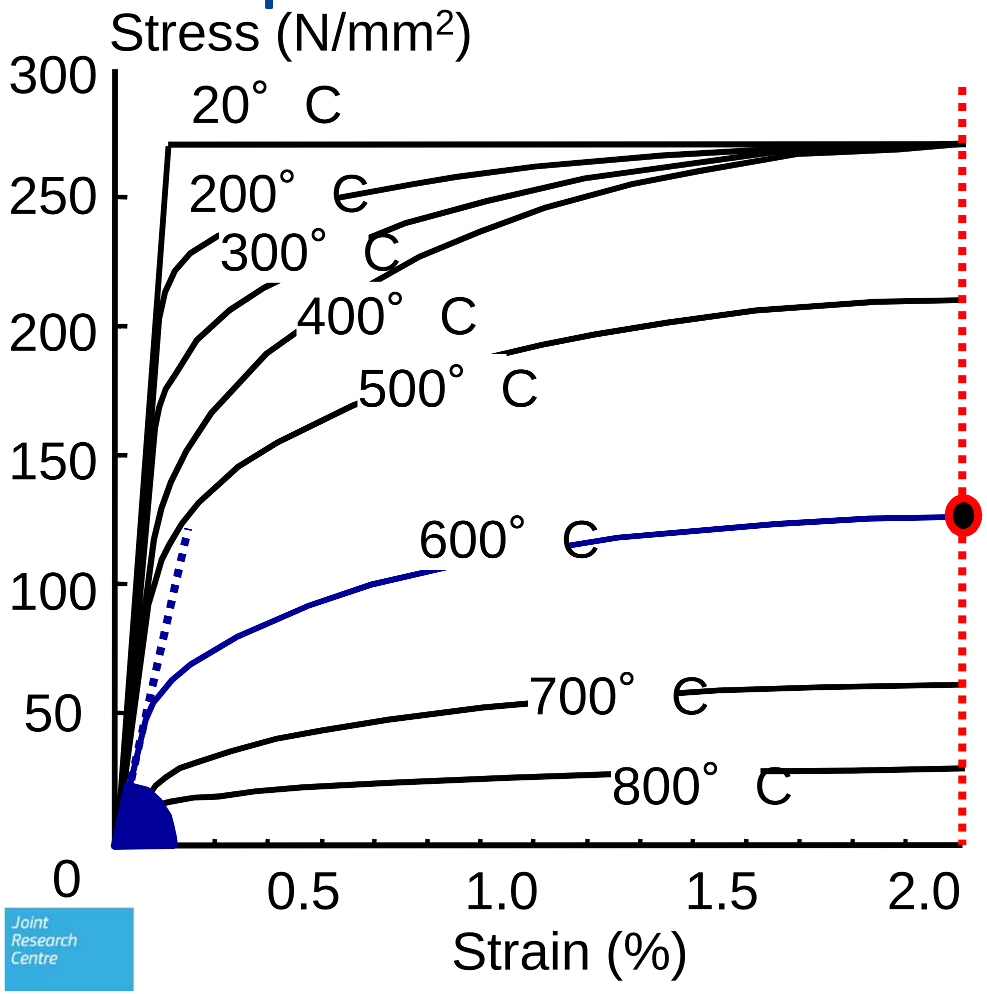


Analysis of: a) Global structure; b) Parts of the structure; c) Members

Mechanical properties of carbon steel

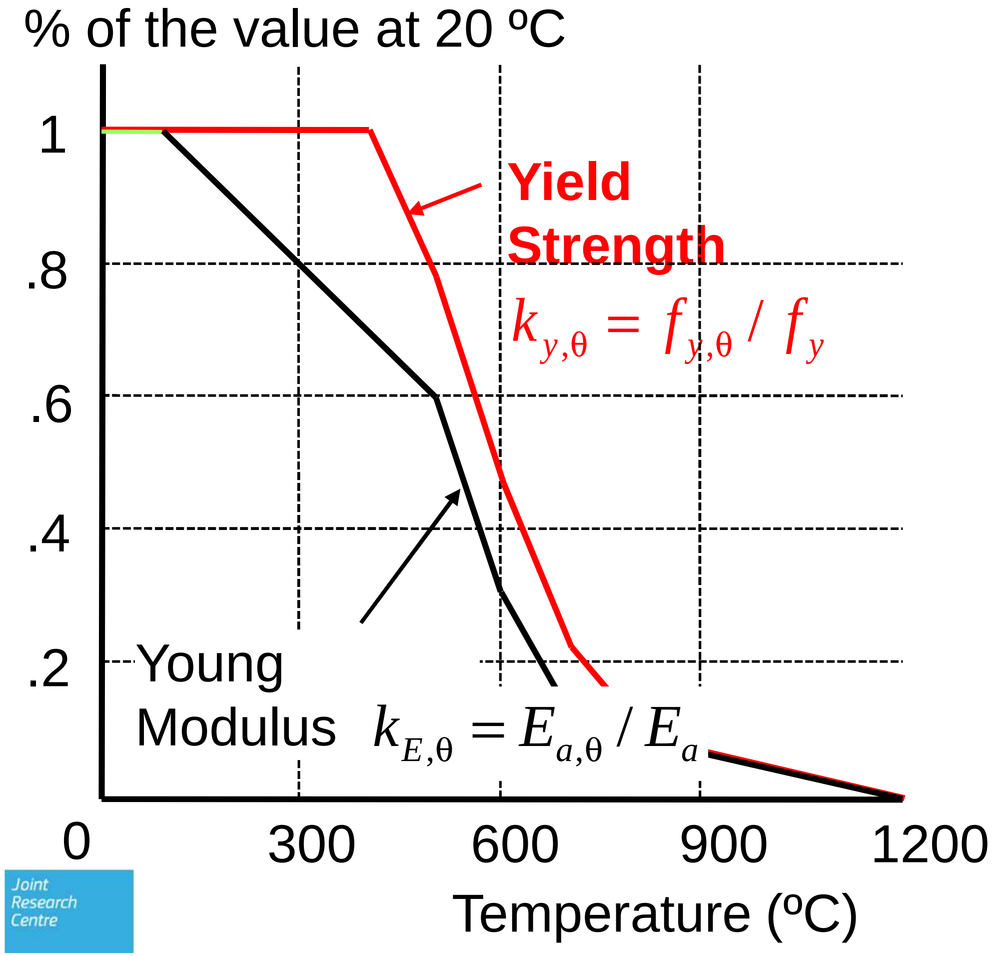
Stress-strain relationship at elevated temperatures

- Strength/stiffness reduction factors for elastic modulus and yield strength (2% total strain).
- Elastic modulus at 600° C reduced by about 70%.
- Yield strength at 600° C reduced by over 50%.



Reduction factors for stress-strain relationship of carbon steel at elevated temperatures

	Reduction factors at temperature θ_a relative to the value of f_y or E_a at 20°C		
Steel Temperature θ_a	Reduction factor (relative to f_y) for effective yield strength $k_{y,\theta} = f_{y,\theta}/f_y$	Reduction factor (relative to f_y) for proportional limit $k_{p,\theta} = f_{p,\theta}/f_y$	Reduction factor (relative to E_a) for the slope of the linear elastic range $k_{E,\theta} = E_{a,\theta}/E_a$
20°C	1,000	1,000	1,000
100°C	1,000	1,000	1,000
200°C	1,000	0,807	0,900
300°C	1,000	0,613	0,800
400°C	1,000	0,420	0,700
500°C	0,780	0,360	0,600
600°C	0,470	0,180	0,310
700°C	0,230	0,075	0,130
800°C	0,110	0,050	0,090
900°C	0,060	0,0375	0,0675
1000°C	0,040	0,0250	0,0450
1100°C	0,020	0,0125	0,0225
1200°C	0,000	0,0000	0,0000



Fire Resistance: Concept of critical temperature

The best fit curve to the points of this table can be obtained as:

Steel Temperature θ_a	Reduction factors at temperature θ_a relative to the value of f_y or E_a at 20°C		
	Reduction factor (relative to f_y) for effective yield strength $k_{y,\theta} = f_{y,\theta}/f_y$	Reduction factor (relative to f_y) for proportional limit $k_{p,\theta} = f_{p,\theta}/f_y$	Reduction factor (relative to E_a) for the slope of the linear elastic range $k_{E,\theta} = E_{a,\theta}/E_a$
20°C	1,000	1,000	1,000
100°C	1,000	1,000	1,000
200°C	1,000	0,807	0,900
300°C	1,000	0,613	0,800
400°C	1,000	0,420	0,700
500°C	0,780	0,360	0,600
600°C	0,470	0,180	0,310
700°C	0,230	0,075	0,130
800°C	0,110	0,050	0,090
900°C	0,060	0,0375	0,0675
1000°C	0,040	0,0250	0,0450
1100°C	0,020	0,0125	0,0225
1200°C	0,000	0,0000	0,0000

$$\theta_{a,cr} = 39.19 \ln\left[\frac{1}{0,9674 k_{y,\theta}^{\,3,833}} - 1\right] + 482$$

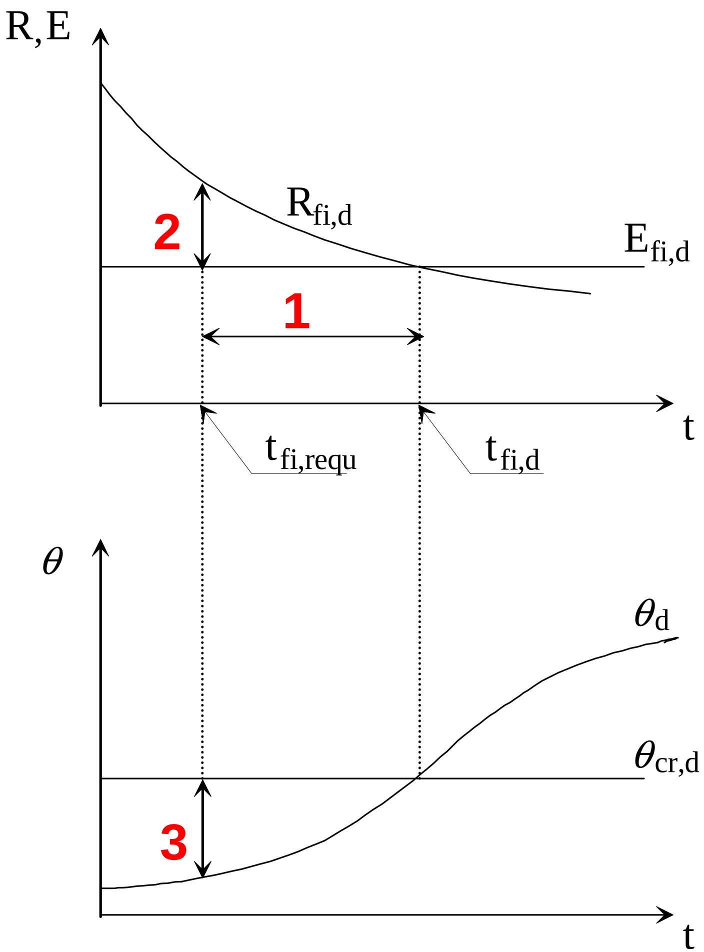
Eurocode 3

$$\theta_{a,cr} = 39.19 \ln\left[\frac{1}{0,9674 \mu_0^{\,3,833}} - 1\right] + 482$$

$$\mu_0 = \frac{E_{fi,d}}{R_{fi,d,0}} = k_{y,\theta}$$

Checking Fire Resistance. Domains of verification

Strategies with nominal fires



1. Time:

$t_{fi,d} > t_{fi,requ}$

2. Load resistance:

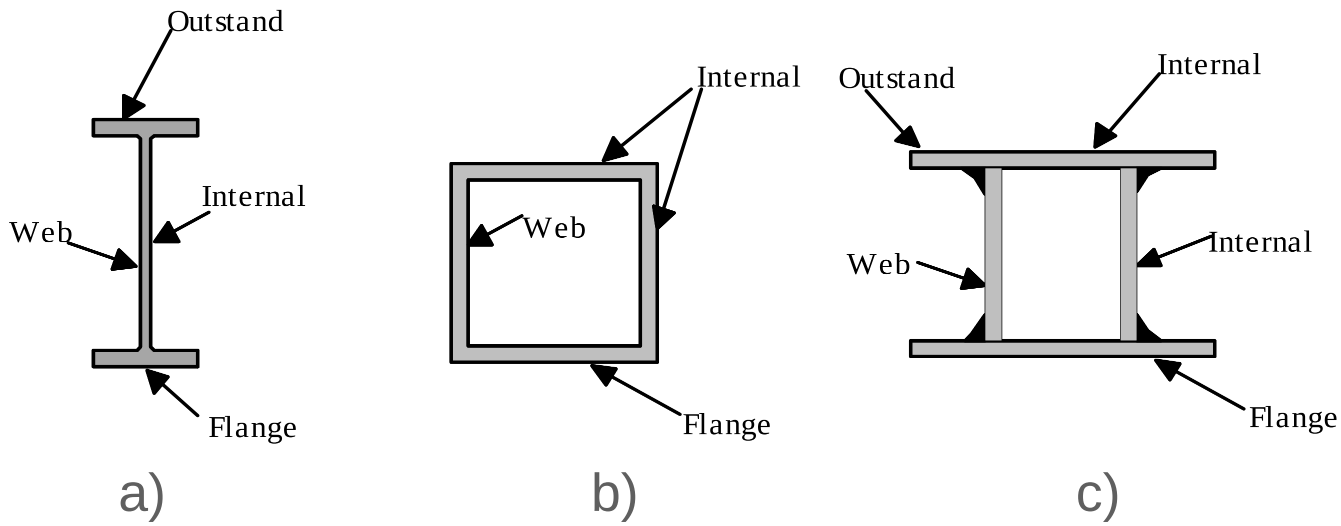
$R_{fi,d,t} > E_{fi,d}$

3. Temperature:

$\theta_d < \theta_{cr,d}$

Classification of the cross-sections - 1

Steel profiles can be considered as an assembly of individual plates



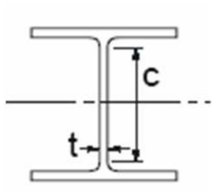
Internal and outstand elements

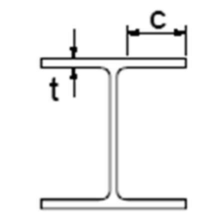
a) Rolled section; b) Hollow section; c) Welded section

Classification of the cross-sections - 2

$$\varepsilon = 0.85\sqrt{\frac{235}{f_y}}$$ and tables from EN 1993-1-1

The same limits as for normal temperature (EN 1993-1-1)





Element	Class 1	Class 2	Class 3
Flange	c / t = 9 ε	c / t = 10 ε	c / t = 14 ε
Web subjected to compression	c / t = 33 ε	c / t = 38 ε	c / t = 42 ε
Web subected tobending	c / t = 72 ε	c / t = 83 ε	c / t = 124 ε

Fire Resistance

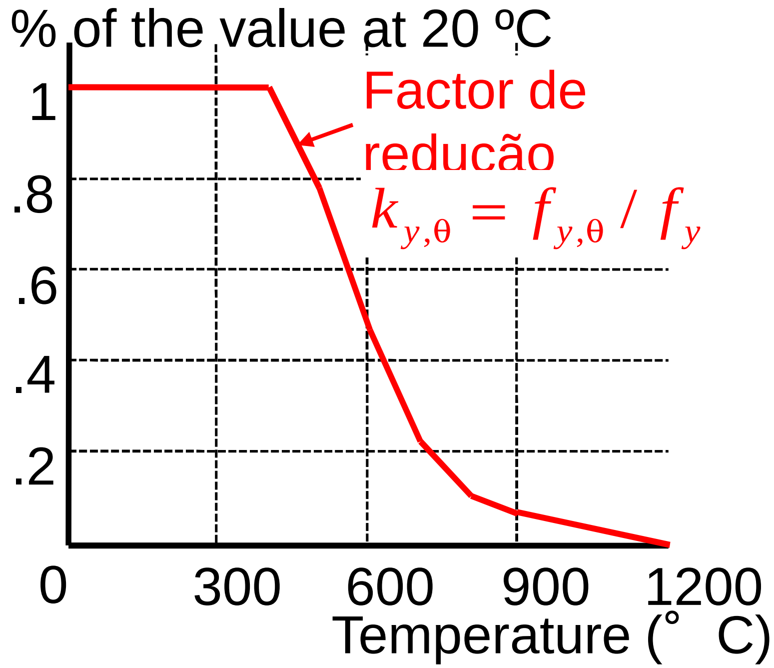
Tension members

- The design resistance of a tension member with uniform temperature θ_a is:

$$N_{fi,\theta,Rd} = k_{y,\theta} A f_y / \gamma_{M,fi}$$

or

$$N_{fi,\theta,Rd} = k_{y,\theta} N_{Rd} [\gamma_{M0} / \gamma_{M,fi}]$$



Fire Resistance

Compression members with Class 1, 2 or 3 cross-sections

$$N_{b,fi,\theta,Rd} = \chi_{fi} A k_{y,\theta} f_y \frac{1}{\gamma_{M,fi}}$$

with

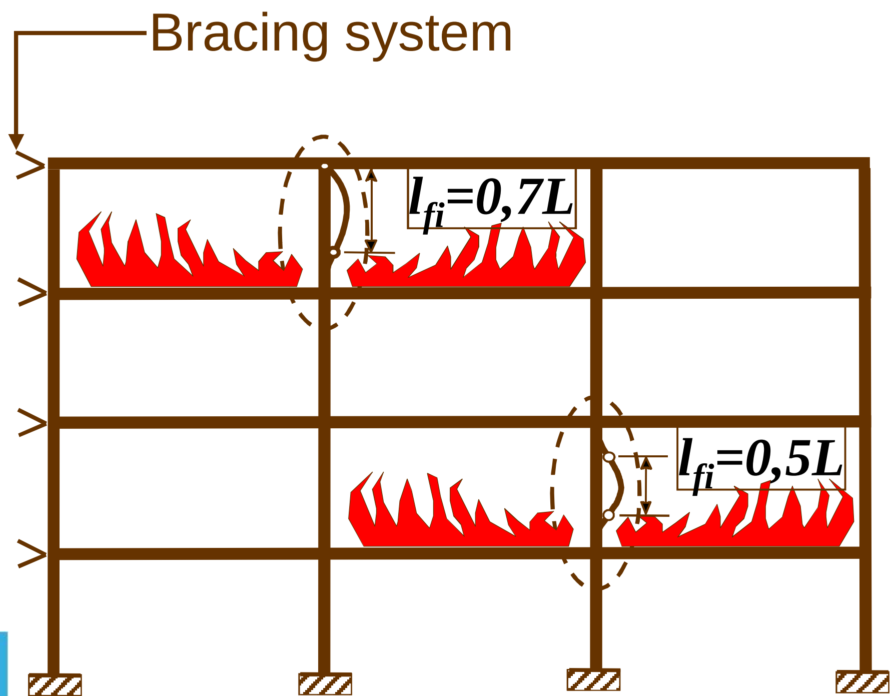
$$\chi_{fi} = \frac{1}{\phi_\theta + \sqrt{\phi_\theta^2 - \bar{\lambda}_\theta^2}}$$

$$\phi_\theta = \frac{1}{2}\left[1 + \alpha \bar{\lambda}_\theta + \bar{\lambda}_\theta^2\right]$$

$$\alpha = 0.65\sqrt{235 / f_y}$$ ~~(Curves a, b, c, d, a~0~)~~

- non dimensional slenderness:

$$\bar{\lambda}_\theta = \bar{\lambda}\sqrt{k_{y,\theta} / k_{E,\theta}}$$



Fire Resistance of Laterally restrained beams - 1

Class 1, 2 or3 cross-sections with uniform temperature

- The design moment resistance of a cross-section with a uniform temperature θ_a is:

$$M_{fi,\theta,Rd} = M_{Rd} k_{y,\theta} \left(\frac{\gamma_{M0}}{\gamma_{M,fi}} \right)$$

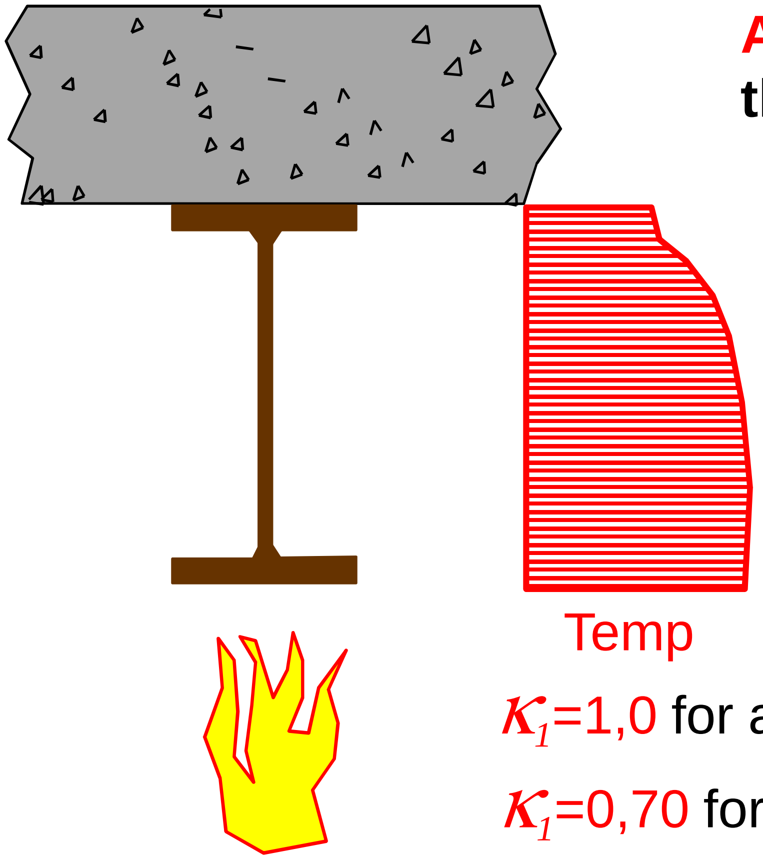
$M_{Rd} = M_{pl,Rd}$ – Class 1 or 2 cross-sections

$M_{Rd} = M_{el,Rd}$ – Class 3 cross-sections

$M_{Rd} = M_{eff,Rd}$ – Class 4 cross-sections

Fire Resistance of Laterally restrained beams - 2

Class 1, 2 or3 cross-sections with non-uniform temperature



Adaptation factors to take into account the non-uniform temperature distribution

Moment Resistance:

$$M_{fi,\theta,Rd} = M_{Rd} k_{y,\theta} \left(\frac{\gamma_{M0}}{\gamma_{M,fi}} \right) \frac{1}{\kappa_1 \kappa_2}$$

κ_1 is an adaptation factor for non-uniform temperature across the cross-section

κ_1=1,0 for a beam exposed on all four sides

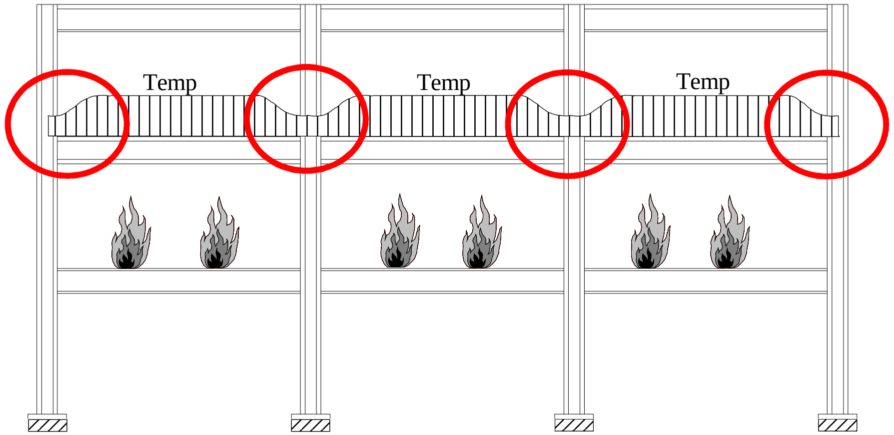
κ_1=0,70 for an unprotected beam exposed on three sides

κ_1=0,85 for a protected beam exposed on three sides

Fire Resistance of Laterally restrained beams - 3

Class 1, 2 or3 cross-sections with non-uniform temperature

Adaptation factors to take into account the non-uniform temperature distribution



Moment Resistance:

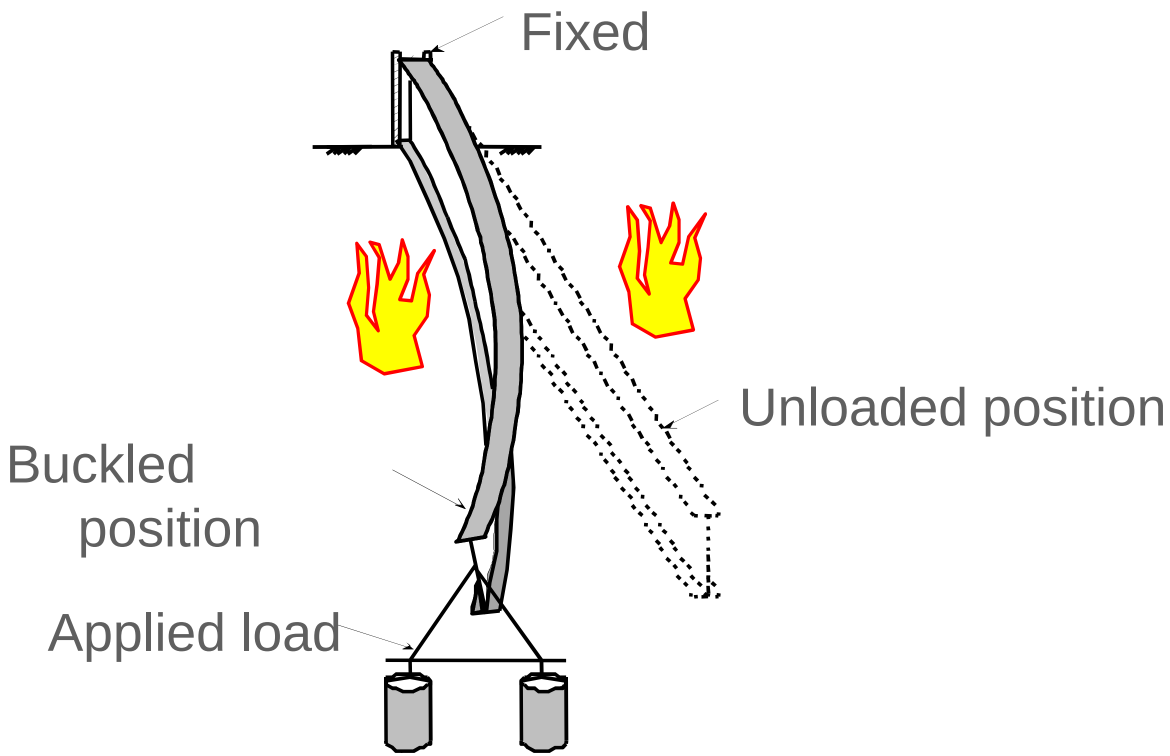
$$M_{fi,\theta,Rd} = M_{Rd} k_{y,\theta} \left(\frac{\gamma_{M0}}{\gamma_{M,fi}} \right) \frac{1}{\kappa_1 \kappa_2}$$

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

κ_2=0,85 at the supports of a statically indeterminate beam

κ_2=1.0 in all other cases

Fire Resistance of Laterally restrained beams - 1



Lateral-torsional buckling

Fire Resistance of Laterally restrained beams - 2

- Design lateral torsional buckling resistance moment of a laterally unrestrained beam at the max. temp. in the comp. flange $\theta_{a.com}$ is →

$$M_{b,fi,\theta,Rd} = \chi_{LT,fi} W_y k_{y,\theta,com} f_y \frac{1}{\gamma_{M,fi}}$$

- $\chi_{LT.fi}$ the reduction factor for lateral-torsional buckling in the fire design situation.

$$\chi_{LT,fi} = \frac{1}{\phi_{LT,\theta,com} + \sqrt{[\phi_{LT,\theta,com}]^2 - [\bar{\lambda}_{LT,\theta,com}]^2}}$$

$$\bar{\lambda}_{LT.\theta.com} = \bar{\lambda}_{LT} \sqrt{k_{y.\theta.com} / k_{E.\theta.com}}$$

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

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_y f_y}{M_{cr}}}$$

$$\alpha = 0.65\sqrt{235/f_y}$$

~~(Curves a, b, c, d)~~

Fire Resistance

Shear Resistance

$$V_{fi,t,Rd} = k_{y,\theta,web} V_{Rd} \left(\frac{\gamma_{M,0}}{\gamma_{M,fi}} \right)$$

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

θ_{web} is the average temperature in the web of the section.

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

Combined bending and axial compression - 1

Without lateral-torsional buckling

Class 1 and 2

$$\frac{N_{fi,Ed}}{\chi_{\min,fi}\, A\, k_{y,\theta}\, \dfrac{f_y}{\gamma_{M,fi}}} + \frac{k_y\, M_{y,fi,Ed}}{W_{pl,y}\, k_{y,\theta}\, \dfrac{f_y}{\gamma_{M,fi}}} + \frac{k_z\, M_{z,fi,Ed}}{W_{pl,z}\, k_{y,\theta}\, \dfrac{f_y}{\gamma_{M,fi}}} \leq 1$$

Class 3

$$\frac{N_{fi,Ed}}{\chi_{\min,fi}\, A\, k_{y,\theta}\, \dfrac{f_y}{\gamma_{M,fi}}} + \frac{k_y\, M_{y,fi,Ed}}{W_{el,y}\, k_{y,\theta}\, \dfrac{f_y}{\gamma_{M,fi}}} + \frac{k_z\, M_{z,fi,Ed}}{W_{el,z}\, k_{y,\theta}\, \dfrac{f_y}{\gamma_{M,fi}}} \leq 1$$

Combined bending and axial compression - 2

With lateral-torsional buckling

Class 1 and 2

$$\frac{N_{fi,Ed}}{\chi_{z,fi}\, A\, k_{y,\theta}\, \dfrac{f_y}{\gamma_{M,fi}}} + \frac{k_{LT}\, M_{y,fi,Ed}}{\chi_{LT,fi}\, W_{pl,y}\, k_{y,\theta}\, \dfrac{f_y}{\gamma_{M,fi}}} + \frac{k_z\, M_{z,fi,Ed}}{W_{pl,z}\, k_{y,\theta}\, \dfrac{f_y}{\gamma_{M,fi}}} \leq 1$$

Class 3

$$\frac{N_{fi,Ed}}{\chi_{z,fi}\, A\, k_{y,\theta}\, \dfrac{f_y}{\gamma_{M,fi}}} + \frac{k_{LT}\, M_{y,fi,Ed}}{\chi_{LT,fi}\, W_{el,y}\, k_{y,\theta}\, \dfrac{f_y}{\gamma_{M,fi}}} + \frac{k_z\, M_{z,fi,Ed}}{W_{el,z}\, k_{y,\theta}\, \dfrac{f_y}{\gamma_{M,fi}}} \leq 1$$

Combined bending and axial compression - 3

$$\mu_y = \left(2\beta_{M,y} - 5\right)\bar{\lambda}_{y,\theta} + 0.44\beta_{M,y} + 0.29 \leq 0.8 \text{ with } \bar{\lambda}_{y,20°C} \leq 1.1$$

and

$$k_z = 1 - \frac{\mu_z N_{fi,Ed}}{\chi_{z,fi} A k_{y,\theta} \dfrac{f_y}{\gamma_{M,fi}}} \leq 3$$

with

and

$$k_{LT} = 1 - \frac{\mu_{LT} N_{fi,Ed}}{\chi_{z,fi} A k_{y,\theta} \dfrac{f_y}{\gamma_{M,fi}}} \leq 1$$

with

$$\mu_{LT} = 0.15\bar{\lambda}_{z,\theta}\ \beta_{M.LT} - 0.15 \leq 0.9$$

Moment diagram	Equivalent uniform moment factor β_M
End moments M_1 … ψM_1 $-1 \leq \psi \leq 1$	$\beta_{M,\psi} = 1,8 - 0,7\,\psi$
Moments due to in-plane lateral loads M_Q	$\beta_{M,Q} = 1,3$
M_Q	$\beta_{M,Q} = 1,4$
Moments due to in-plane lateral loads plus end moments M_1, M_Q, ΔM	$\beta_M = \beta_{M,\psi} + \frac{M_Q}{\Delta M}(\beta_{M,Q} - \beta_{M,\psi})$ $M_Q = \lvert \max M \rvert$ due to lateral load only ΔM = $\lvert \max M \rvert$ for moment diagram without change of sign; $\lvert \max M \rvert + \lvert \min M \rvert$ for moment diagram with change of sign

Fire Resistance of a member in tension - 1

Example 3

Consider a HE 200 A profile in S275 grade steel that was designed at normal temperature for an axial tension load N_{Ed} = 1200 kN. The unprotected member is heated on all four sides and is part of an office building with a required fire resistance time to the standard fire of t_{requ} = 30 minutes (R30).

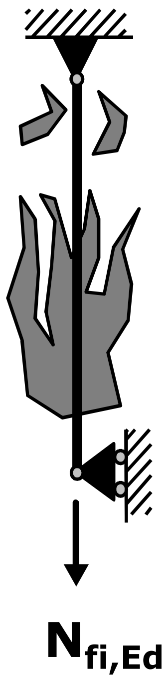
a) Evaluate de critical temperature of the profile;

b) Verify the fire resistance of the member:

b1) in the temperature domain;

b2) in the time domain;

b3) in the resistance domain.



Fire Resistance of a member in tension - 2

Example 3

Solution:

a) Assuming a reduction factor for the load in a fire situation of, $\eta_{fi} = 0.65$, the axial load in fire situation is given by

$$N_{fi,Ed} = \eta_{fi} N_{Ed} = 0.65 \times 1200 = 780 \text{ kN.}$$

The cross-sectional area of an HE 200 A is $A = 5380$ mm^2. The degree of utilisation takes the value:

$$\mu_0 = \frac{E_{fi,d}}{R_{fi,d,0}} = \frac{N_{fi,Ed}}{Af_y / \gamma_{M,fi}} = \frac{780000}{5380 \cdot 275 / 1.0} = 0.527$$

The value of the critical temperature is:

$$\theta_{a,cr} = 39.19 \ln\left[\frac{1}{0.9674\mu_0^{3.833}} - 1\right] + 482 = 576.1^{\circ}\text{C}$$

Fire Resistance of a member in tension - 3

Example 3

Another way to obtain the critical temperature is to consider that the collapse occurs when:

$$N_{fi,Ed} = N_{fi,Rd}$$

so

$$N_{fi,Ed} = Ak_{y,\theta} f_y / \gamma_{M,fi}$$

from where $k_{y,\theta}$ can be taken:

$$k_{y,\theta} = \frac{N_{fi,Ed}}{Af_y / \gamma_{M,fi}} = \frac{780000}{5380 \cdot 275 / 1.0} = 0.527$$

Knowing $k_{y,\theta}$, the value of the critical temperature can be obtained using:

$$\theta_{a,cr} = 39.19 \ln\left[\frac{1}{0.9674 k_{y,\theta}^{3.833}} - 1\right] + 482 = 576.1^{\text{o}}\text{C}$$

Instead of using this equation, interpolation on the table for $k_{y,\theta}$, gives:

$$\theta_{a,cr} = 581.6^{\text{o}}\text{C}$$

Fire Resistance of a member in tension - 4

Example 3

Table 5.2: Reduction factors for carbon steel for the design at elevated temperatures

Steel Temperature θ_a	Reduction factors at temperature θ_a relative to the value of f_y or E_a at 20°C			
	Reduction factor (relative to f_y) for effective yield strength $k_{y,\theta}=f_{y,\theta}/f_y$	Reduction factor (relative to f_y) for proportional limit $k_{p,\theta}=f_{p,\theta}/f_y$	Reduction factor (relative to E_a) for the slope of the linear elastic range $k_{E,\theta}=E_{a,\theta}/E_a$	Reduction factor (relative to f_y) for the design strength of hot rolled and welded thin walled sections (Class 4) $k_{0.2p,\theta}=f_{0.2p,\theta}/f_y$
20 °C	1.000	1.000	1.000	1.000
100 °C	1.000	1.000	1.000	1.000
200 °C	1.000	0.807	0.900	0.890
300 °C	1.000	0.613	0.800	0.780
400 °C	1.000	0.420	0.700	0.650
500 °C	0.780	0.360	0.600	0.530
600 °C	0.470	0.180	0.310	0.300
700 °C	0.230	0.075	0.130	0.130
800 °C	0.110	0.050	0.090	0.070
900 °C	0.060	0.0375	0.0675	0.050
1000 °C	0.040	0.0250	0.0450	0.030
1100 °C	0.020	0.0125	0.0225	0.020
1200 °C	0.000	0.0000	0.0000	0.000
NOTE: For intermediate values of the steel temperature, linear interpolation may be used.				

$K_{y,\theta} = 0.527$

581.6°C

Fire Resistance of a member in tension - 5

Example 3

b1) Verification in the temperature domain

The temperature in the section exposed on 4 sides after 30 min of standard fire ISO 834 exposure is 802°C (see example 1). As this temperature is greater than the critical temperature:

$$\theta_d > \theta_{cr,d} \text{, at time } t_{fi,requ}$$

and can not be classified as R30.

Fire Resistance of a member in tension - 6

Example 3

b2) Verification in the time domain

The time needed to reach the critical temperature can be obtained by double interpolation of the values given in Table of temperatures of unprotected members for a modified section factor

$$k_{sh}[A_m / V] = 0.618 \cdot 211 = 130.4 \text{ m}^{-1}$$

It is found that the critical temperature is reached at

$$t_{fi,d} = 14.08 \text{ min}$$

leading to the conclusion that the member is not safe because:

$$t_{fi,d} < t_{fi,requ}$$

The member doesn't fulfil the fire resistance criterion R30.

Fire Resistance of a member in tension - 7

Example 3

Temperature of unprotected steel in ºC, exposed to the ISO 834 fire curve

for different values of $k_{sh} \frac{A_m}{V}$, $[m^{-1}]$

130.4 m^{-1}

Time [min.]	10 m^{-1}	15 m^{-1}	20 m^{-1}	25 m^{-1}	30 m^{-1}	40 m^{-1}	60 m^{-1}	100 m^{-1}	200 m^{-1}	300 m^{-1}	400 m^{-1}
0	20	20	20	20	20	20	20	20	20	20	20
1	21	22	23	24	24	26	29	34	48	61	73
2	25	27	29	31	33	38	46	62	100	133	162
3	29	33	37	41	45	53	68	97	161	214	259
4	33	40	46	52	59	71	94	136	226	296	351
5	39	48	57	65	74	90	122	178	291	373	430
6	45	57	68	79	90	111	151	221	354	441	494
7	51	66	80	94	108	133	181	265	413	498	545
8	58	76	93	110	126	156	213	308	466	545	584
9	65	86	106	126	144	180	245	351	512	583	615
10	73	97	120	142	164	204	277	392	552	614	640
11	80	108	134	159	183	229	309	432	587	640	660
12	88	119	149	177	204	253	340	469	616	662	678
13	97	131	164	195	224	278	372	503	641	680	693
14	105	143	179	213	244	303	402	[illegible]	[illegible]	95	705
15	114	155	194	231	265	328	432	[illegible]	[illegible]	08	716

14.08 min

576.1ºC

Fire Resistance of a member in tension - 8

Example 3

b3) Verification in the resistance domain

After 30 minutes of standard fire exposure, the temperature is $\theta_d = 802$ ºC. Interpolating for this temperature in Table of $k_{y,\theta}$ leads to the reduction factor for the yield strength of

$$k_{y,\theta} = 0.109$$

and the resistance of the member after 30 minutes is

$$N_{fi,Rd} = Ak_{y,\theta} f_y / \gamma_{M,fi} = 5380 \cdot 0.109 \cdot 275 \times 10^{-3} / 1.0 = 161.3 \text{ kN}$$

which is less than the applied load in fire situation, $N_{fi,Ed} = 780$ kN, i. e.,

$$N_{fi,Ed} > N_{fi,Rd}$$

The member doesn’t fulfil the fire resistance criterion R30.

Fire Resistance of a member in tension - 9

Example 3

Table 5.2: Reduction factors for carbon steel for the design at elevated temperatures

Steel Temperature θ_a	Reduction factors at temperature θ_a relative to the value of f_y or E_a at 20°C			
	Reduction factor (relative to f_y) for effective yield strength $k_{y,\theta}=f_{y,\theta}/f_y$	Reduction factor (relative to f_y) for proportional limit $k_{p,\theta}=f_{p,\theta}/f_y$	Reduction factor (relative to E_a) for the slope of the linear elastic range $k_{E,\theta}=E_{a,\theta}/E_a$	Reduction factor (relative to f_y) for the design strength of hot rolled and welded thin walled sections (Class 4) $k_{0.2p,\theta}=f_{0.2p,\theta}/f_y$
20 °C	1.000	1.000	1.000	1.000
100 °C	1.000	1.000	1.000	1.000
200 °C	1.000	0.807	0.900	0.890
300 °C	1.000	0.613	0.800	0.780
400 °C	1.000	0.420	0.700	0.650
500 °C	0.780	0.360	0.600	0.530
600 °C	0.470	0.180	0.310	0.300
700 °C	0.230	0.075	0.130	0.130
800 °C	0.110	0.050	0.090	0.070
900 °C	0.060	0.0375	0.0675	0.050
1000 °C	0.040	0.0250	0.0450	0.030
1100 °C	0.020	0.0125	0.0225	0.020
1200 °C	0.000	0.0000	0.0000	0.000
NOTE: For intermediate values of the steel temperature, linear interpolation may be used.				

802°C → X

$K_{y,\theta} = 0.109$

Fire Resistance of a Laterally restrained beam - 1

Example 4

Consider a simply supported restrained beam 4.0 m long, constructed from an IPE 300 section, in steel grade S235, supporting a concrete slab. Assuming the steel beam does not act compositely with the concrete slab and that the design load in the fire situation is $q_{fi,Ed} = 33.8$ kN/m, verify if it is necessary to protect the beam for a fire resistance period of R90.

Solution:

The relevant geometrical characteristics of the profile for the cross section classification are

h = 300 mm

b = 150 mm

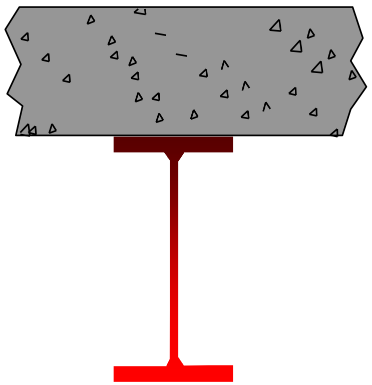
t_w = 7.1 mm

t_f = 10.7 mm

r = 15 mm

$c = b/2 - t_w/2 - r$ = 56.45 mm (flange)

$c = h - 2t_f - 2r$ = 248.6 mm (web)



Fire Resistance of a Laterally restrained beam - 2

Example 4

As the steel grade is S235

$$\varepsilon = 0.85\sqrt{235/f_y} = 0.85$$

The class of the flange in compression is

$$c/t_f = 56.45/10.7 = 5.3 < 9\varepsilon = 6.8 \Rightarrow \text{Class 1}$$

For the web in bending the class is

$$d/t_w = 248.6/7.1 = 35 < 72\varepsilon = 61.2 \Rightarrow \text{Class 1}$$

The cross section of the IPE 300 in bending and in fire situation is Class 1.

Fire Resistance of a Laterally restrained beam - 3

Example 4

- Critical temperature considering shear force at the supports

The design value of the shear force in a fire situation at the supports is:

$$V_{fi,Ed} = \frac{q_{fi,Ed} \cdot L}{2} = \frac{33.8 \cdot 4}{2} = 67.6 \text{ kN}$$

The shear area of the IPE 300 is

$$A_v = A - 2bt_f + (t_w + 2r)t_f =$$

$$= 5380 - 2 \cdot 150 \cdot 10.7 + (7.1 + 2 \cdot 15) \cdot 10.7 = 2567 \text{ mm}^2$$

At the collapse

$$V_{pl,y,fi,Rd} = V_{fi,Ed} \Rightarrow \frac{A_v k_{y,\theta} f_y}{\sqrt{3}\gamma_{M,fi}} = \frac{2567 \cdot k_{y,\theta} \cdot 235}{\sqrt{3} \cdot 1.0} \times 10^{-3} = 67.6 \text{ kN}$$

$$k_{y,\theta} = 0.194 \quad \Rightarrow \quad \theta_{a,cr} = 39.19 \ln\left(\frac{1}{0.9674 \cdot k_{y,\theta}^{3.833}} - 1\right) + 482 = 736.7^\circ \text{C}$$

Fire Resistance of a Laterally restrained beam - 4

Example 4

Critical temperature considering the bending moment at mid span:

The next step is to calculate the fire resistance of the unprotected beam. The design value of the resistance moment at time $t = 0$, $M_{fi,0,Rd}$, is

$$M_{fi,0,Rd} = W_{pl,y} f_y / (k_1 k_2 \gamma_{M,fi})$$

where:

$k_1 = 0.7$ for an unprotected beam exposed on three sides, with a concrete slab on the fourth side;

$k_2 = 1.0$ for sections not at the supports.

The plastic section modulus $W_{pl,y}$ of the IPE 300 profile is

$$W_{pl,y} = 628 \times 10^{-6} \text{ m}^3$$

and

$$M_{fi,0,Rd} = 211 \text{ kNm}$$

Fire Resistance of a Laterally restrained beam - 5

Example 4

The degree of utilisation takes the value

$$\mu_0 = \frac{M_{fi,Ed}}{M_{fi,0,Rd}} = \frac{67.6}{211} = 0.32$$

and the critical temperature is

$$\theta_{a,cr} = 39.19 \ln\left(\frac{1}{0.9674 \cdot \mu_0^{3.833}} - 1\right) + 482 = 654^{\circ}\mathrm{C}$$

The critical temperature for the beam considering shear and bending is:

$$\theta_{a,cr} = \min(736.7^{o}C; 654^{o}C) = 654^{o}C$$

Considering that the section factor for the IPE 300 is $A_m / V = 187$ m^{-1}, and that

$h = 300$ mm; $b = 150$ mm and $A = 53.8$ cm^2

the box value of the section factor $[A_m / V]_b$, is

$$[A_m / V]_b = \frac{2h + b}{A} = \frac{2 \cdot 0.3 + 0.15}{53.8 \times 10^{-4}} = 139.4 \text{ m}^{-1}$$

Fire Resistance of a Laterally restrained beam - 6

Example 4

The value of A_m / V, which is

$$A_m / V = 187.7 \text{ m}^{-1}$$

The correction factor for the shadow effect is

$$k_{sh} = 0.9[A_m / V]_b / [A_m / V] = 0.9 \cdot 139.4 / 187.7 = 0.6684$$

and the modified section factor thus takes the value

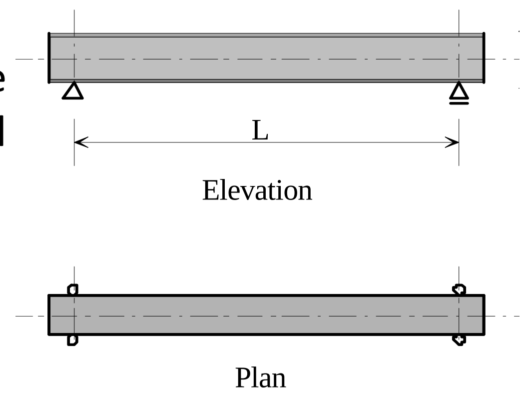
$$k_{sh}[A_m / V] = 0.6684 \cdot 187.7 = 125.5 \text{ m}^{-1}$$

Interpolation of the values given in the Table of temperatures of unprotected profiles yields a time of 18 min to reach the critical temperature, $\theta_{a,cr} = 654$ ºC. This is less than the required 90 min and therefore fire protection is necessary to achieve the required fire resistance (see the procedure in Example 2). Using fire protection it should be used $k_1 = 0.85$ when evaluating the resistance moment under fire conditions.

Fire Resistance of a Laterally unrestrained beam - 1

Example 5

Consider a simply supported IPE 300, S235 grade steel beam from an office building, with fork supports. The member is 5.0 m long and is subjected to a transverse uniform load at normal temperature $q_{Ed} = 19.2$ kN/m. The transverse loading is assumed to act at the shear centre of the beam. Evaluate the critical temperature assuming that lateral-torsional buckling can occur.



Solution:

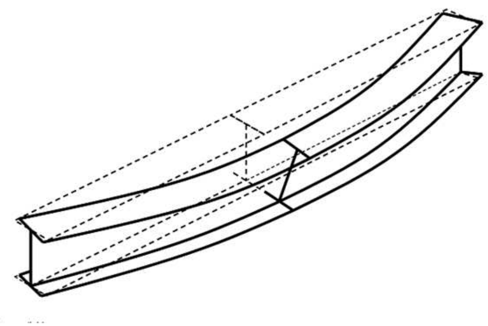
The IPE 300 in bending is Class 1.

For an office building a reduction factor for the loads in a fire situation can be taken as $\eta_{fi} = 0.65$, leading to

$$q_{fi,Ed} = \eta_{fi} q_{Ed} = 0.65 \cdot 19.2 = 12.48 \text{ kN/m}$$

Thus the design value of the moment in a fire situation is

$$M_{fi,Ed} = \frac{q_{fi,Ed} L^2}{8} = \frac{12.48 \cdot 5^2}{8} = 39.0 \text{ kNm}$$



Fire Resistance of a Laterally unrestrained beam - 2

Example 5

The following data is relevant for solving the problem:

- Young's modulus $E = 210000\ \text{N/mm}^2$
- Shear modulus $G = E/[2(1+\nu)]\ \text{N/mm}^2$
- Poisson's ratio $\nu = 0.3$
- Length L = 5000 mm
- Cross section classification Class 1
- Plastic section modulus $W_{pl,y} = 628400\text{mm}^3$
- Second moment of area about the minor axis

  $I_z = 603.8 \times 10^4\ \text{mm}^4$
- Warping constant $I_w = 125.9 \times 10^9\ \text{mm}^6$
- Torsion constant $I_t = 20.12 \times 10^4\ \text{mm}^4$

Fire Resistance of a Laterally unrestrained beam - 3

Example 5

- Critical temperature considering shear resistance at the supports

The design value of the shear force in a fire situation at the supports is:

$$V_{fi,Ed} = \frac{q_{fi,Ed} \cdot L}{2} = \frac{12.48 \cdot 5}{2} = 31.2 \text{ kN}$$

The shear area of the IPE 300 is

$$A_v = A - 2bt_f + (t_w + 2r)t_f =$$

$$= 5380 - 2 \cdot 150 \cdot 10.7 + (7.1 + 2 \cdot 15) \cdot 10.7 = 2567 \text{ mm}^2$$

At the collapse

$$V_{pl,y,fi,Rd} = V_{fi,Ed} \Rightarrow \frac{A_v k_{y,\theta} f_y}{\sqrt{3}\gamma_{M,fi}} = \frac{2567 \cdot k_{y,\theta} \cdot 235}{\sqrt{3} \cdot 1.0} \times 10^{-3} = 31.2 \text{ kN}$$

$$k_{y,\theta} = 0.09 \Rightarrow \theta_{a,cr} = 840^{o}\text{C}$$

Fire Resistance of a Laterally unrestrained beam - 4

Example 5

- Critical temperature considering Lateral-torsional buckling

The design lateral-torsional buckling resistance moment $M_{b,fi,0,Rd}$ at time $t = 0$ of a laterally unrestrained beam is given by:

$$M_{b,fi,0,Rd} = \chi_{LT,fi} W_{pl,y} f_y / \gamma_{M,fi}$$

As the reduction factor for the lateral-torsional buckling moment, $\chi_{LT,fi}$, depends on the temperature, an iterative procedure must be used.

This reduction factor in a fire situation is given by:

$$\chi_{LT,fi} = \frac{1}{\phi_{LT,\theta} + \sqrt{[\phi_{LT,\theta}]^2 - [\bar{\lambda}_{LT,\theta}]^2}}$$

with

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

Fire Resistance of a Laterally unrestrained beam - 5

Example 5

The imperfection factor is:

$$\alpha = 0.65\sqrt{235 / f_y}$$

and

$$\bar{\lambda}_{LT,\theta} = \bar{\lambda}_{LT}\sqrt{k_{y,\theta} / k_{E,\theta}}$$

The non-dimensional slenderness at normal temperature is given by:

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_{pl,y} f_y}{M_{cr}}}$$

The elastic critical moment for lateral-torsional buckling is given by:

$$M_{cr} = 1.12\frac{\pi^2 EI_z}{L^2}\sqrt{\frac{I_w}{I_z} + \frac{L^2 GI_t}{\pi^2 EI_z}} \times 10^{-6} = 129.4 \text{ kNm}$$

and

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_{pl,y} f_y}{M_{cr}}} = 1.068$$

Fire Resistance of a Laterally unrestrained beam - 6

Example 5

At normal temperature

$$\bar{\lambda}_{LT,20°C} = \bar{\lambda}_{LT}\sqrt{k_{y,20°C}/k_{E,20°C}} = 1.068\sqrt{1.0/1.0} = 1.068$$

and

$$\phi_{LT,20°C} = \frac{1}{2}(1 + 0.65 \cdot 1.068 + 1.068^2) = 1.42$$

and

$$\chi_{LT,fi} = \frac{1}{1.42 + \sqrt{1.42^2 - 1.068^2}} = 0.424$$

Giving the design lateral-torsional buckling resistance moment $M_{b,fi,0,Rd}$ at time $t = 0$:

$$M_{b,fi,0,Rd} = 0.424 \cdot 628400 \cdot 235 \times 10^{-6} = 62.6\,\text{kNm}$$

The degree of utilisation at time $t = 0$, is

$$\mu_0 = \frac{M_{fi,d}}{M_{b,fi,0,Rd}} = \frac{39.0}{62.6} = 0.623$$

Fire Resistance of a Laterally unrestrained beam - 7

Example 5

And the critical temperature

$$\theta_{a,cr} = 39.19 \ln\left(\frac{1}{0.9674 \cdot 0.623^{3.833}} - 1\right) + 482 = 548 \ ^{o}\text{C}$$

Using this temperature, the non-dimensional slenderness $\bar{\lambda}_{LT,\theta}$ may be corrected. The procedure must be repeated until convergence is reached, as shown in the next table:

θ [ºC]	$\sqrt{\frac{k_{y,\theta}}{k_{E,\theta}}}$	$\bar{\lambda}_{LT,\theta} = \bar{\lambda}_{LT} \cdot \sqrt{\frac{k_{y,\theta}}{k_{E,\theta}}}$	$\chi_{LT,fi}$	$M_{b,fi,0,Rd} = \chi_{LT,fi} W_{pl,y} f_y$ [kNm]	$\mu_0 = \frac{M_{fi,Ed}}{M_{b,fi,0,Rd}}$	$\theta_{a,cr}$ [ºC]
20	1.00	1.068	0.424	62.6	0.623	548
548	1.16	1.239	0.358	52.9	0.740	515
515	1.15	1.229	0.364	53.8	0.725	519
519	1.15	1.222	0.364	53.8	0.725	519

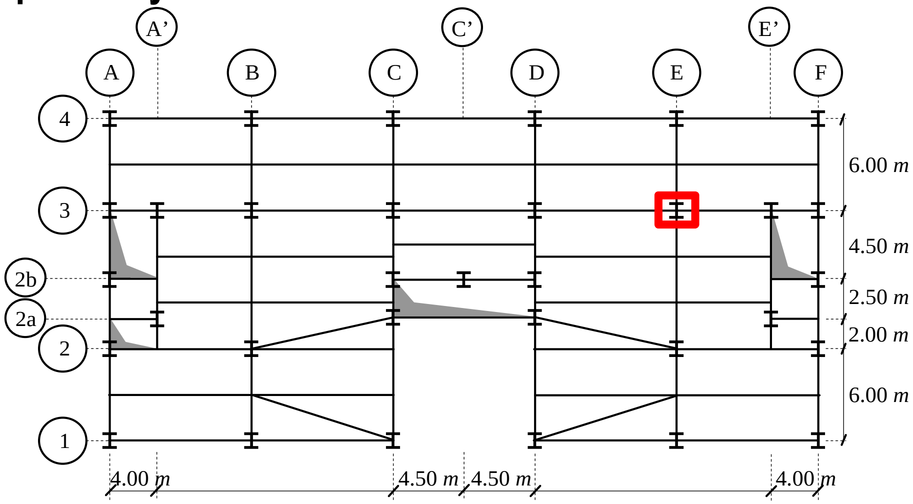
Convergence was reached at a critical temperature of $\theta_{a,cr} = 519$ ºC.

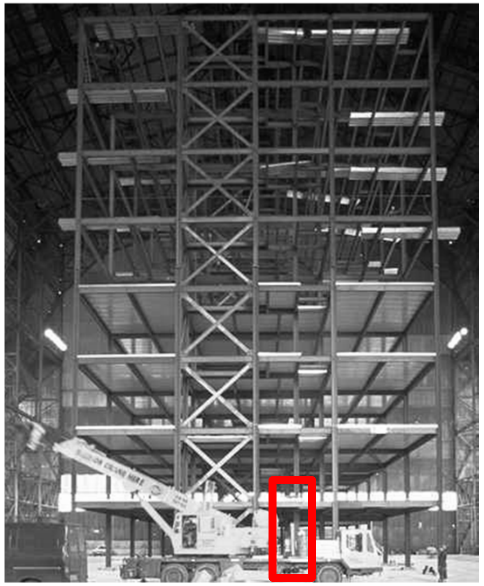
$$\theta_{a,cr} = \min(840^{o}C; 519^{o}C) = 519^{o}C$$

Fire design of columns - 1

EXAMPLE 6

Evaluate the thickness of gypsum boards needed to protect the inner column E-3 at the base level of the building represented in the figure to have a fire resistance of R90. The column has a length of 4.335 m and is composed by a section HEB 340 in steel S355.





Building – master example

In this column the bending moments (and the shear force) may be neglected; the **design axial force** (of compression) at normal temperature takes the value $\boldsymbol{N_{Ed}}$ **= 3326.0 kN**.

(see the presentation of Rui Simões)

Fire design of columns - 2

EXAMPLE 6

Solution:

- As the design for normal temperature has already made, and we don't know the value of G_k neither the value of Q_k to evaluate η_{fi} as

$$\eta_{fi} = \frac{\gamma_{GA} G_k + \psi_{1,1} Q_{k,1}}{\gamma_G G_k + \gamma_{Q,1} Q_{k,1}}$$

Under fire conditions the design axial compression force is taken as:

$$N_{fi,Ed} = \eta_{fi} N_{Ed} = 0.65 \times 3326 = 2161.9 \text{ kN.}$$

- Assuming that the supports are fixed and that the frame is braced, the buckling length in fire situation is:

$$l_{y,fi} = l_{z,fi} = 0.5L = 0.5 \times 4335 = 2167.5 \text{ mm}$$

Fire design of columns - 3

EXAMPLE 6

Classification of the cross section:

The relevant geometrical characteristics of the profile for the cross section classification are: h = 340 mm; b = 300 mm; t_w = 12 mm; t_f = 21.5 mm; r = 27 mm.

$c = b/2 - t_w/2 - r$ = 117 mm (flange)

$c = h - 2t_f - 2r$ = 243 mm (web)

As the steel grade is S355, $\varepsilon = 0.85\sqrt{235/f_y} = 0.692$

The class of the flange in compression is

$$c/t_f = 117/21.5 = 5.44 < 9\varepsilon = 6.23 \Rightarrow \text{Class 1}$$

The class of the web in compression is

$$d/t_w = 240/12 = 20.25 < 33\varepsilon = 22.8 \Rightarrow \text{Class 1}$$

The cross section of the HE 340 B under compression, in fire situation is Class 1.

Fire design of columns - 4

EXAMPLE 6

Evaluation of the critical temperature:

As the buckling length in both directions is the same and in fire design there is only one buckling curve, it is only necessary consider buckling about z-z.

For the HE 340 B:

Area, A = 17090 mm^2

Second moment of area, I_z = 96900000 mm^4

The design value of the compression load in fire situation: $N_{fi,Ed} = 2161.9\ \text{kN}$

The Euler critical load takes the value:

$$N_{cr} = \frac{\pi^2 EI}{l_{fi}^{\,2}} = 42748867\ \text{N}$$

Fire design of columns - 5

EXAMPLE 6

The non-dimensional slenderness at elevated temperature is given by

$$\bar{\lambda}_\theta = \bar{\lambda} \cdot \sqrt{\frac{k_{y,\theta}}{k_{E,\theta}}}$$

This is temperature dependent and an iterative procedure is needed to calculate the critical temperature. Starting with a temperature of 20 °C at which $k_{y,\theta} = k_{E,\theta} = 1.0$:

$$\bar{\lambda}_\theta = \bar{\lambda}\sqrt{\frac{k_{y,\theta}}{k_{E,\theta}}} = \bar{\lambda} = \sqrt{\frac{Af_y}{N_{cr}}} = \sqrt{\frac{17090 \cdot 355}{42748867}} = 0.377$$

The imperfection factor takes the value:

$$\alpha = 0.65\sqrt{235/\ f_y} = 0.65\sqrt{235/355} = 0.529$$

and

$$\phi = \frac{1}{2}(1 + 0.529 \cdot 0.377 + 0.377^2) = 0.673$$

Fire design of columns - 6

EXAMPLE 6

Therefore the reduction factor for flexural buckling is:

$$\chi_{fi} = \frac{1}{0.673 + \sqrt{0.673^2 - 0.377^2}} = 0.813$$

The design value of the buckling resistance $N_{b,fi,t,Rd}$ at time:

$$N_{b,fi,0,Rd} = \chi_{fi} A f_y / \gamma_{M,fi} = 4932\text{kN}$$

and the degree of utilisation takes the value:

$$\mu_0 = \frac{N_{fi,Ed}}{N_{fi,0,Rd}} = \frac{2161.9}{4932} = 0.438$$

For this degree of utilisation the critical temperature takes the value:

$$\theta_{a,cr} = 39.19 \ln\left[\frac{1}{0.9674\mu_0^{3.833}} - 1\right] + 482 = 607.4^\circ\text{C}$$

Fire design of columns - 7

EXAMPLE 6

Using this temperature, the non-dimensional slenderness $\bar{\lambda}_\theta$ can be corrected, which leads to another critical temperature. The iterative procedure should continue until convergence is reached, as illustrated in the next table:

θ [ºC]	$\sqrt{\dfrac{k_{y,\theta}}{k_{E,\theta}}}$	$\bar{\lambda}_\theta = \bar{\lambda} \cdot \sqrt{\dfrac{k_{y,\theta}}{k_{E,\theta}}}$	χ_{fi}	$N_{fi,0,Rd} = \chi_{fi} A f_y$ [kN]	$\mu_0 = \dfrac{N_{fi,Ed}}{N_{fi,0,Rd}}$	$\theta_{a,cr}$ [ºC]
20	1,00	0,377	0,813	4932	0,438	607.4
607.4	1,21	0,456	0,725	4708	0,459	598.5
598.5	1,21	0,456	0,725	4708	0,459	598.5

After three iterations a critical temperature of $\theta_{a,cr} = 598.5°C$ is obtained.

Fire design of columns - 8

EXAMPLE 6

Following the same procedure of Example 2 a thickness of $\boldsymbol{d_p}$ **= 14 mm of gypsum boards** was obtained so that the column can be classified as **R90**.

This value will be confirmed using the **software Elefir-EN** in the next slides.

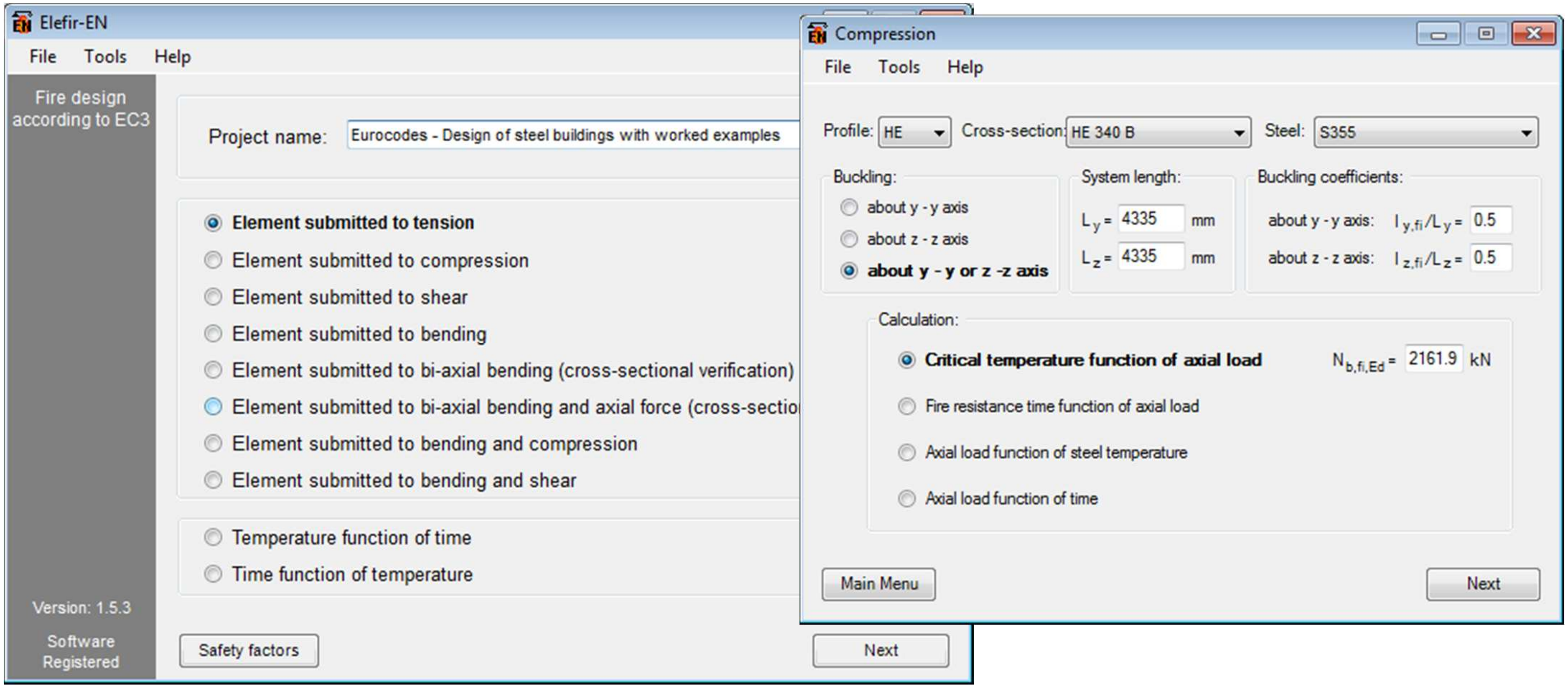
Using the software Elefir-EN - 1

EXAMPLE 6

Elefir-EN V1.5.3 (2013), Paulo Vila Real and Jean-Marc Franssen,
http://elefiren.web.ua.pt

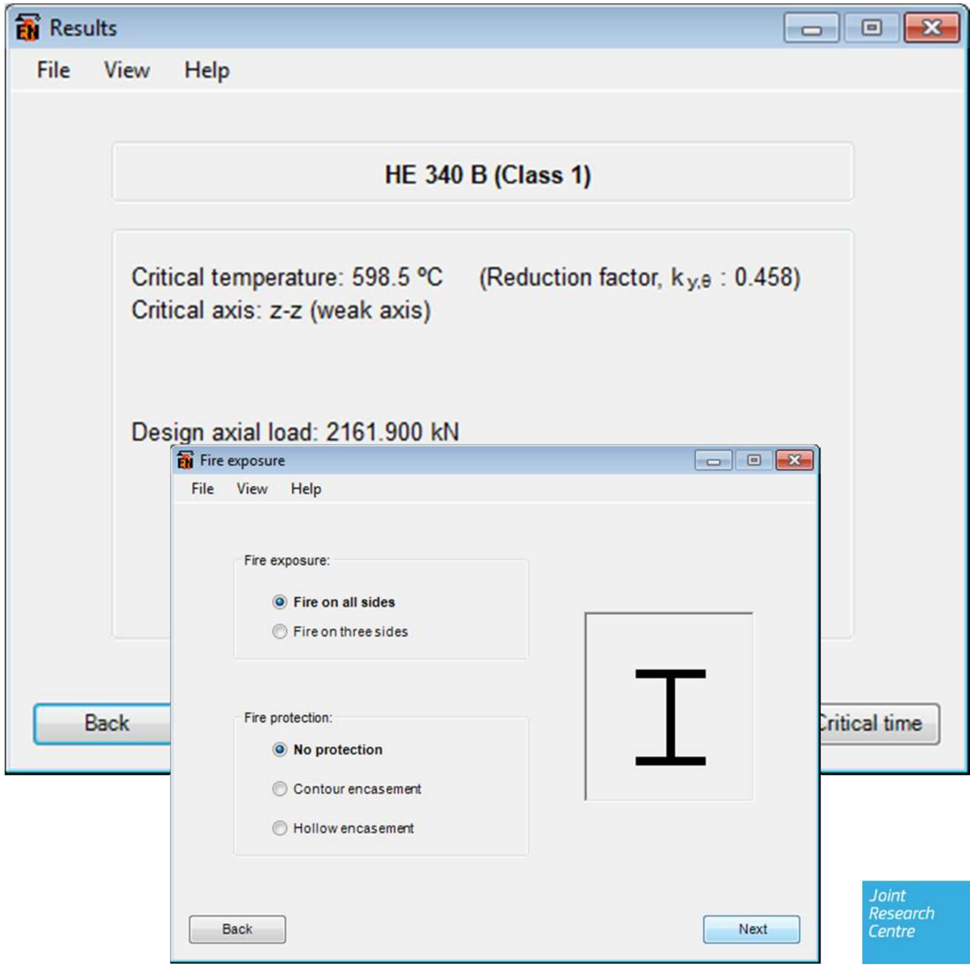
Fire design of columns - 2

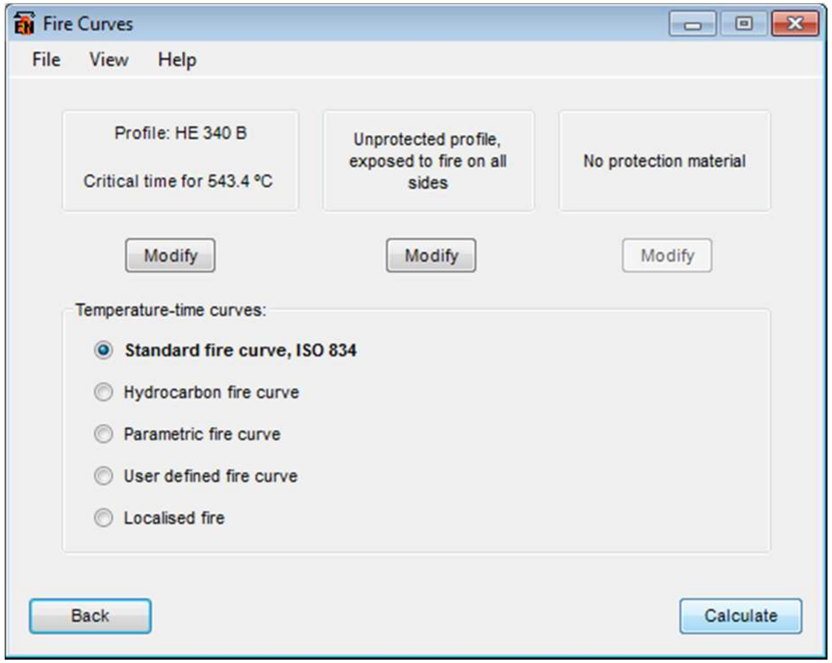
EXAMPLE 6



Fire design of columns - 3

EXAMPLE 6





Fire design of columns - 4

EXAMPLE 6

Standard fire curve, ISO 834

File View Help

Profile: HE 340 B (unprotected)

Section factor: $[A_m/V] = 105.9\ m^{-1}$

Modified section factor: $k_{sh}[A_m/V] = 67.4\ m^{-1}$ (used in calculation)

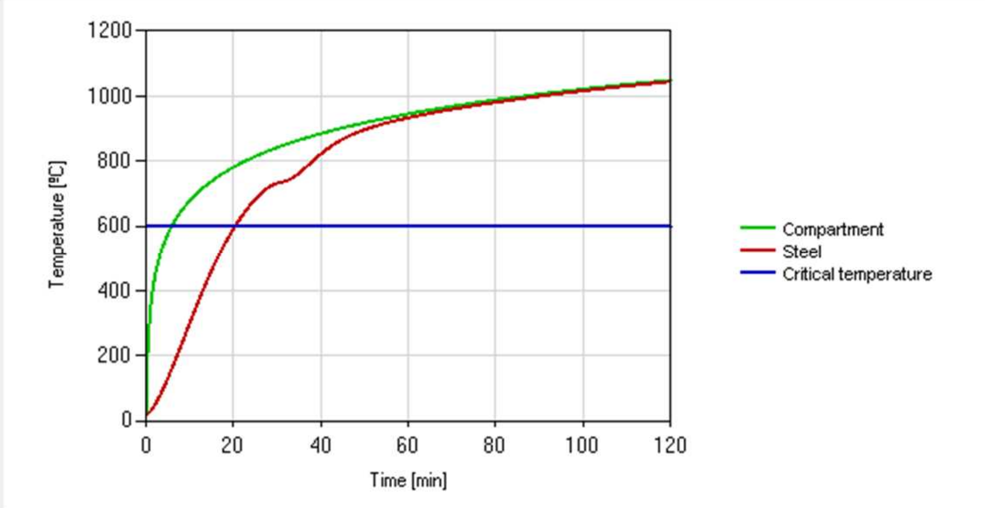
A critical temperature of 598.5 °C is reached after 20.30 minutes

Design axial load: 2161.900 kN

Back

Graph

File View Help



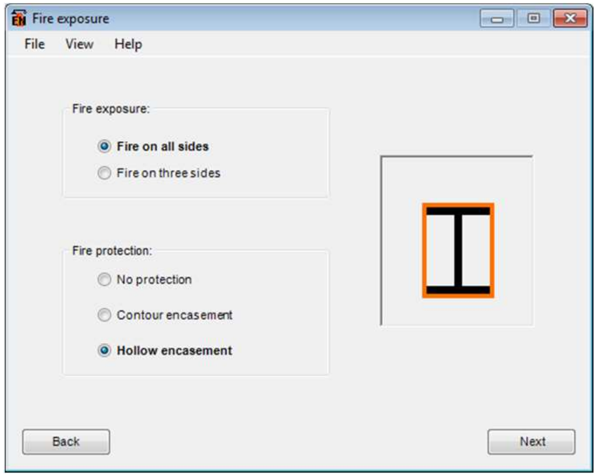
[Maximum temperature] Compartment: 1049.0 °C at 120 min / Steel: 1045 °C at 120 min

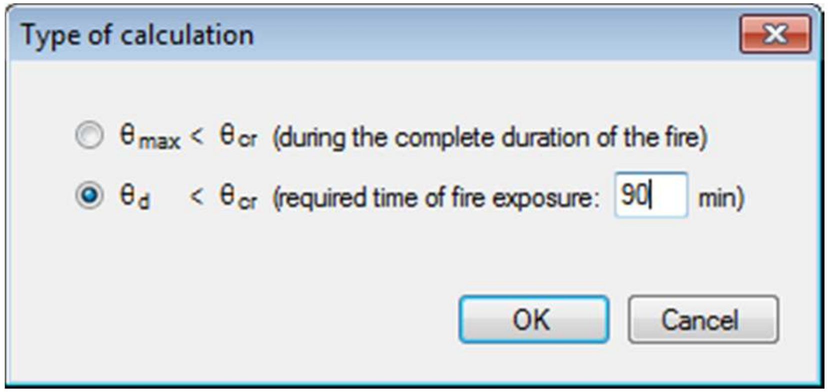
A critical temperature of 598.5 °C is reached after 20.30 minutes

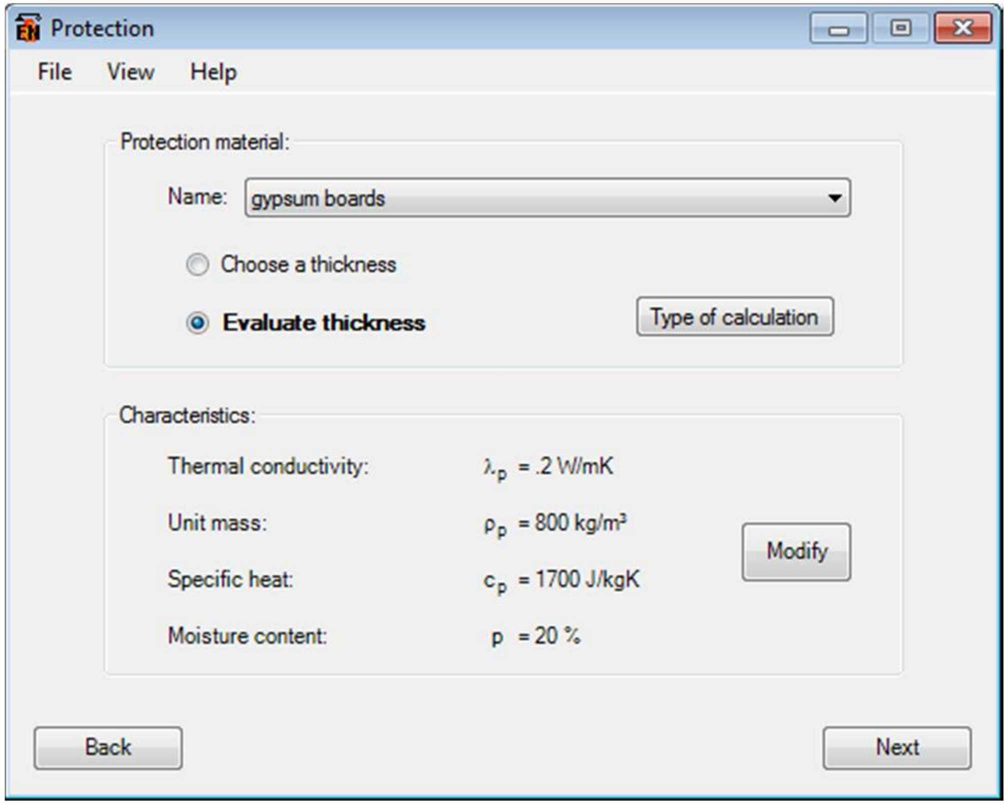
Back Main menu Report

Fire design of columns - 5

EXAMPLE 6

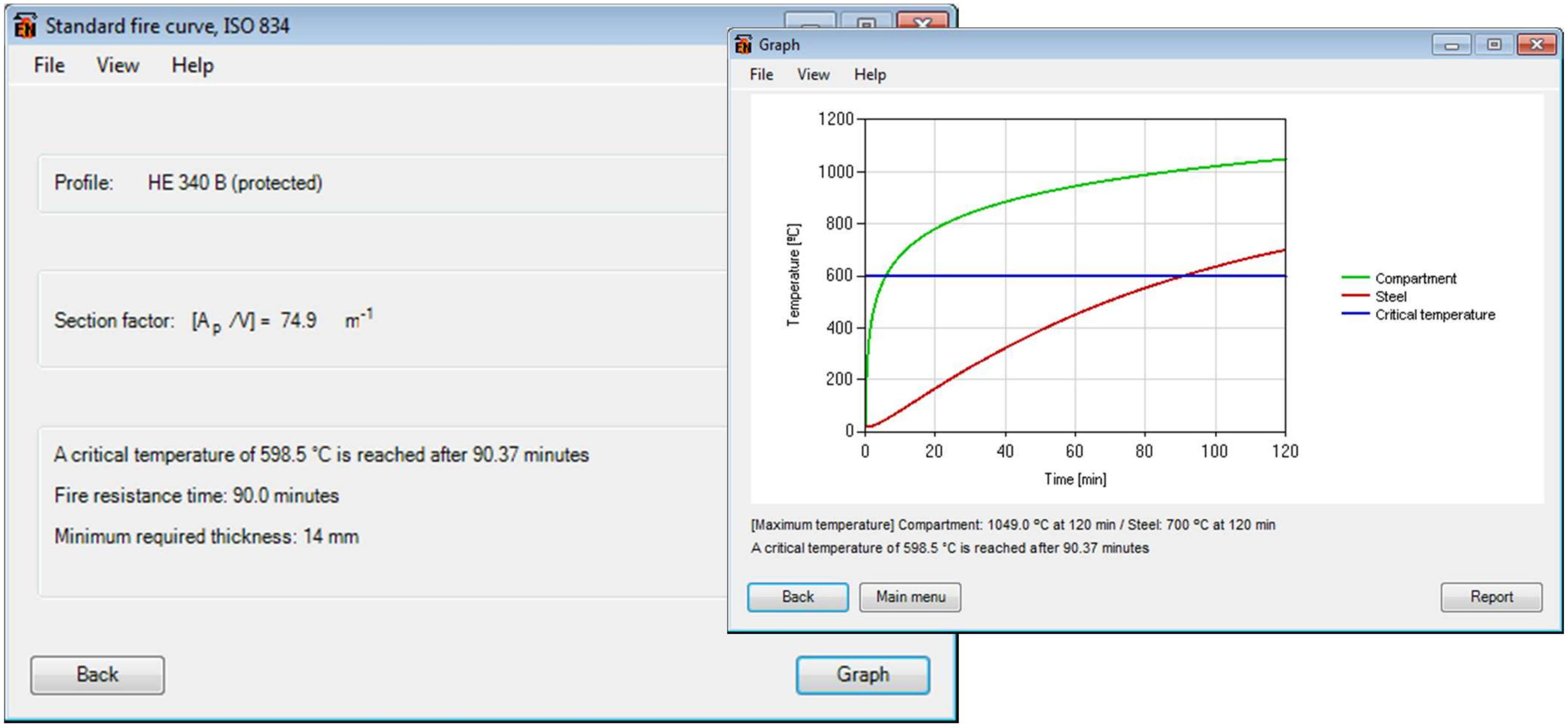






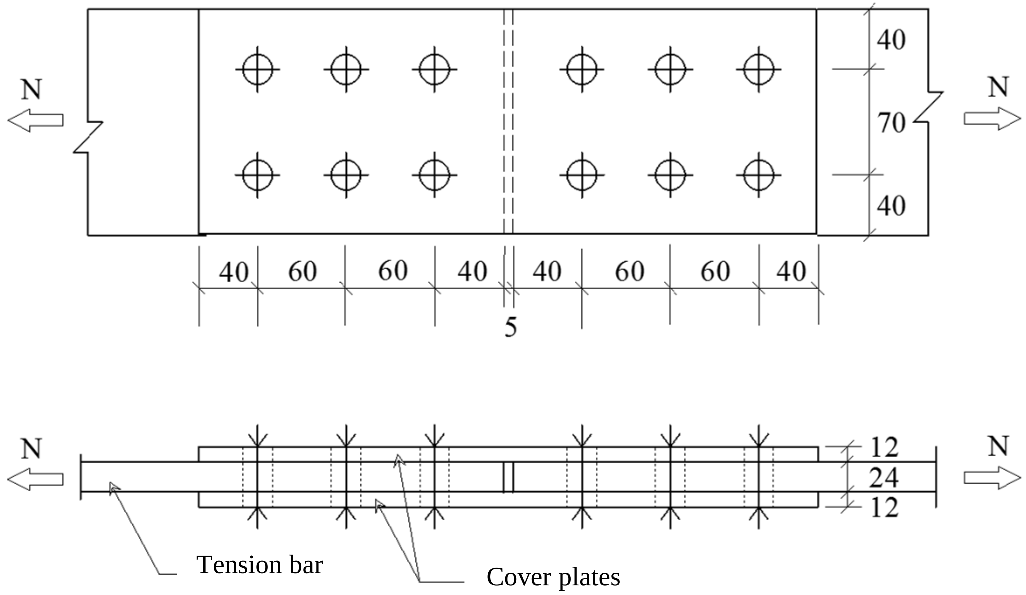
Fire design of columns - 6

EXAMPLE 6



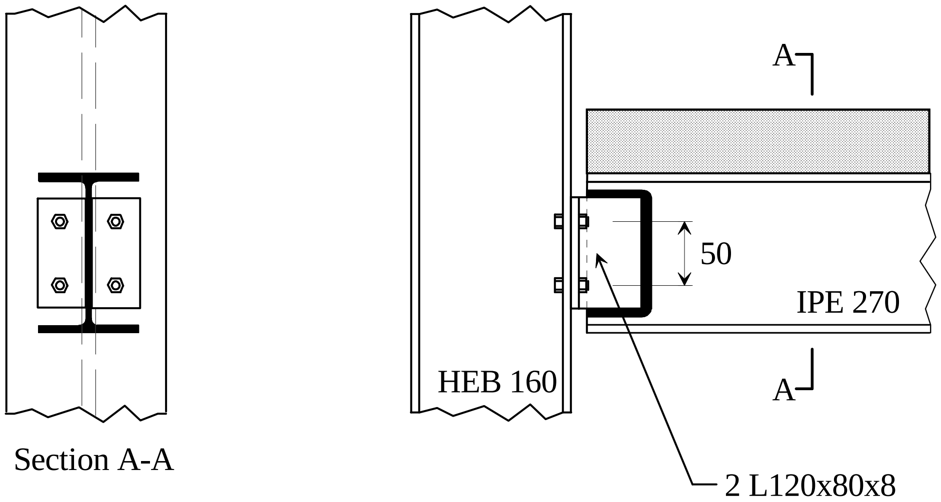
Fire resistance of connections

Bolted and welded connections



Fire resistance of connections

Annex D from EN 1993-1-2



Annex D

D1 – Bolted connections

D2 – Design Resistance of Welds

D3 – Temperature of connections in fire

Fire resistance of connections - Annex D from EN 1993-1-2

Design Resistance of Bolts in Shear and in Tension

Design Resistance of Bolts in Shear:

$$F_{v,t,Rd} = F_{v,Rd} k_{b,\theta} \frac{\gamma_{M2}}{\gamma_{M,fi}}$$

Design bearing resistance of bolts:

$$F_{b,t,Rd} = F_{b,Rd} k_{b,\theta} \frac{\gamma_{M2}}{\gamma_{M,fi}}$$

Design tension resistance:

$$F_{ten,t,Rd} = F_{t,Rd} k_{b,\theta} \frac{\gamma_{M2}}{\gamma_{M,fi}}$$

Design Resistance of welds

Design resistance of a fillet weld:

$$F_{w,t,Rd} = F_{w,Rd} k_{w,\theta} \frac{\gamma_{M2}}{\gamma_{M,fi}}$$

Fire resistance of connections - Annex D from EN 1993-1-2

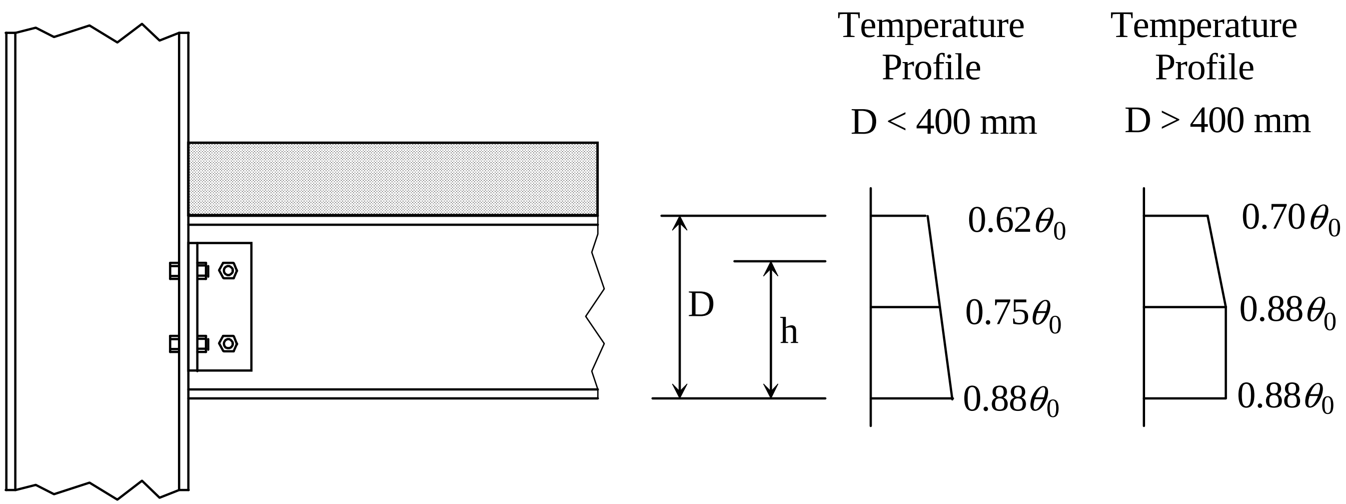
Strength Reduction Factors for Bolts and Welds

Table D.1: Strength Reduction Factors for Bolts and Welds

Temperature θ_a	Reduction factor for bolts, $k_{b,\theta}$ (Tension and shear)	Reduction factor for welds, $k_{w\,\theta}$
20	1,000	1,000
100	0,968	1,000
150	0,952	1,000
200	0,935	1,000
300	0,903	1,000
400	0,775	0,876
500	0,550	0,627
600	0,220	0,378
700	0,100	0,130
800	0,067	0,074
900	0,033	0,018
1000	0,000	0,000

Fire resistance of connections - Annex D from EN 1993-1-2

Temperature of connections in fire

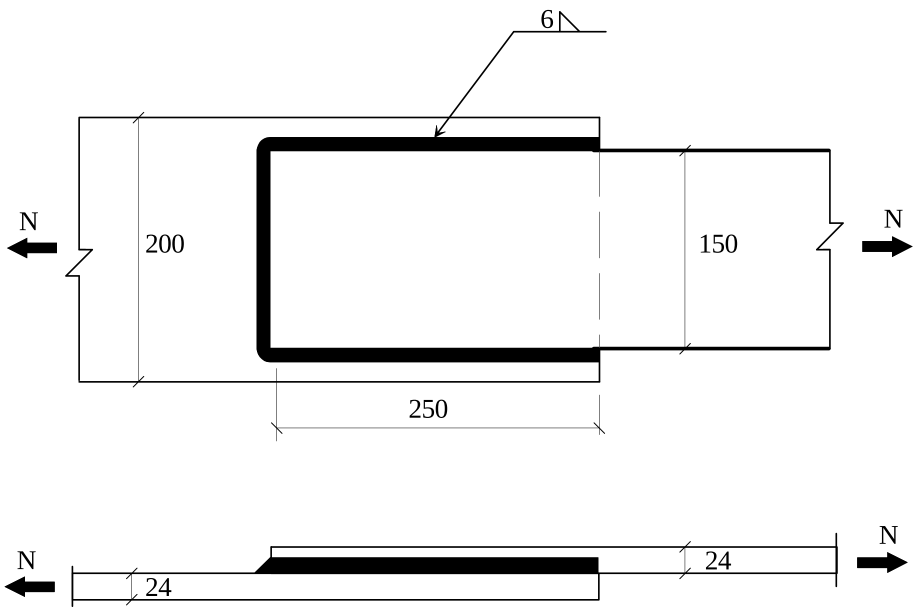


θ_0 - is the bottom flange temperature of the steel beam remote from the connection

Fire resistance of welded connections - 1

Example 7

Consider a welded tension joint in S355 steel, as shown in figure. Assuming that the design value of the tension force in the fire situation is $N_{fi,Ed} = 190\,\text{kN}$ and that the throat thickness of the fillet welds is 6 mm, verify if the unprotected joint has a fire resistance of R30.



Welded tension joint

Fire resistance of welded connections - 2

Example 7

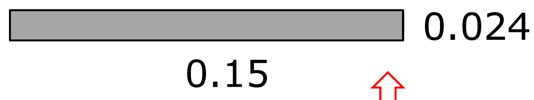
Solution:

Verification of the fire resistance of the gross cross-section

The smallest gross cross-section of the connected members in tension is $150 \times 24 \mathrm{mm}^2$. The corresponding section factor is

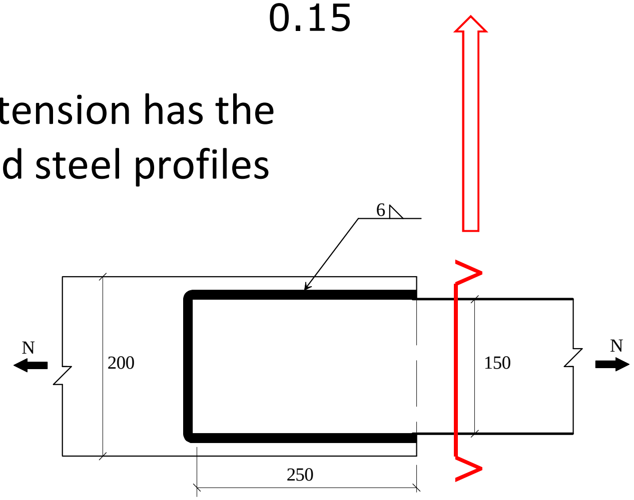
$$\left[\frac{A_m}{V}\right] = \frac{2(0.024+0.15)}{0.024 \cdot 0.15} = 96.7\,\mathrm{m}^{-1}$$

Gross cross-section of the member in tension



After 30 minutes of standard fire exposure the member in tension has the following temperatures obtained from Table of unprotected steel profiles using $k_{sh} = 1.0$

$$\theta_{ar} = 763\,^{\circ}\mathrm{C}$$



Fire resistance of welded connections - 3

Example 7

Verification of the fire resistance of the gross cross section

For a temperature of 763°C, the reduction factor for the effective yield strength, has the value

$$k_{y,763°\mathrm{C}} = 0.1544$$

The gross cross-section design resistance $N_{fi,\theta,Rd}$ of the tension member with a uniform temperature θ_a = 763°C is:

$$N_{fi,\theta,Rd} = Ak_{y,763°\mathrm{C}} f_y / \gamma_{M,fi} =$$
$$= 24 \cdot 150 \cdot 0.1544 \cdot 355 / 1.0 = 197 \times 10^3 \text{ N} = 197 \text{ kN}$$

This is higher than the applied tension load in fire situation $N_{fi,Ed} = 190$ kN.

Fire resistance of welded connections - 4

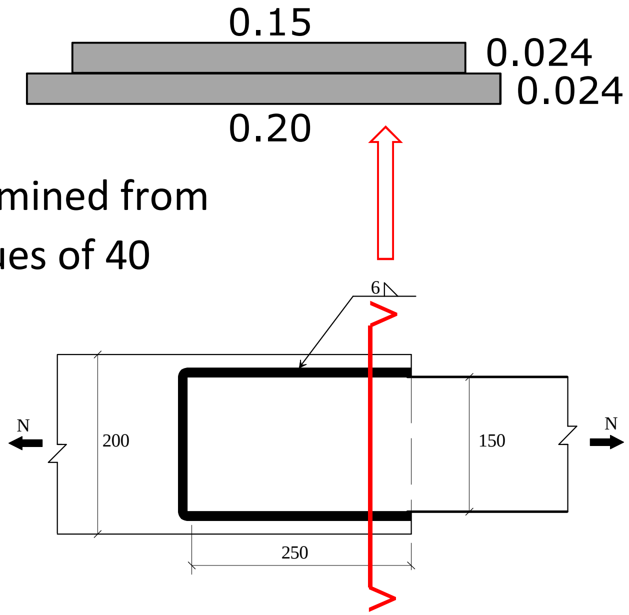
Example 7

Verification of the fire resistance of the fillet weld

The section factor, A_m/V, of the joint can be determined by considering a cross section perpendicular to the main direction of the joint.

$$\frac{A_m}{V} = \frac{0.2 + 4 \cdot 0.024 + 2 \cdot 0.025 + 0.15}{0.2 \cdot 0.024 + 0.15 \cdot 0.024} = 59\,\mathrm{m}^{-1}$$

Gross cross-section of the joint



Assuming $k_{sh} = 1.0$, the temperature of the joint can be determined from Table of unprotected profiles by interpolating between the values of 40 and 60 m^{-1}.

$$\theta = 717\ ^{\circ}\mathrm{C}$$

Fire resistance of welded connections - 5

Example 7

According to the simplified method for design resistance of fillet weld presented in EN 1993-1-8, the design resistance of the fillet weld may be assumed to be adequate if, at every point along its length, the resultant of all the forces per unit length transmitted by the weld satisfy the following criterion:

$$F_{w,Rd} = \frac{f_u / \sqrt{3}}{\beta_w \gamma_{M2}} a$$

where

$f_u = 510\,\mathrm{N/mm^2}$, for the steel grade S355

$\beta_w = 0.9$, for the steel grade S355

$a = 6\,\mathrm{mm}$ (effective throat thickness of the fillet weld)

$\gamma_{M2} = 1.25$

leading to

$$F_{w,Rd} = \frac{f_u / \sqrt{3}}{\beta_w \gamma_{M2}} a = \frac{510/\sqrt{3}}{0.9 \cdot 1.25} \cdot 6 = 1570\,\mathrm{N/mm} = 1.57\,\mathrm{kN/mm}$$

Fire resistance of welded connections - 6

Example 7

The design resistance per unit length of the fillet weld in fire should be determined from the following equation:

$$F_{w,t,Rd} = F_{w,Rd} k_{w,\theta} \frac{\gamma_{M2}}{\gamma_{M,fi}}$$

The reduction factor for welds can be determined from the Table by interpolating between 700 and 800 ^{o}C:

$$k_{w,717^{o}\mathrm{C}} = 0.12$$

leading to

$$F_{w,t,Rd} = 1.57 \cdot 0.12 \cdot \frac{1.25}{1.0} = 0.236 \,\mathrm{kN/mm}$$

Fire resistance of welded connections - 7

Example 7

Multiplying this value by the total length of the fillet weld ($l = 650\,\mathrm{mm}$) the design value of the fire resistance of the fillet weld is:

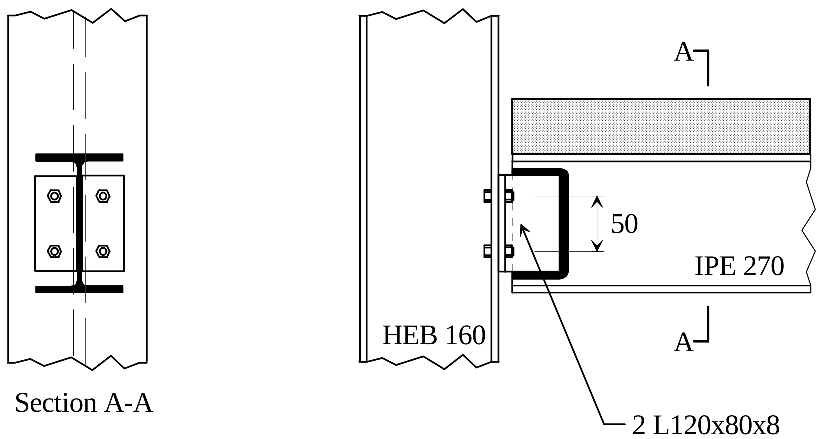
$$F_{w,t,Rd,TOTAL} = 0.236 \cdot 650 = 153\,\mathrm{kN}$$

This value is lower than the applied axial load in the fire situation, $N_{fi,Ed} = 190\,\mathrm{kN}$. The required fire resistance of R30 is not achieved and fire protection should be used or the thickness of the fillet weld should be increased (a thickness of 8 mm would be enough).

Fire resistance of bolted connections - 1

Example 8

Consider a beam-to-column joint between an IPE 270 beam and a HE 160 B column in steel grade S235 connected by a double angle cleat $\mathrm{L}120\times80\times8$, welded to the web of the beam and bolted with Grade 4.6 M16 to the flange of the column, as shown in the figure. Assuming that the design value of the shear force at the connection in fire situation is $V_{fi,Ed}=30$ kN, verify the shear resistance of the bolts in fire situation, if the shear plane passes through the unthreaded portion of the bolt and the required fire resistance is R30.



Double angle cleat. Beam-to-column connection

Fire resistance of bolted connections - 2

Example 8

Solution:

The relevant characteristics of the IPE 270 to solve this design example are:

$$b = 135\,\text{mm}$$

$$t_f = 6.6\,\text{mm}$$

The section factor of the bottom flange of the beam is:

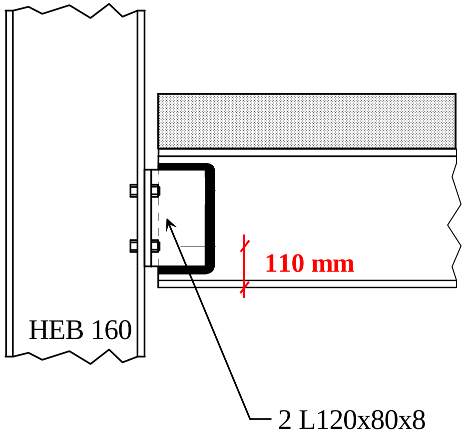
$$\frac{A_m}{V} = \frac{2(b + t_f)}{bt_f} = \frac{2 \cdot (0.135 + 0.0066)}{0.135 \cdot 0.0066} = 318\,\text{m}^{-1}$$

Making a conservative assumption of $k_{sh} = 1.0$, the temperature of the bottom flange of the beam can be determined from Table of temperature of unprotected profiles by interpolating between 300 and 400 m^{-1}. This gives a temperature after 30 minutes of:

$$\theta_0 = 835.4\ ^\circ\text{C}$$

Fire resistance of bolted connections - 3

Example 8



The height of the first row of bolts above the bottom of the beam is $h = 110\,\text{mm}$, and accordingly to the given equation in EN 1993-1-2, its temperature takes the value:

$$\theta_h = 0.88 \cdot 835.4 \cdot [1 - 0.3 \cdot (110/270)] = 645\ ^\circ\text{C}$$

At this temperature the strength reduction factor for bolts takes the value:

$$k_{b,645^\circ\text{C}} = 0.166$$

The design shear resistance of the bolt per shear plane calculated assuming that the shear plane passes through the threads portion of the bolt, as EN 1993-1-2 imposes, is, for a M16 bolts of Class 4.6 at normal temperature, given in Part 1-8 of Eurocode 3:

$$F_{v,Rd} = \frac{0.6 f_{ub} A_s}{\gamma_{M2}} = \frac{0.6 \cdot 400 \cdot 157}{1.25} = 30.1 \times 10^3\ \text{N} = 30.1\text{kN}$$

Fire resistance of bolted connections - 4

Example 8

The design value of the shear resistance of a bolt after 30 minutes of standard fire exposure is

$$F_{v,t,Rd} = F_{v,Rd} k_{b,645°\mathrm{C}} \frac{\gamma_{M2}}{\gamma_{M,fi}} = 30.1 \cdot 0.166 \cdot \frac{1.25}{1.0} = 6.25\,\mathrm{kN}$$

This value is lower than the applied shear load on the bolts, i.e., $30/4 = 7.5\,\mathrm{kN}$ (4 bolts and only one shear plane) and thus the bolts do not satisfy the fire resistance criterion R30.

References

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• EN 1990, EN 1991-1-2, EN 1993-1-1, EN 1993-1-2; EN 1993-1-5

Thank you for your attention

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