SUSCOS

FIRE

Composite steel-concrete structures

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1.Introduction

There are different types of composite members.

Different verification methods have been developed for each type.

In this course, we will treat:

1)Composite floors

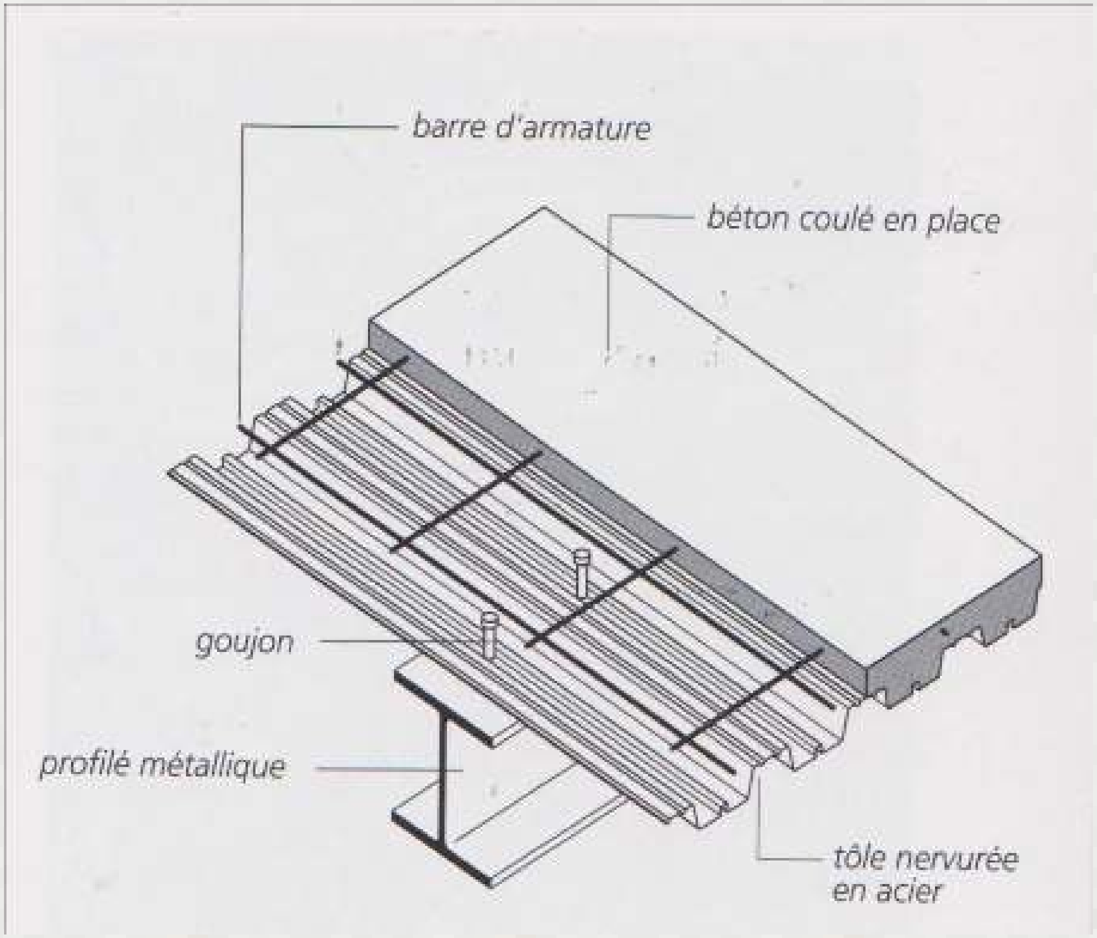
2)Composite columns

3)Composite beams

The verification methods presented here are based on Eurocode 4 (En 1994-1-2),

Most of the illustrations are from the book *Construction mixte*, Maquoi R, Debruyckere R, Demonceau J-F & Lincy P, Infosteel, Brussels, 2012

2. Verification of the composite slab

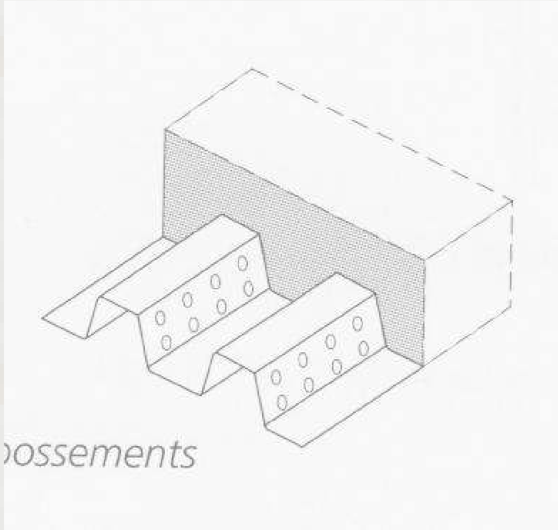


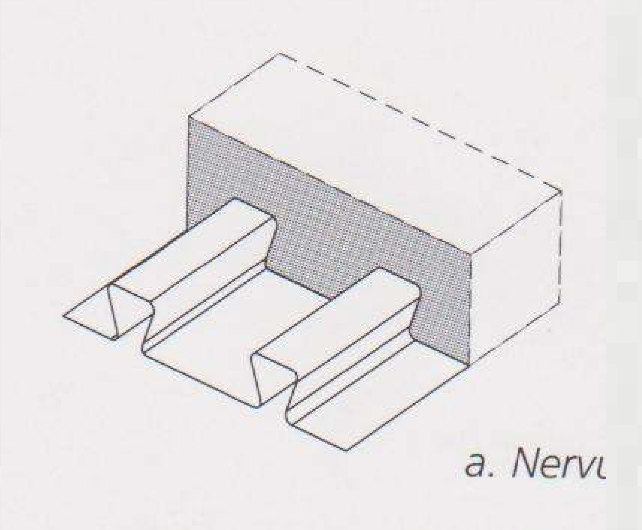
Principle of a composite steel concrete floor

Notes:

1) the steel beam is a support for the floor,

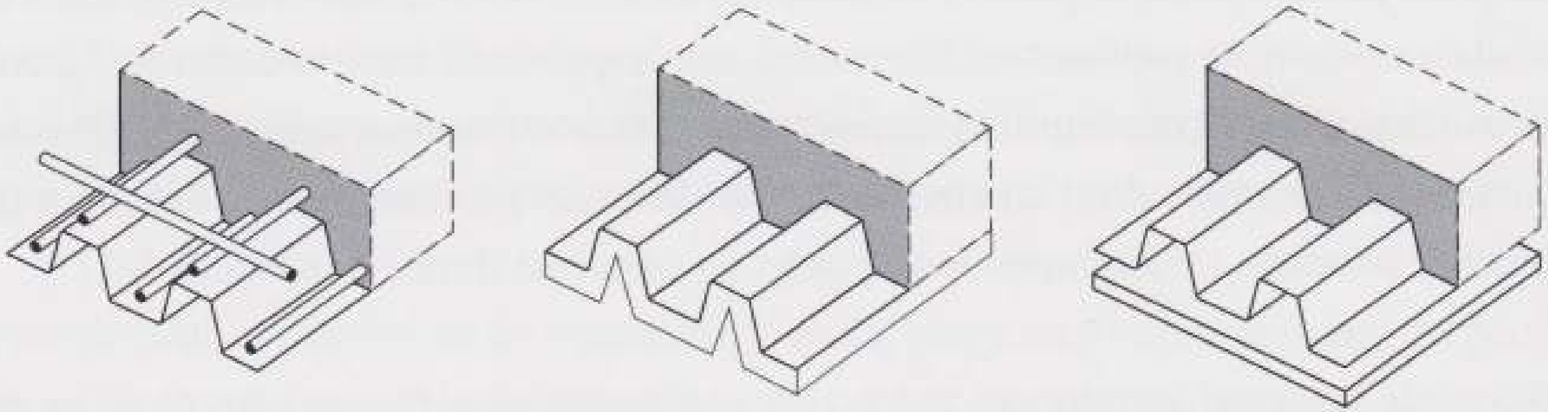
2) The headed studs are there to generate composite action in the beam





Trapezoidal profile

Re-entrant profile



Three different techniques for improving the fire resistance R.

From left to right:

1)Additional re-bars in the ribs (method considered in the Eurocode, has no effect on I)

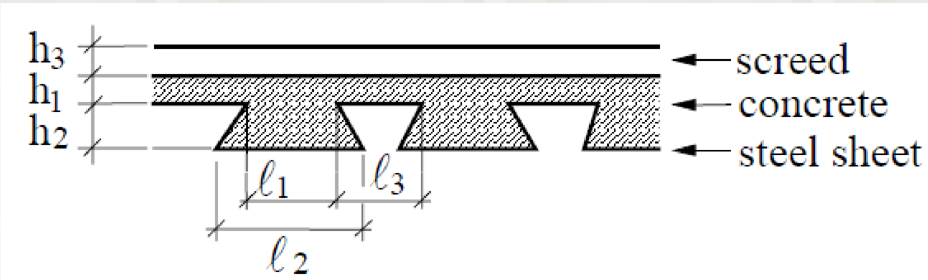
2)Projected insulation (beneficial also for I)

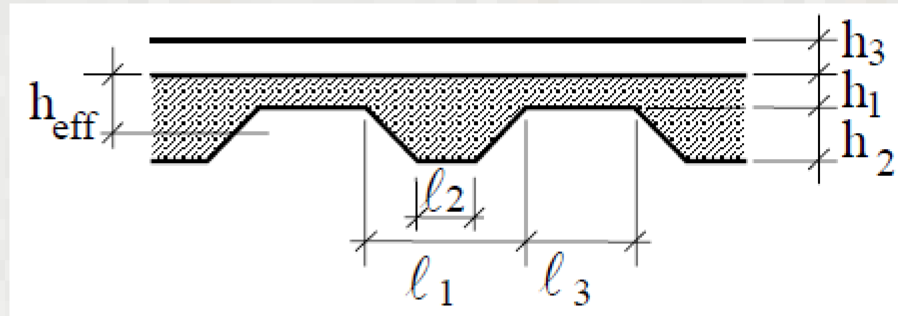
3)Suspended ceiling (beneficial also for I)

Note: tensile membrane action is a more modern and more efficient technique

A composite floor designed according to EN 1994-1-1 under room temperature conditions is deemed to have a fire resistance REI_{30}

2.1. Field of application for unprotected composite slabs





for re-entrant steel sheet profiles					for trapezoidal steel profiles				
77,0 ≤	ℓ_1	≤	135,0	mm	80,0 ≤	ℓ_1	≤	155,0	mm
110,0 ≤	ℓ_2	≤	150,0	mm	32,0 ≤	ℓ_2	≤	132,0	mm
38,5 ≤	ℓ_3	≤	97,5	mm	40,0 ≤	ℓ_3	≤	115,0	mm
30,0 ≤	h_1	≤	60,0	mm	50,0 ≤	h_1	≤	100,0	mm
50,0 ≤	h_2	≤	130	mm	50,0 ≤	h_2	≤	100,0	mm

2. Verification of the composite slab

2.1. Field of application for unprotected composite slabs

2.2. Integrity: E

For composed slabs, E is supposed to be satisfied.

2. Verification of the composite slab

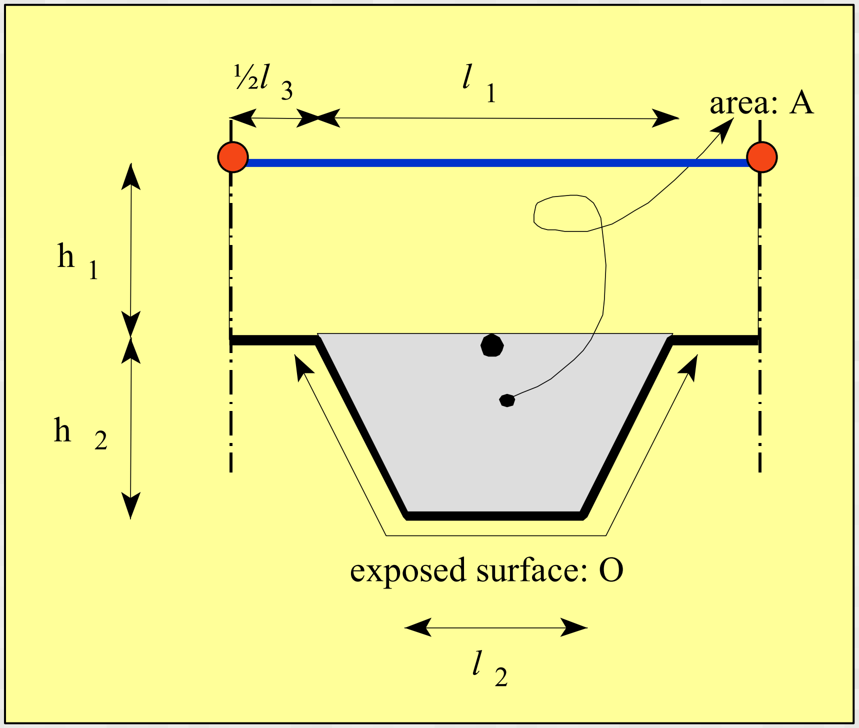
2.1. Field of application for unprotected composite slabs

2.2. Integrity: E

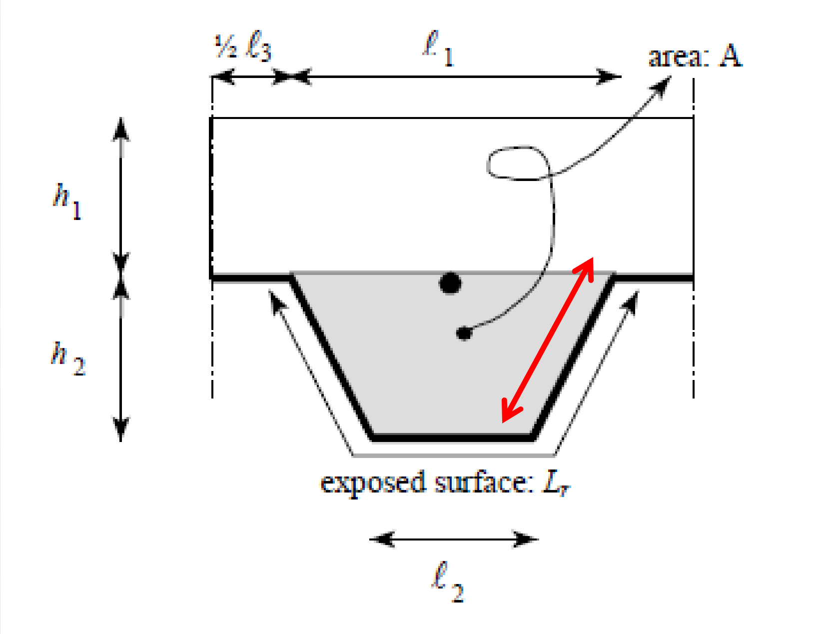
2.3. Thermal insulation: I

Criteria:

- ✓ **ΔT_{max} ≤ 180 K**
- ✓ **$\Delta T_{average}$ ≤ 140 K**



Method 1 for evaluating criteria I

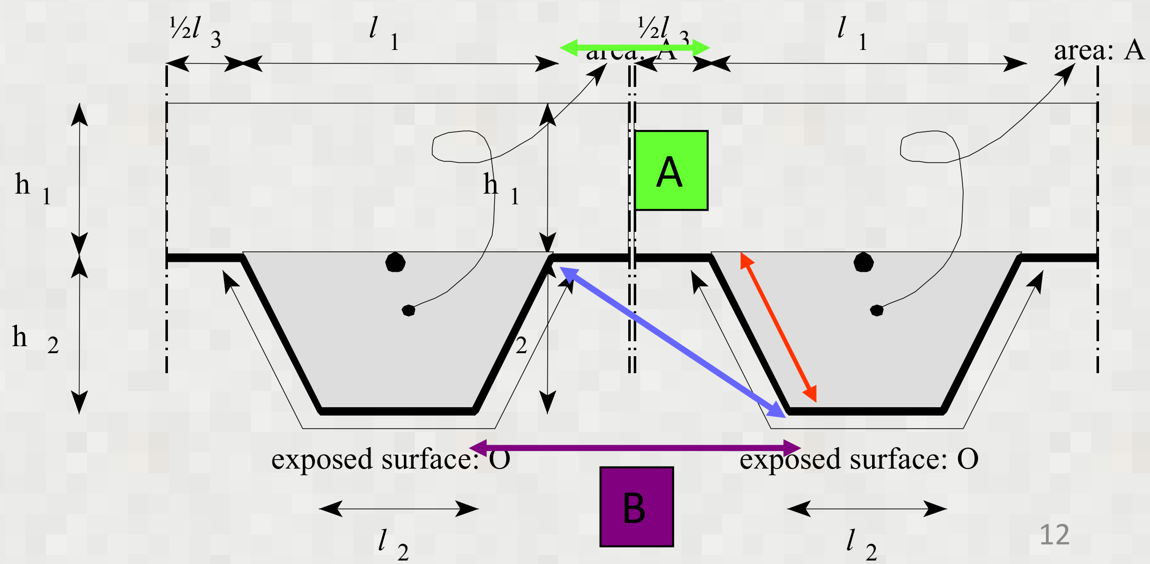


1) Evaluate the equivalent thickness (massivity) of the rib, in mm.

$$\frac{A}{L_r} = \frac{h_2 \cdot \left(\frac{\ell_1 + \ell_2}{2}\right)}{\ell_2 + 2\sqrt{h_2^{\,2} + \left(\frac{\ell_1 - \ell_2}{2}\right)^2}}$$

2) Evaluate the view factor Φ between the surface of the slab that is not protected by the ribs « A » and the opening between two ribs « B »

$$\Phi = \left(\sqrt{h_2^2 + \left(l_3 + \frac{l_1 - l_2}{2} \right)^2} - \sqrt{h_2^2 + \left(\frac{l_1 - l_2}{2} \right)^2} \right) \Big/ l_3$$

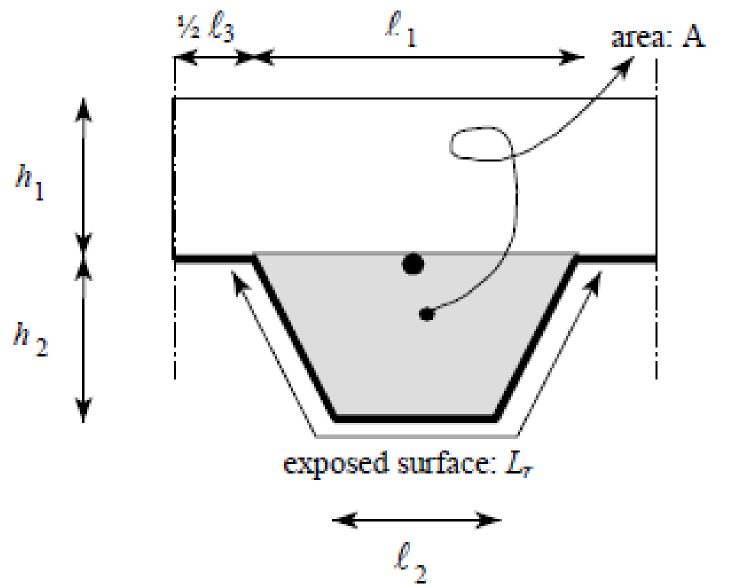


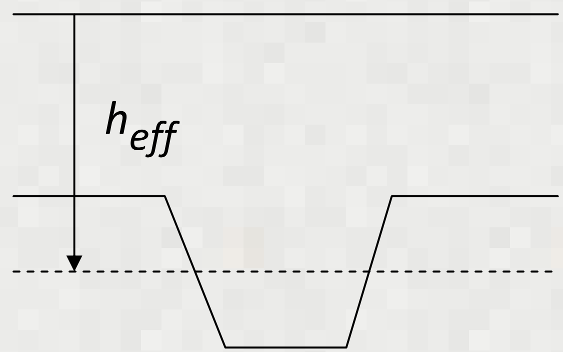
3) Calculate the fire resistance to I, in min, by the following best fit polynom (constants valid for normal weigth concrete).

$$t_I = -28{,}8 + 1{,}55\, h_1 - 12{,}6\, \Phi + 0{,}33\, A/L_r - 735\, 1/l_3 + 48\, A/L_r\, 1/l_3$$

Method 2 for evaluating criteria I

1) Calculate the effective depth of the floor h_{eff}





$$h_{eff} = h_1 + 0{,}5\, h_2 \left(\frac{\ell_1 + \ell_2}{\ell_1 + \ell_3} \right) \quad \text{for } h_2/h_1 \le 1{,}5 \text{ and } h_1 > 40 \text{ mm}$$

$$h_{eff} = h_1 \left[1 + 0{,}75 \left(\frac{\ell_1 + \ell_2}{\ell_1 + \ell_3} \right) \right] \quad \text{for } h_2/h_1 > 1{,}5 \text{ and } h_1 > 40 \text{ mm}$$

$$h_{eff} = h_1 \quad \text{for } l_3 > 2\, l_1$$

Method 2 for evaluating criteria I

2) The Table underneath gives the minimum value of the effective thickness to be provided, with h_3 the thickness of the eventual creed (additional top layer of non structural concrete)

Standard Fire Resistance	Minimum effective thickness h_{eff} [mm]
R 30	60 - h_3
R 60	80 - h_3
R 90	100 - h_3
R 120	120 - h_3
R 180	150 - h_3
R 240	175 - h_3

Example: for I 30,

$h_{eff} \geq 60 - h_3$

$h_{eff} + h_3 \geq 60$

Load bearing capacity: R

The bending capacity has to be determined by a plastic design.

Bending capacity in sagging, $M_{fi,Rd}^{+}$

(1) The temperature θ_a of the lower flange, web and upper flange of the steel decking may be given by:

$$\theta_a = b_0 + b_1 \cdot \frac{1}{\ell_3} + b_2 \cdot \frac{A}{O} + b_3 \cdot \Phi + b_4 \cdot \Phi^2 \qquad \text{(D.2.1)}$$

Table D.2.1: Coefficients for the determination of the temperatures of the parts of the steel decking

Concrete	Fire resistance [min]	Part of the steel sheet	b_0 [°C]	b_1 [°C]. mm	b_2 [°C]. mm	b_3 [°C]	b_4 [°C]
Normal weight concrete	60	Lower flange	951	-1197	-2,32	86,4	-150,7
		Web	661	-833	-2,96	537,7	-351,9
		Upper flange	340	-3269	-2,62	1148,4	-679,8
	90	Lower flange	1018	-839	-1,55	65,1	-108,1
		Web	816	-959	-2,21	464,9	-340,2
		Upper flange	618	-2786	-1,79	767,9	-472,0
	120	Lower flange	1063	-679	-1,13	46,7	-82,8
		Web	925	-949	-1,82	344,2	-267,4
		Upper flange	770	-2460	-1,67	592,6	-379,0

(3) The temperature θ_s of the reinforcement bars in the rib, if any according to figure D.2.1, as follows:

$$\theta_s = c_0 + c_1 \cdot \frac{u_3}{h_2} + c_2 \cdot z + c_3 \cdot \frac{A}{O} + c_4 \cdot \alpha + c_5 \cdot \frac{1}{\ell_3} \qquad \text{(D.2.2)}$$

where:

θ_s	the temperature of additional reinforcement in the rib	[°C];
u_3	distance to lower flange	[mm];
z	indication of the position in the rib (see (4))	[$mm^{-0.5}$];
α	angle of the web	[degrees];

$$\frac{1}{z} = \frac{1}{\sqrt{u_1}} + \frac{1}{\sqrt{u_2}} + \frac{1}{\sqrt{u_3}} \qquad \text{(D.2.3)}$$

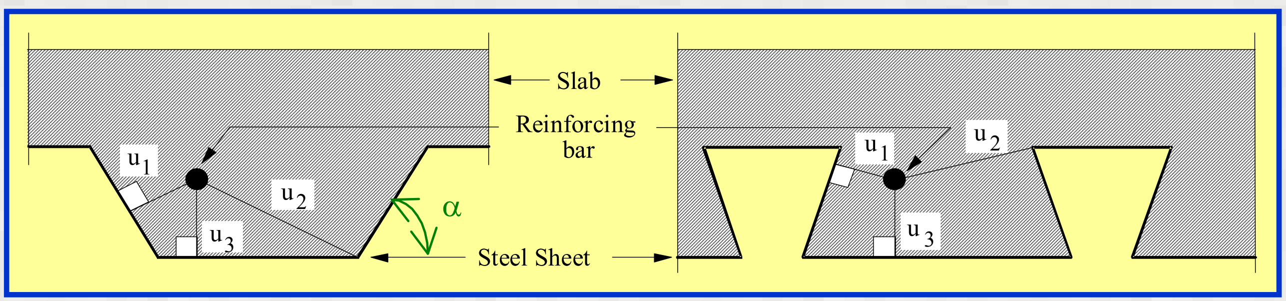


Table D.2.2: Coefficients for the determination of the temperatures of the reinforcement bars in the rib.

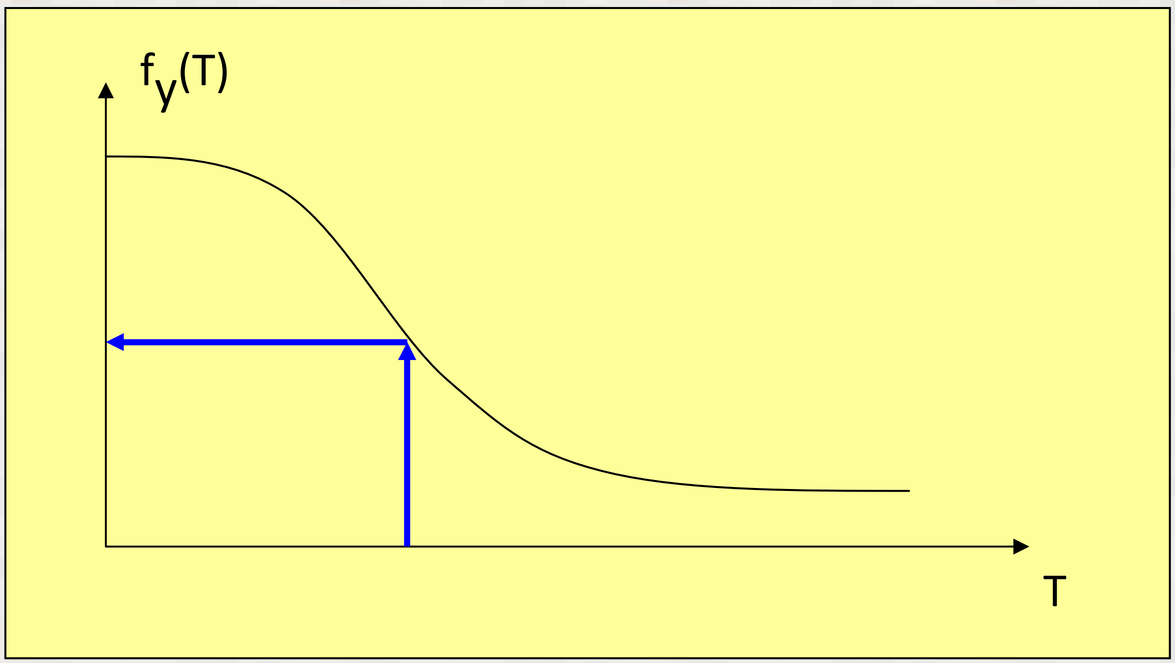
Concrete	Fire resistance [min]	c_0 [^{o}C]	c_1 [^{o}C]	c_2 [^{o}C]. mm$^{0.5}$	c_3 [^{o}C].mm	c_4 [^{o}C/^{o}]	c_5 [^{o}C].mm
Normal weight concrete	60	1191	-250	-240	-5,01	1,04	-925
	90	1342	-256	-235	-5,30	1,39	-1267
	120	1387	-238	-227	-4,79	1,68	-1326
Light weight concrete	30	809	-135	-243	-0,70	0,48	-315
	60	1336	-242	-292	-6,11	1,63	-900
	90	1381	-240	-269	-5,46	2,24	-918
	120	1397	-230	-253	-4,44	2,47	-906

Temperatures in the steel sheet and in the reinforcement bars

+

Material model

$\Big\} \Rightarrow f_y\,(T)$



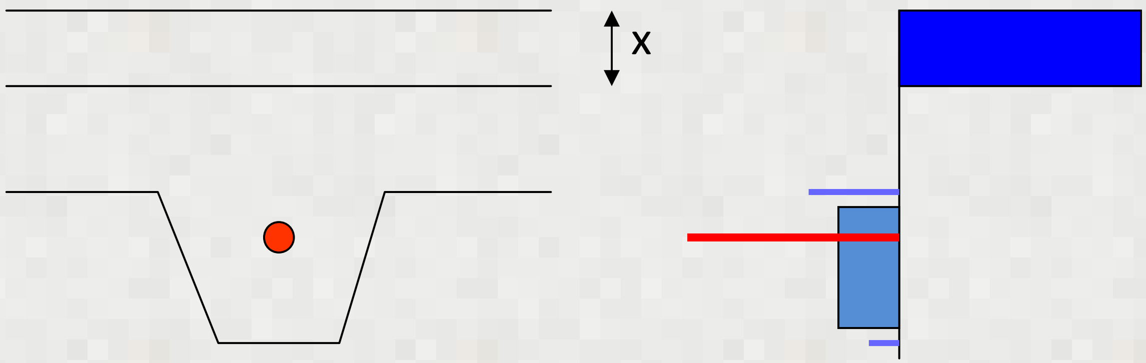
T	$f_y(T)/f_y$	$f_c(T)/f_c$
20	1.00	1.00
100	1.00	1.00
200	1.00	0.95
300	1.00	0.85
400	1.00	0.75
500	0.78	0.60
600	0.47	0.45
700	0.23	0.30
800	0.11	0.15
900	0.06	0.08
1000	0.04	0.04
1100	0.02	0.01
1200	0.00	0.00

Neutral axis

(4) The plastic neutral axis of a composite slab or composite beam may be determined from:

$$\sum_{i=1}^{n} A_i k_{y,\theta,i} \left(\frac{f_{y,i}}{\gamma_{M,fi,a}} \right) + \alpha_{slab} \sum_{j=1}^{m} A_j k_{c,\theta,j} \left(\frac{f_{c,j}}{\gamma_{M,fi,c}} \right) = 0 \qquad (4.2)$$

$\alpha_{slab} = 0{,}85.$



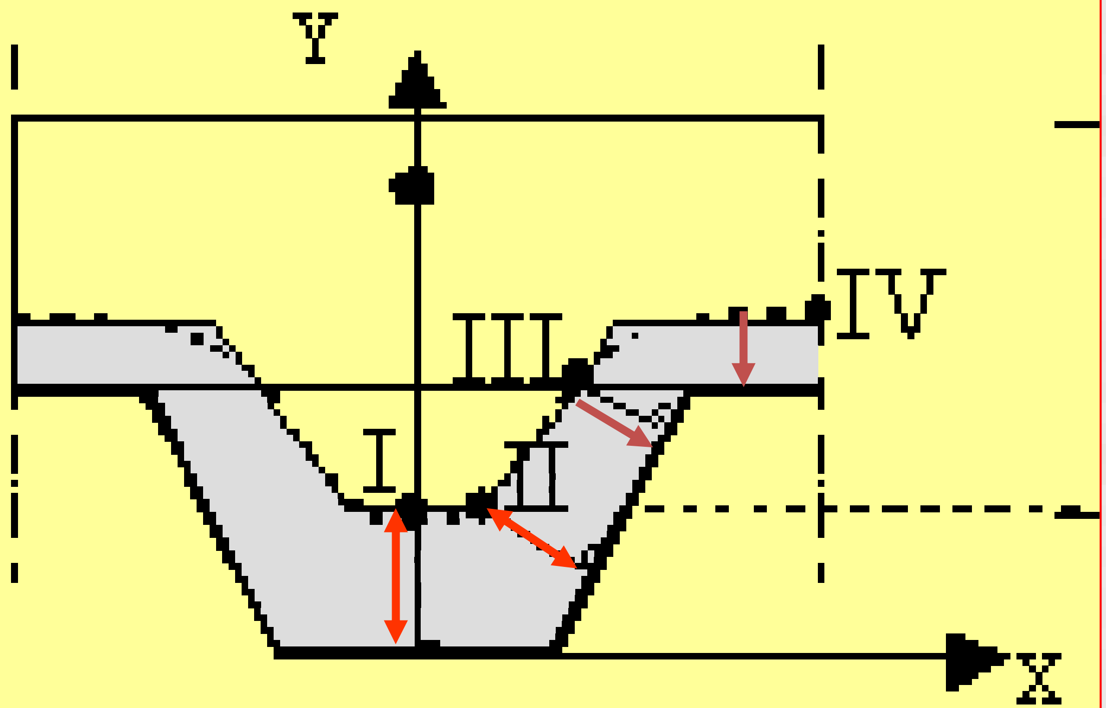
Design moment resistance

(5) The design moment resistance $M_{fi,t,Rd}$ may be determined from:

$$M_{fi,t,Rd} = \sum_{i=1}^{n} A_i z_i k_{y,\theta,i} \left(\frac{f_{y,i}}{\gamma_{M,fi}} \right) + \alpha_{slab} \sum_{j=1}^{m} A_j z_j k_{c,\theta,j} \left(\frac{f_{c,j}}{\gamma_{M,fi,c}} \right) \qquad (4.3)$$

Bending capacity in hogging, M_{fi,Rd^-} D.3

B) Schematisation specfic isotherm $\vartheta = \vartheta_{lim}$



(4) The limiting temperature, θ_{lim} is given by:

$$\theta_{lim} = d_0 + d_1 \cdot N_s + d_2 \cdot \frac{A}{L_r} + d_3 \cdot \Phi + d_4 \cdot \frac{1}{\ell_3} \qquad \text{(D.7)}$$

Put $\theta_s = \theta_{lim}$ and $u_3/h_2 = 0.75$ in D.2.2 and find z

$$\theta_s = c_0 + c_1 \cdot \frac{u_3}{h_2} + c_2 \cdot z + c_3 \cdot \frac{A}{O} + c_4 \cdot \alpha + c_5 \cdot \frac{1}{\ell_3} \qquad \text{(D.2.2)}$$

$$X_I = 0 \qquad \text{(D.3.5)}$$

$$Y_I = Y_{II} = \frac{1}{\left(\frac{1}{z} - \frac{4}{\sqrt{L_1 - L_2}}\right)^2} \qquad \text{(D.3.6)}$$

$$X_{II} = \frac{1}{2} L_2 + \frac{Y_I}{\sin\alpha} \cdot (\cos\alpha - 1) \qquad \text{(D.3.7)}$$

with: $\alpha = \arctan\left(\frac{2\,h_2}{L_1 - L_2}\right)$

$$X_{III} = \frac{1}{2} L_1 - \frac{b}{\sin\alpha} \qquad \text{(D.3.8)}$$

with: $a = \left(\frac{1}{z} - \frac{1}{\sqrt{h_2}}\right)^2 L_1 \sin\alpha$

$$Y_{III} = h_2 \qquad \text{(D.3.9)}$$

with: $b = \frac{1}{2} L_1 \sin\alpha \left(1 - \frac{\sqrt{a^2 - 4ac}}{a}\right)$

$a^2 - 4a + c$

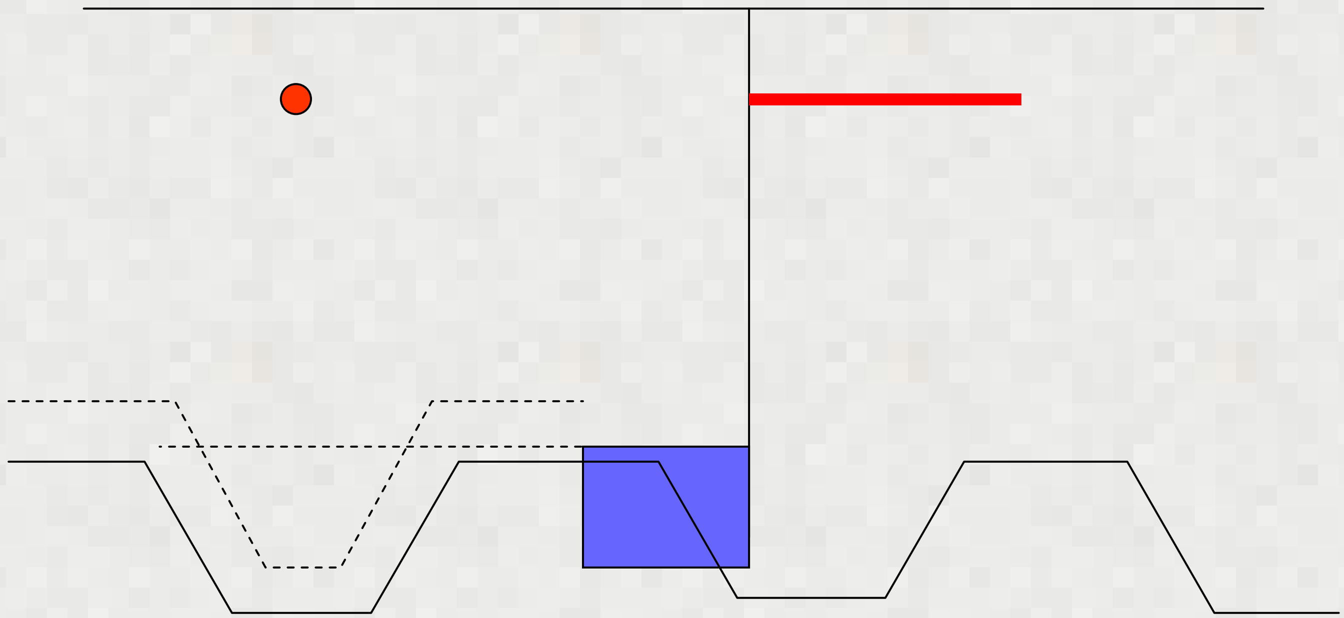
$$X_{IV} = \frac{1}{2} L_1 \qquad \text{(D.3.10)}$$

$(L_1 + L_3)/2$

with: $c = -8\left(1 + \sqrt{1 + a}\right);\ a \geq 8$

$$Y_{IV} = h_2 + b \qquad \text{(D.3.11)}$$

with: $c = +8\left(1 + \sqrt{1 + a}\right);\ a < 8$

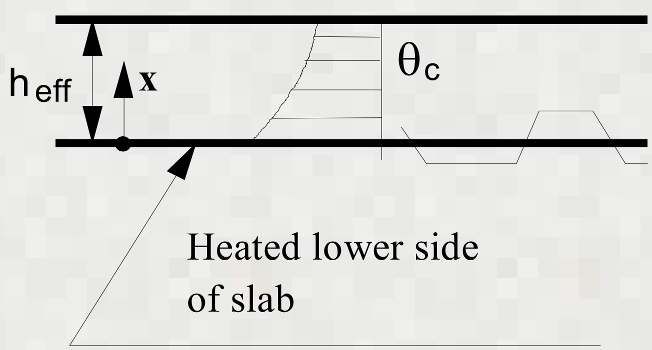


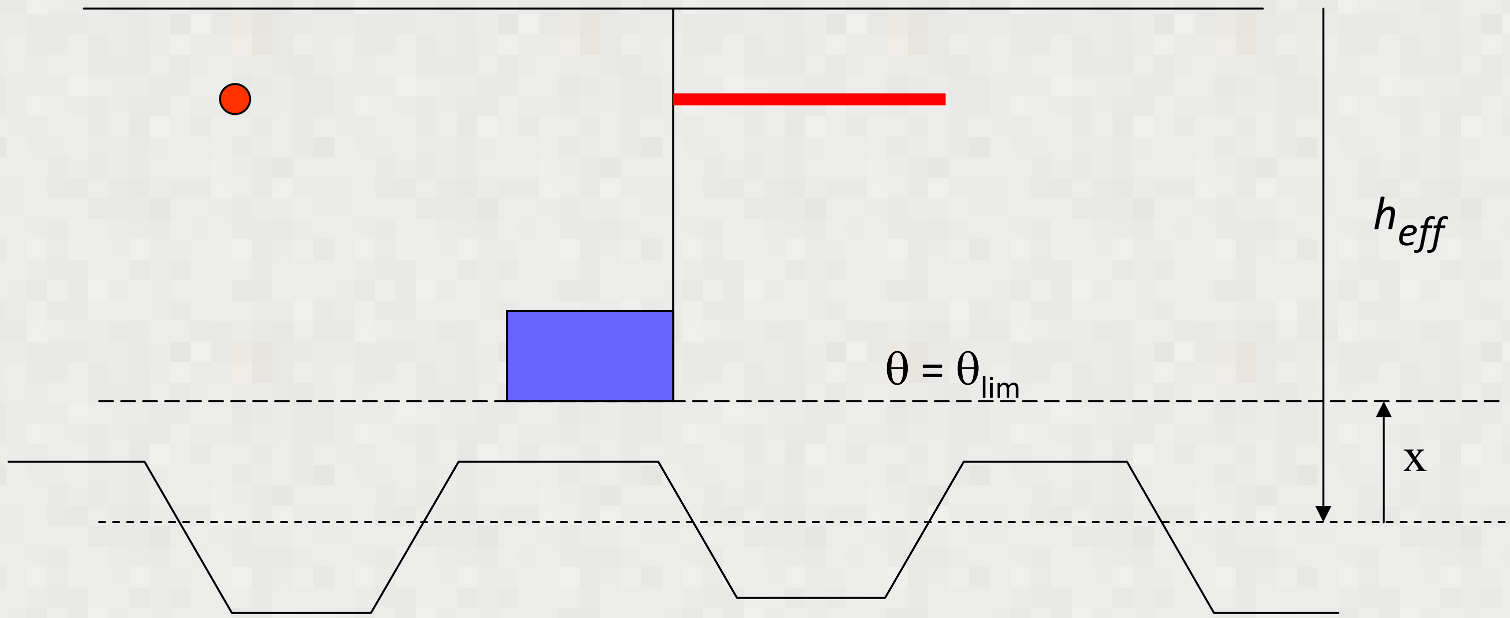
Alternative procedure if $Y_I > h_2$.

Table D.5 may be used to obtain the location of the isotherm as a conservative approximation; for lightweight concrete, Table D.5 may be used as well.

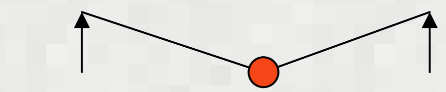
Table D.5: Temperature distribution in a solid slab of 100 mm thickness composed of normal weight concrete and not insulated.

Depth x mm	Temperature θ_C[°C] after a fire duration in min. of 30'	60'	90'	120'	180'	240'
5	535	705				
10	470	642	738			
15	415	581	681	754		
20	350	525	627	697		
25	300	469	571	642	738	
30	250	421	519	591	689	740
35	210	374	473	542	635	700
40	180	327	428	493	590	670
45	160	289	387	454	549	645
50	140	250	345	415	508	550
55	125	200	294	369	469	520
60	110	175	271	342	430	495
80	80	140	220	270	330	395
100	60	100	160	210	260	305

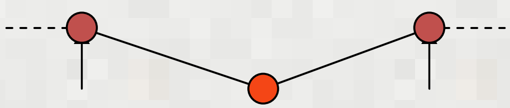



h_{eff}
θ = θ_{lim}
x

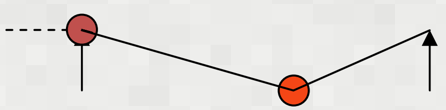
Bending capacity of the element



$$P L^2 / 8 = M^+$$



$$P L^2 / 8 = M^+ + M^-$$

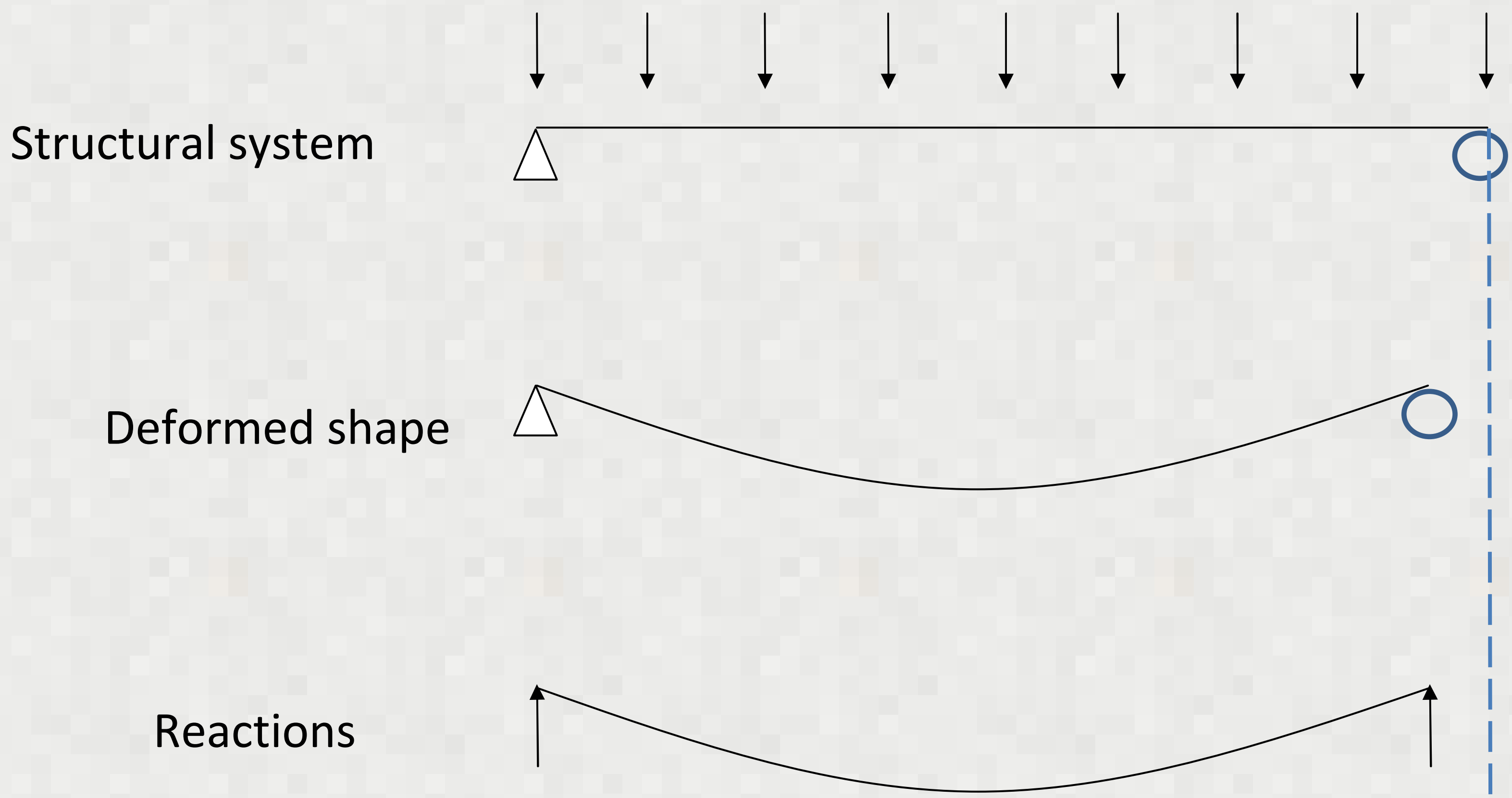


$$P L^2 / 8 \approx M^+ + M^-/2$$

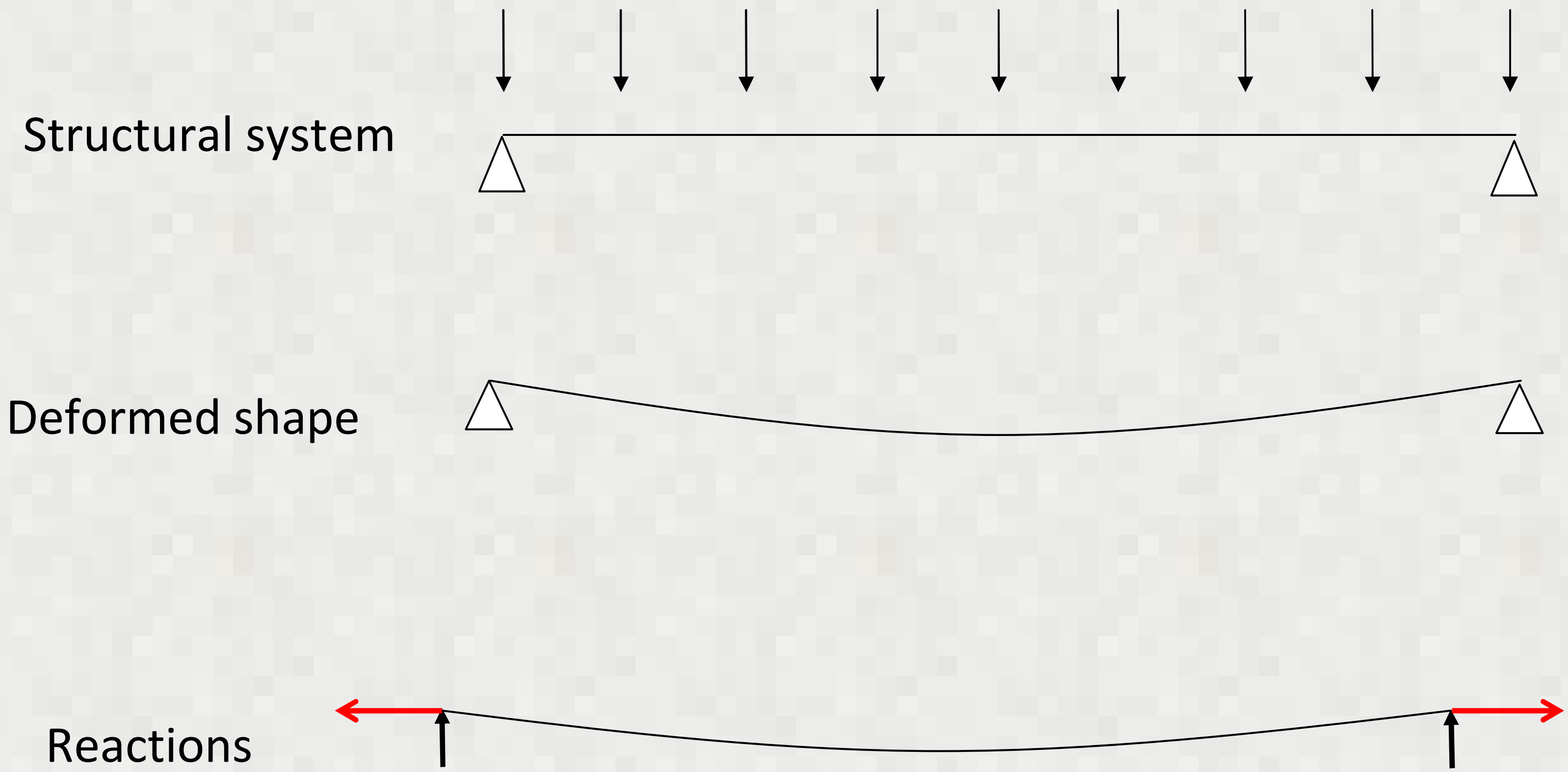
$$P L^2 / 8 = M^+/2 + M^-/4 + [(M^- + 2M^+)^2 - (M^-)^2]^{0.5}/4$$

COMPOSITE SLAB IN TENSILE MEMBRANE ACTION

- **One way bending element simply supported**



- **One way element in catenary action**



1. Catenary action is not easily activated in one way building elements.
2. Membrane action can be activated in 2 way spanning slabs.

=> Tensile membrane action

How does it work?

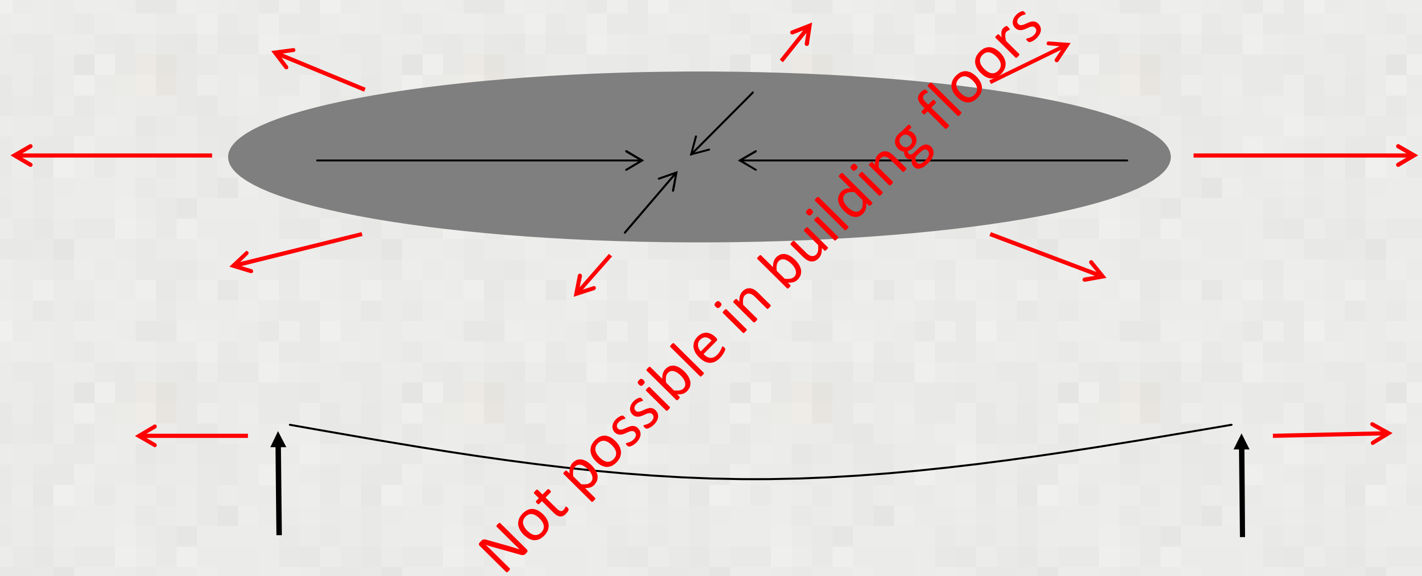
The concrete slab is like a piece of fabric.

When it is highly deformed, it is subjected to in plane tensile forces (supported by the steel mesh).

Question: how are these tensile forces being equilibrated ?

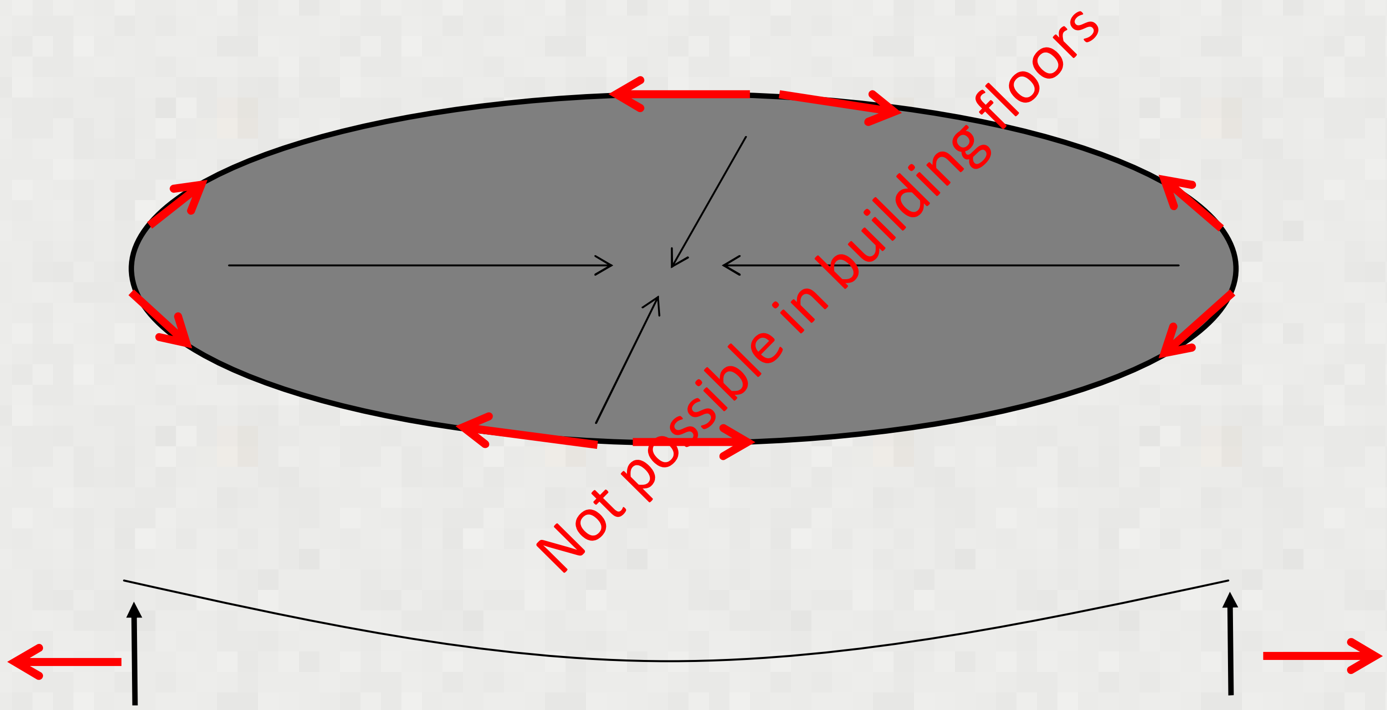
Question: how are these tensile forces being equilibrated ?

1 : by external horizontal forces



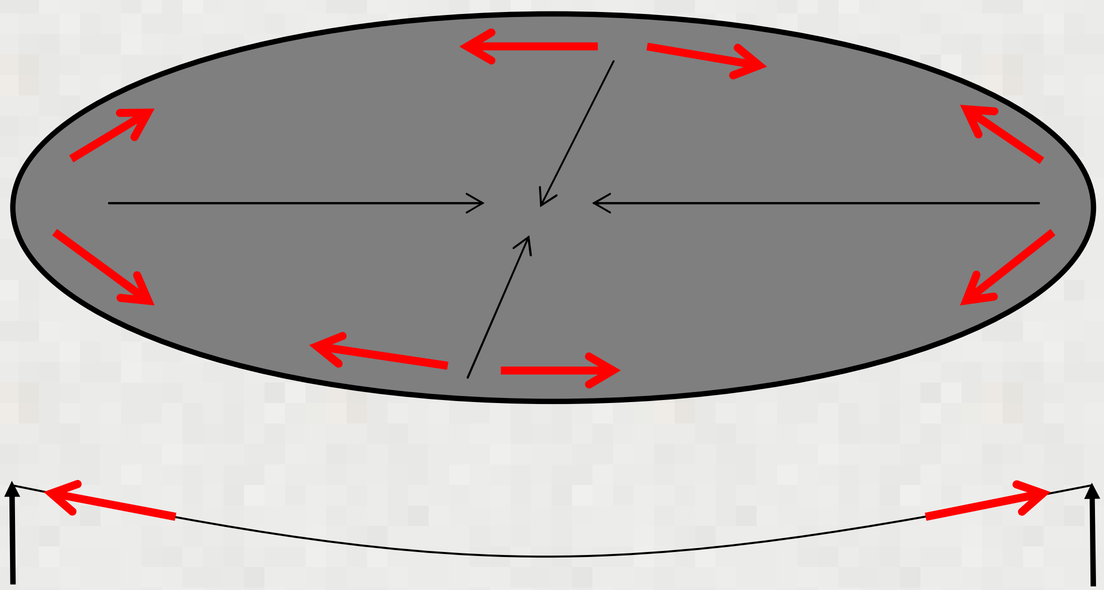
Question: how are these tensile forces being equilibrated ?

2 : by peripheric structural elements (compression ring)

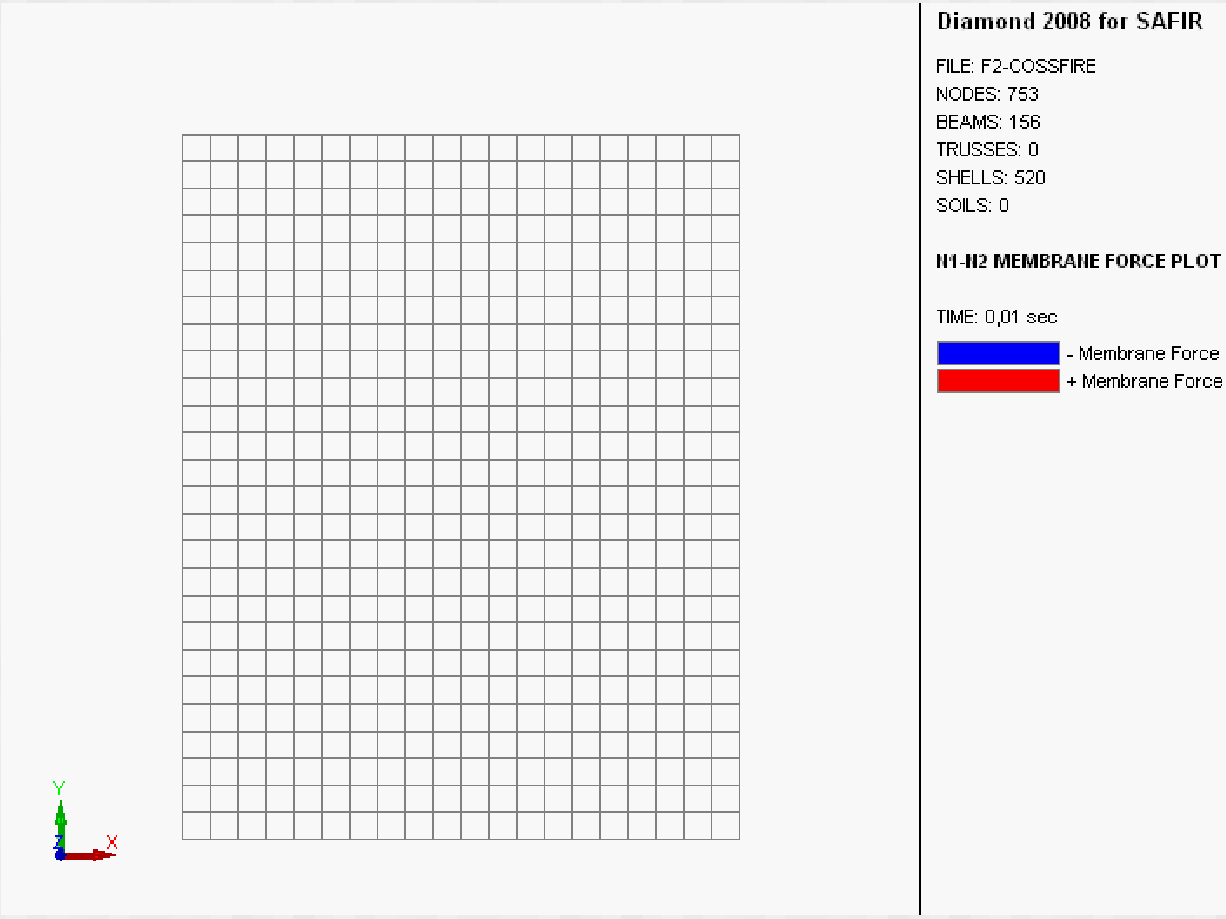


Question: how are these tensile forces being equilibrated ?

3 : by compression in the floor itself

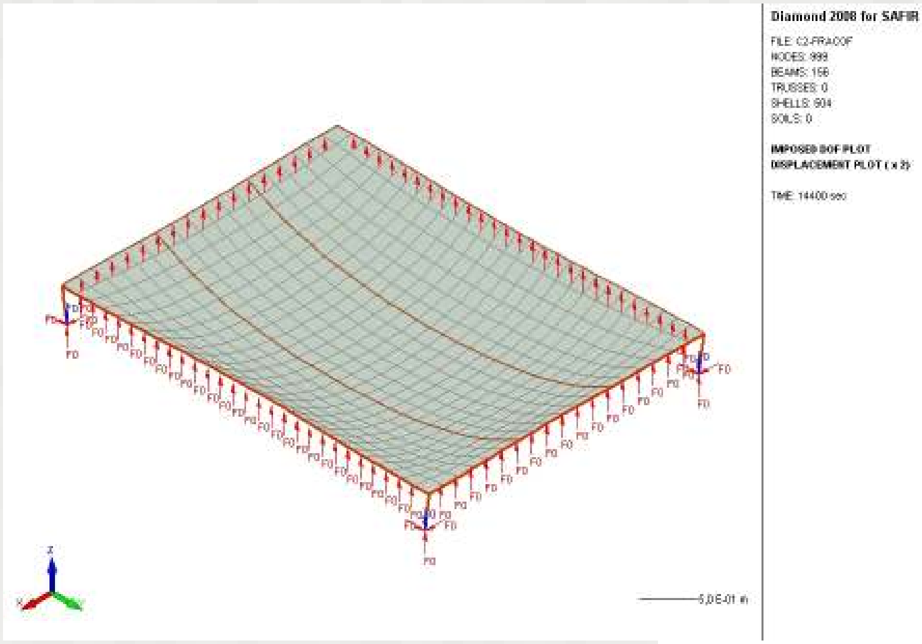


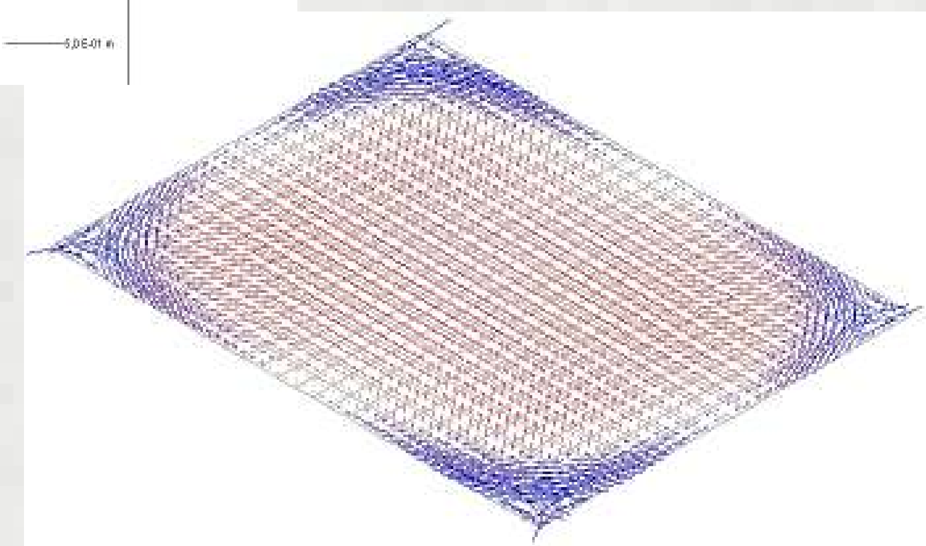
It works also for square or rectangular slabs



SAFIR Simulation

How does it work?





Conceptual design

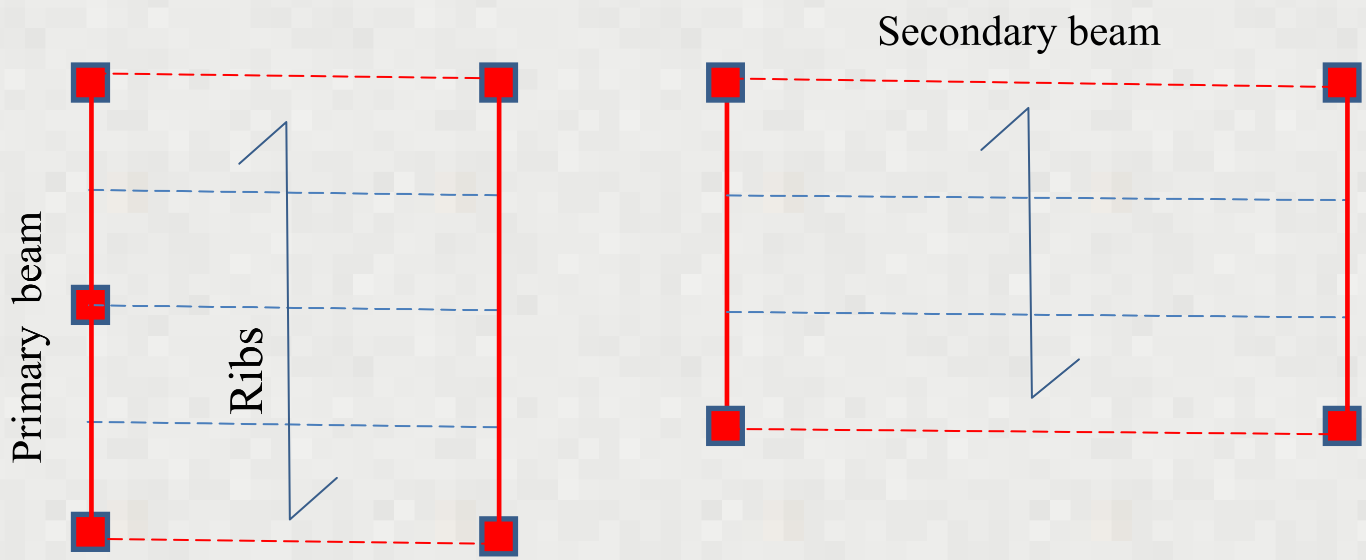
In the plan view:

•Divide the floor into rectangular « slab panels » (with aspect ratio < 3).

•A column must be present at each corner. No column inside the panel.

•Protect the beams at the boundaries of the panels (at least, ensure the vertical support at the edges of the panels).

•Leave the inside beams of the panels unprotected



Conceptual design

Conceptual design

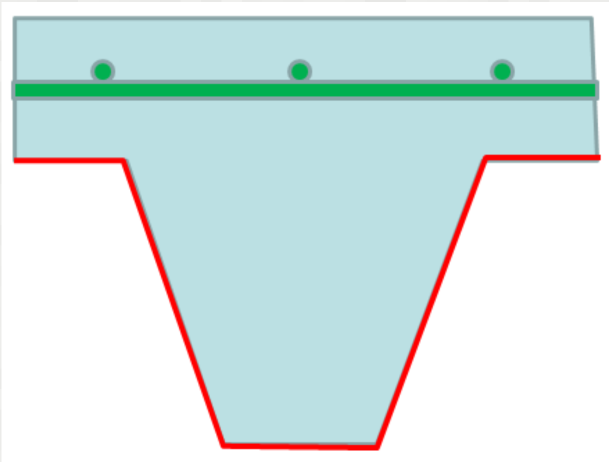
In the section:

•Place a steel mesh in the slab at mid-level above the steel sheets with :

- A sufficient section to support tensile membrane forces (if steel sections are not the same in both directions, put the highest section parallel to the long side of the panel).
- Sufficient cover between adjacent meshes.

•Provide a thickness of the slab that is sufficient to :

- Support membrane compression forces.
- Thermally protect the steel mesh.



Conceptual design

Typical failure modes are:

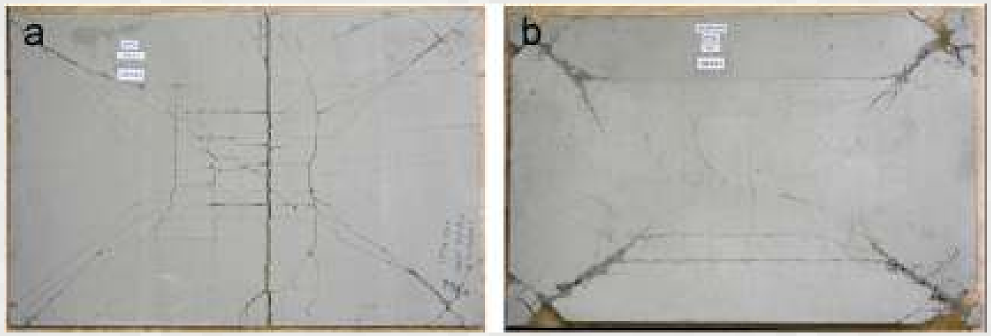
a)Tension in the slab (central crack parallel to the short side of the panel).

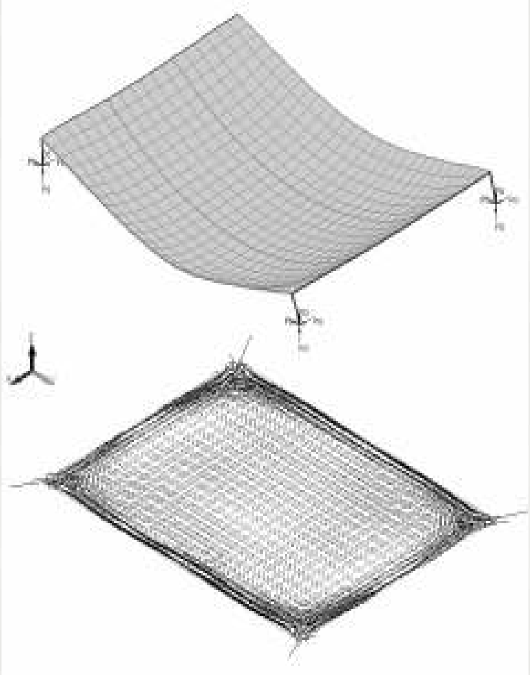
b)Concrete crushing in the compression ring or in the corners of the slab.

c)Bending in the side protected beams (they receive more load than in the room temperature configuration).

d)Joints between:

1. Unprotected beams and protected beams.
2. Protected beams and columns.





1. Introduction

2. Verification of the composite slab

3. Verification of simply supported beams

No tabulated data
=> Simple calculation model: 4.3
!! Method limited to the standard fire

General rules for composite slabs and composite beams: 4.3.1
Composite beams: 4.3.4

3. Verification of simply supported beams

3.1. Determination of the temperatures

See § 4.3.4.2.2

4.3.4.2.2 Heating of the cross-section

Steel beam

(1) When calculating the temperature distribution of the steel section, the cross section may be divided into various parts according to Figure 4.3.

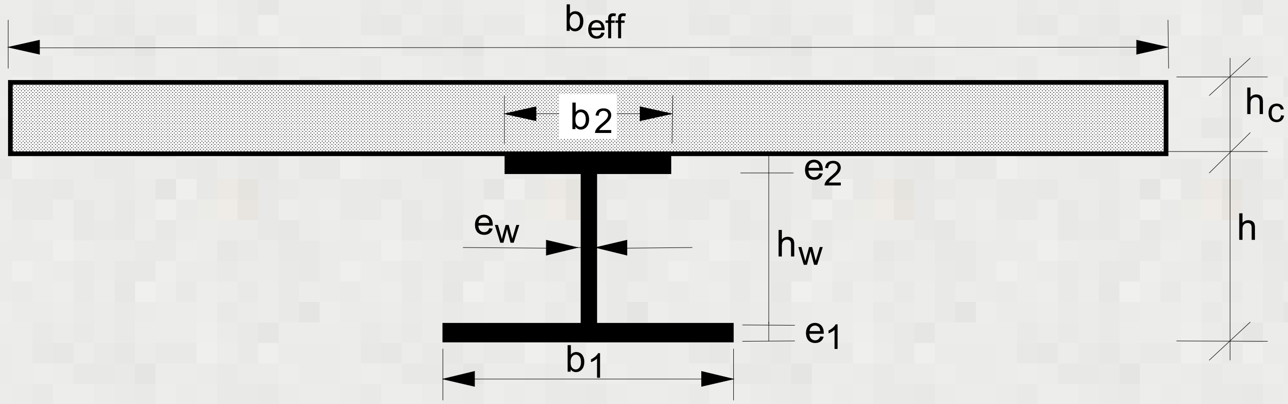


Figure 4.3

(3) The increase of temperature $\Delta\theta_{a,t}$ of the various parts of an **unprotected steel beam** during the time interval Δt may be determined from:

$$\Delta\theta_{a.t} = k_{shadow}\left(\frac{1}{c_a\rho_a}\right)\left(\frac{A_i}{V_i}\right)\dot{h}_{net}\ \Delta t \qquad [°C] \qquad (4.6)$$

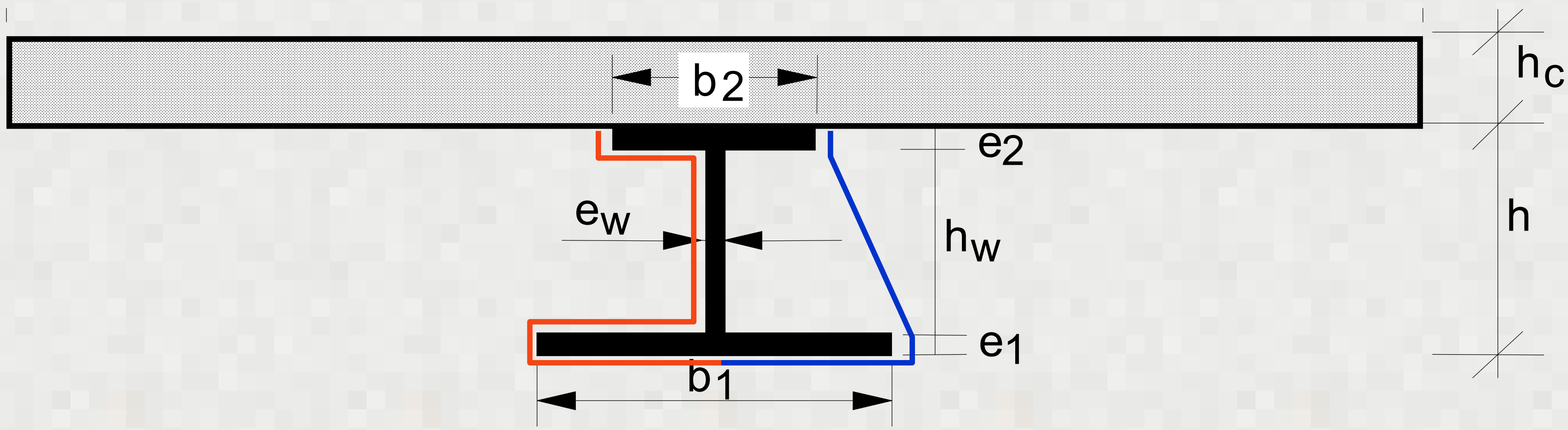


Figure 4.3

(4) The shadow effect may be determined from:

$$k_{shadow} = [0,9] \cdot \frac{e_1 + e_2 + 1/2 \cdot b_1 + \sqrt{h_w^{\ 2} + 1/4 \cdot (b_1 - b_2)^2}}{h_w + b_1 + 1/2 \cdot b_2 + e_1 + e_2 - e_w} \qquad (4.7)$$

$$= 0.9 \frac{\text{box}}{\text{contour}}$$

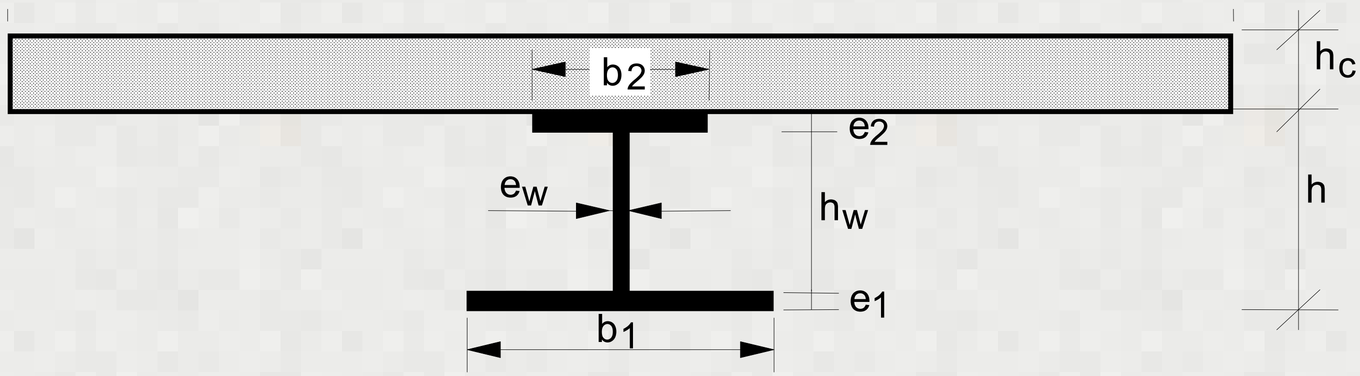


Figure 4.3

Upper flange

$$A_i/V_i \text{ or } A_{p,i}/V_i = (b_2 + 2\,e_2)/b_2\,e_2 \qquad (4.9b)$$

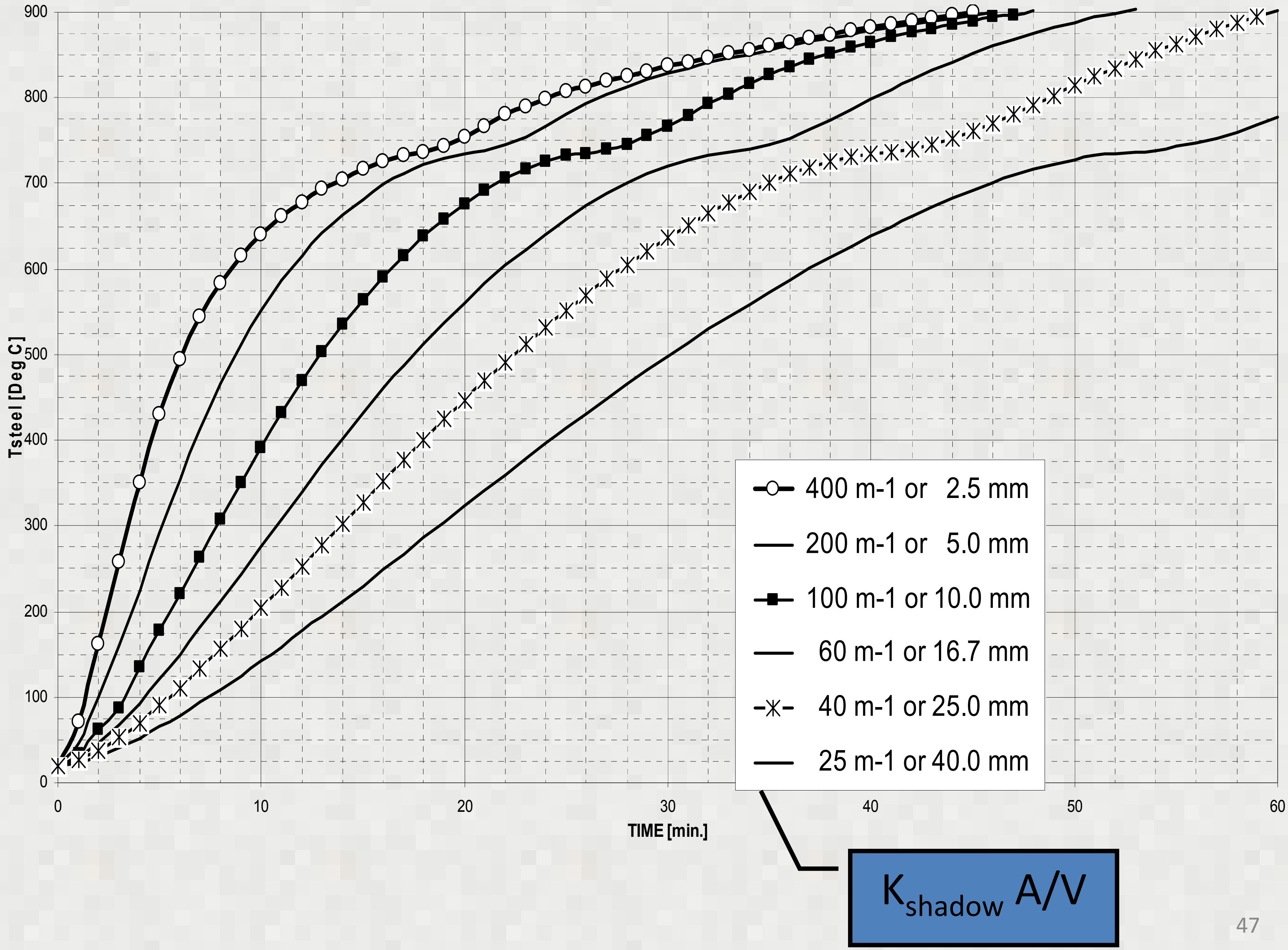
$$A_i/V_i \text{ or } A_{p,i}/V_i = 2(b_2 + e_2)/b_2\,e_2 \qquad (4.9c)$$

Web

(10) If the beam depth h does not exceed 500 mm, the temperature of the web may be taken as equal to that of the lower flange.

Lower flange

$$A_i/V_i \text{ or } A_{p,i}/V_i = 2(b_1 + e_1)/b_1\,e_1 \qquad (4.9a)$$



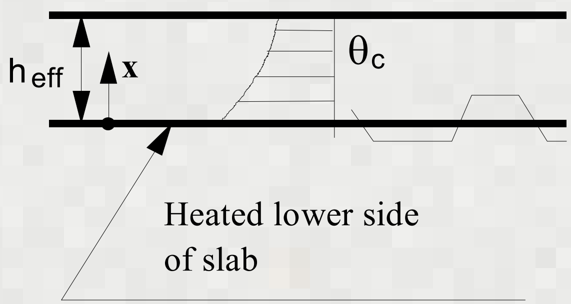
Tsteel [Deg C]
TIME [min.]
400 m-1 or 2.5 mm
200 m-1 or 5.0 mm
100 m-1 or 10.0 mm
60 m-1 or 16.7 mm
40 m-1 or 25.0 mm
25 m-1 or 40.0 mm
K_{shadow} A/V

Temperatures in the slab: See Table D.5

Table D.5 may be used to obtain the location of the isotherm as a conservative approximation; for lightweight concrete, Table D.5 may be used as well.

Table D.5: Temperature distribution in a solid slab of 100 mm thickness composed of normal weight concrete and not insulated.

Depth x	Temperature θ_C[°C] after a fire duration in min. of					
mm	30'	60'	90'	120'	180'	240'
5	535	705				
10	470	642	738			
15	415	581	681	754		
20	350	525	627	697		
25	300	469	571	642	738	
30	250	421	519	591	689	740
35	210	374	473	542	635	700
40	180	327	428	493	590	670
45	160	289	387	454	549	645
50	140	250	345	415	508	550
55	125	200	294	369	469	520
60	110	175	271	342	430	495
80	80	140	220	270	330	395
100	60	100	160	210	260	305



3. Verification of simply supported beams

3.1. Determination of the temperatures

3.2. Structural behaviour – critical temperature model

See § 4.3.4.2.3

If h < 500 mm, h_c > 120 mm, simply supported beam in sagging, then:

for R30 $$0{,}9\ \eta_{fi,t} = f_{ay,\theta cr} / f_{ay} \qquad (4.10a)$$

in any other case $$1{,}0\ \eta_{fi,t} = f_{ay,\theta cr} / f_{ay} \qquad (4.10b)$$

where $\eta_{fi,t} = E_{fi,d,t} / R_d$ and $E_{fi,d,t} = \eta_{fi}\ E_d$ according to (7)P of 4.1 and (3) of 2.4.2.

with θ_{cr} calculated in the lower flange.

3. Verification of simply supported beams

3.1. Determination of the temperatures

3.2. Structural behaviour – critical temperature model

3.3. Structural behaviour – bending moment resistance model

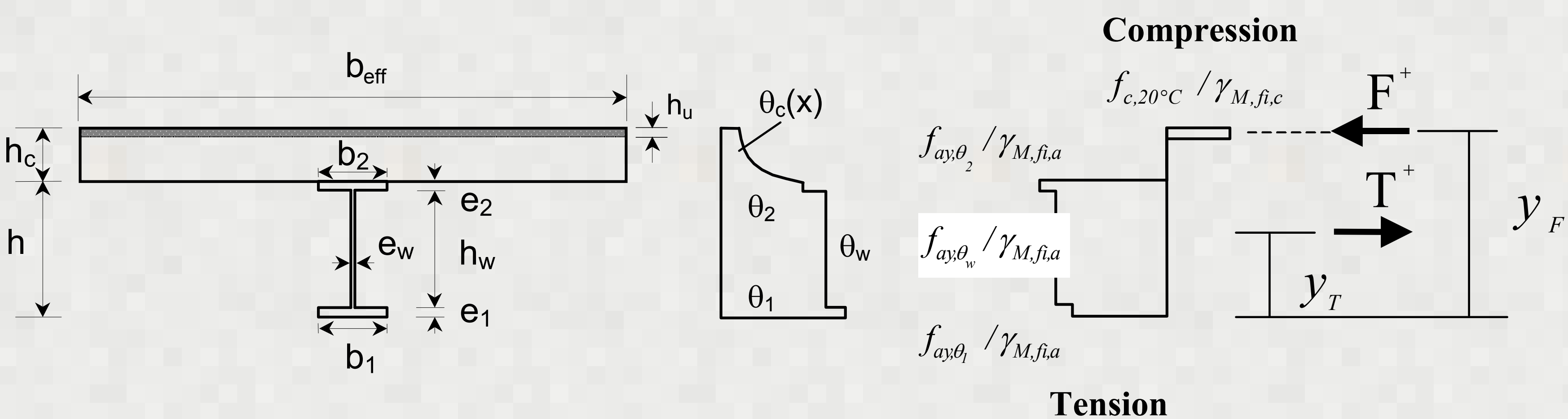
See § 4.3.4.2.4

Classical determination of the bending moment resistance, taking into account the variation of material properties with temperatures, see Annex E.

- No strength reduction in concrete if T < 250°C

- The value of the tensile force is limited by the resistance of the shear connectors:

$$T^{+} \leq N\ P_{fi,Rd}$$



3. Verification of simply supported beams

3.1. Determination of the temperatures

3.2. Structural behaviour – critical temperature model

3.3. Structural behaviour – bending moment resistance model

3.4. Verification of stud connectors

See § 4.3.4.2.5

$P_{fi,Rd}$ = minimum of the 2 following values:

$P_{fi,Rd} = 0,8 \cdot k_{u,\theta} \cdot P_{Rd}$, with P_{Rd} as obtained from equation 6.18 of EN 1994-1-1 or (4.11a)

$P_{fi,Rd} = k_{c,\theta} \cdot P_{Rd}$, with P_{Rd} as obtained from equation 6.19 of EN 1994-1-1 and (4.11b)

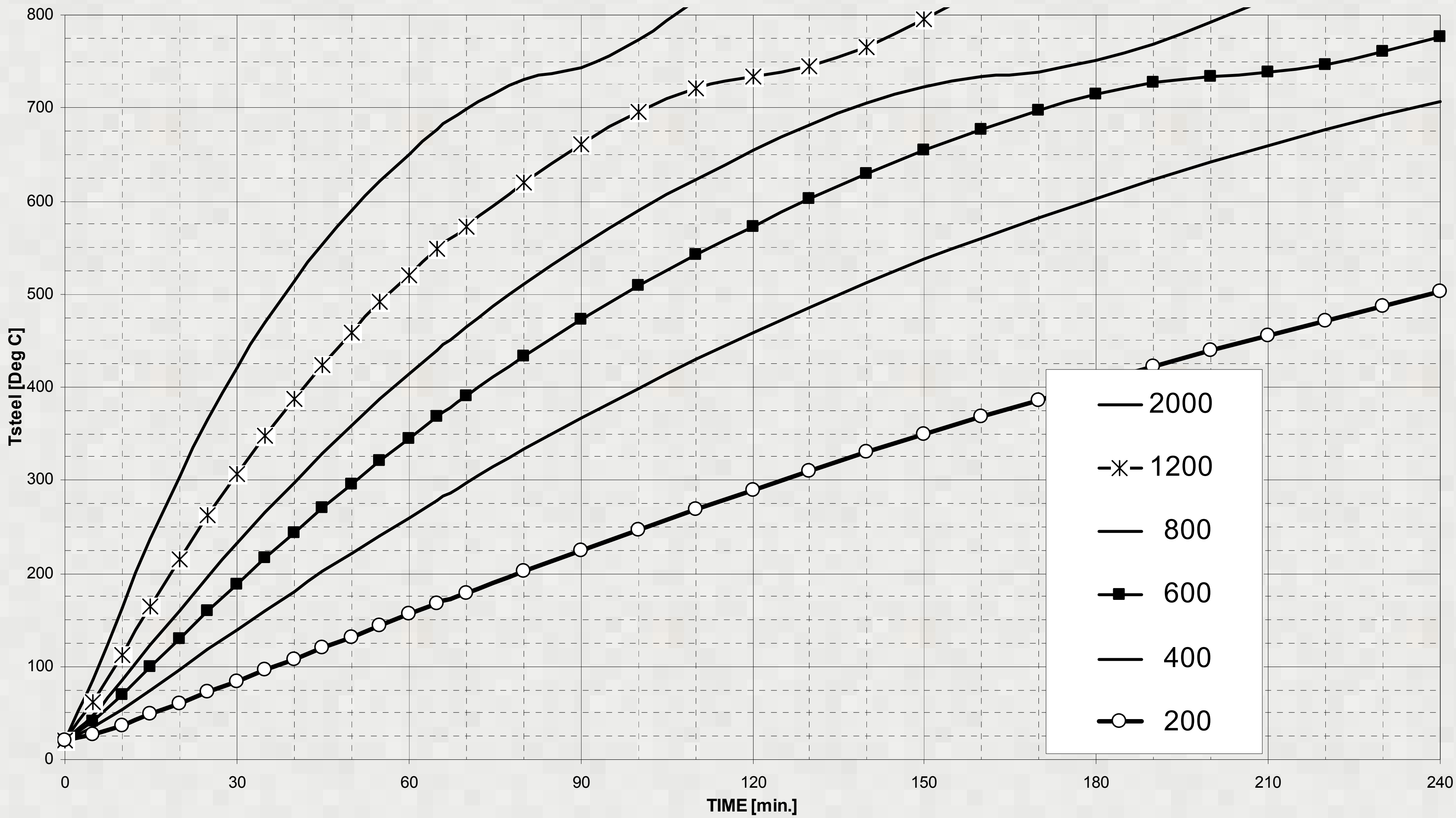
with:

- $\gamma_{m,fi}$ used instead of γ_{v}
- $k_{u,\theta}$ and $k_{c,\theta}$ defining the decrease of material strength
- θ_{u} in the stud = 0.80 $\theta_{upper\ flange}$
- θ_{c} of the concrete = 0.40 $\theta_{upper\ flange}$

(6) The increase of temperature $\Delta\theta_{a,t}$ of various parts of an **insulated steel beam** during the time interval Δt may be obtained from:

$$\Delta\theta_{a,t} = \left[\left(\frac{\lambda_p / d_p}{c_a \rho_a}\right)\left(\frac{A_{p,i}}{V_i}\right)\left(\frac{1}{1+w/3}\right)\left(\theta_t - \theta_{a,t}\right)\Delta t\right] - \left[\left(e^{w/10} - 1\right)\Delta\theta_t\right] \qquad (4.8)$$

with $$w = \left(\frac{c_p \ \rho_p}{c_a \ \rho_a}\right) d_p \left(\frac{A_{p,i}}{V_i}\right)$$



$\theta_{a,t}$ as a function of time for different values of $\lambda_p A_{pi} / d_p V_i$ ($w_i = 0$)

3. Verification of simply supported beams

3.1. Determination of the temperatures

3.2. Structural behaviour – critical temperature model

3.3. Structural behaviour – bending moment resistance model

3.4. Verification of stud connectors

3.5.Vertical shear resistance

See § 4.3.4.1.3

Neglect the contribution of the concrete slab, except if test evidence.

3. Verification of simply supported beams

3.1. Determination of the temperatures

3.2. Structural behaviour – critical temperature model

3.3. Structural behaviour – bending moment resistance model

3.4. Verification of stud connectors

3.5. Vertical shear resistance

3.6. Local resistance at supports

Temperature of the stiffener according to its own section factor A_r / V_r

$R = \min (R_{crushing} ; R_{buckling})$

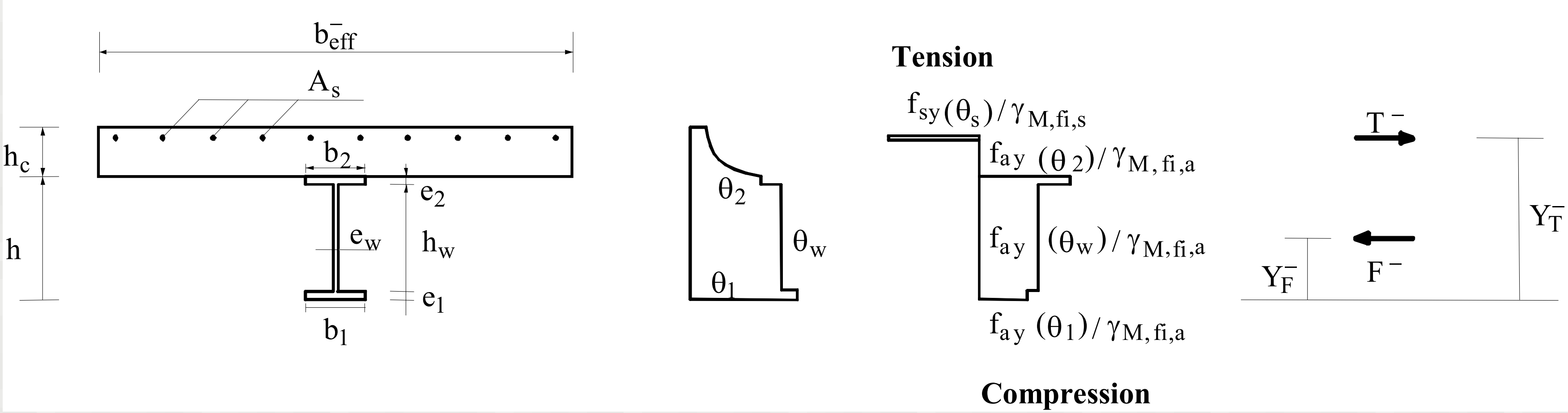
1. Introduction
2. Verification of the composite slab
3. Verification of simply supported beams
4. Verification of continuous beams

All provisions given for simply supported beams have to be met.

One additional consideration: Calculation of the hogging moment resistance: E.2

Hogging moment resistance at an intermediate support

Choose the effective width of the slab to have the slab completely cracked, but $b_{eff}(T) \leq b_{eff}(20°C)$.



If web or lower flange are Class 3, reduce it's width according to EN 1993-1-5.
If web or lower flange are Class 4, its resistance may be neglected.

Note: classification according to EN 1993-1-2

$$\varepsilon = 0,85\ [235 / f_y]^{0,5}$$

(6) The value of the compressive force F in the slab, at the critical cross section within the span, see (2) of E.1, may be such as :

$$F \leq N \times P_{fi,Rd} - T^{-}$$

where:

N is the number of shear connectors between the critical cross-section and the intermediate support (or the restraining support),

$P_{fi,Rd}$ is the shear resistance of a shear connector in case of fire, as mentioned in clause 4.3.4.2.5,

T^{-} is the total tensile force of the reinforcing bars at the intermediate support.

4.3.1. 3(P)

For composite beams in which the effective section is Class 1 or Class 2 (see EN 1993-1-1), and for composite slabs, the design bending resistance shall be determined by plastic theory.

4.3.1(6)

For continuous composite slabs and beams, the rules of EN 1992-1-2 and EN 1994-1-1 apply in order to guarantee the required rotation capacity.

4.3.4.1.2. (1)

The design bending resistance may be determined by plastic theory for any class of cross sections except for class 4.

4.3.4.1.2. (3)

For class 4 steel cross-sections, refer to 4.2.3.6 of EN 1993-1-2. (Note: resistance deemed to be ensured if the temperature is limited to 350°C)

E.2. (8)

When the steel web or the lower steel flange of the composite section is of class 3 in the fire situation, its width may be reduced to an effective value adapted from EN 1993-1-5, where f_y and E are respectively replaced by $f_{ay,\theta}$ and $E_{a,\theta}$,

E.2. (9)

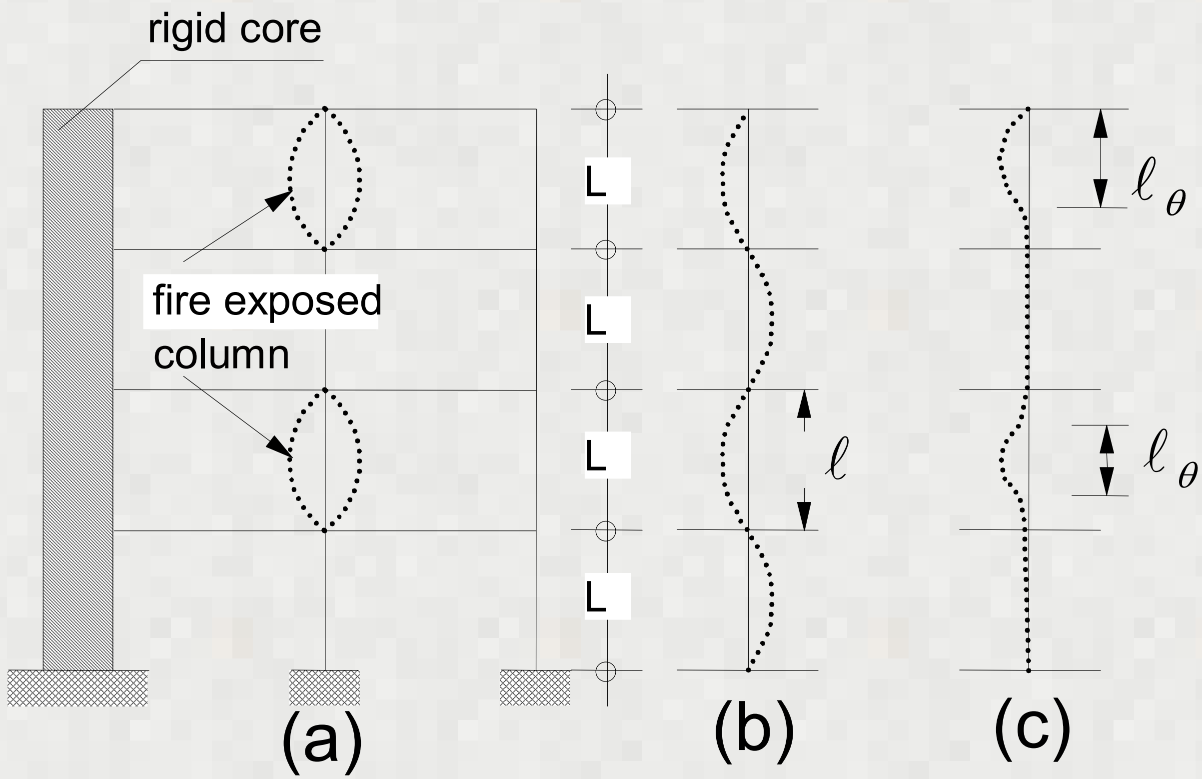
When the steel web or the bottom steel flange of the composite section is of class 4 in the fire situation, its resistance may be neglected.

1. Introduction
2. Verification of the composite slab
3. Verification of simply supported beams
4. Verification of continuous beams
5. Verification of composite columns made of partially encased steel sections

5.1.Tabulated data: 4.2, 4.2.3 & 4.2.3.3

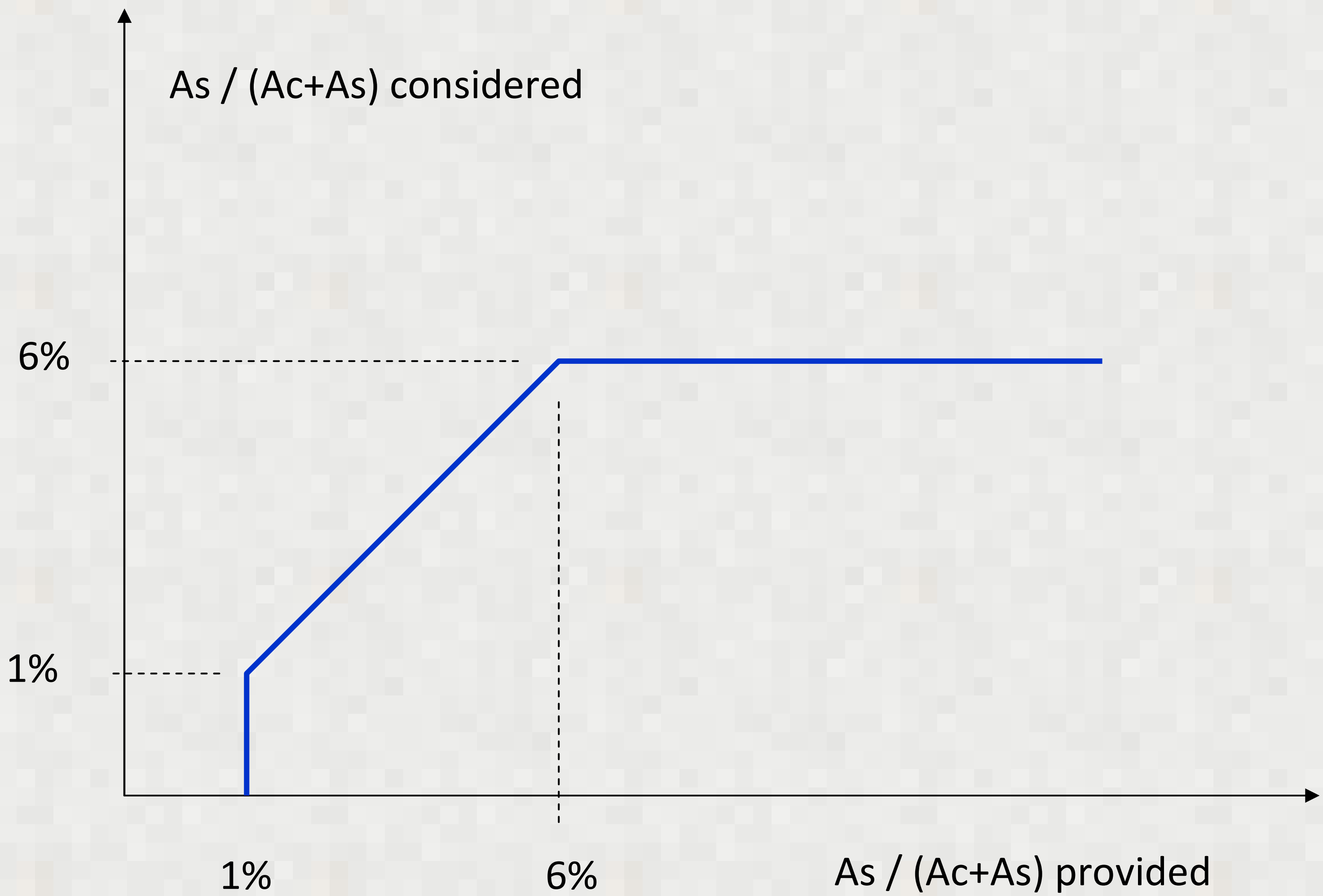
- Valid only for the standard fire exposure.
- Valid only for braced frames.
- Valid if $L \leq 30 \min(b,h)$.
- Main parameter: $\eta_{fi,t} = E_{fi,d,t} / R_d$
- R_d has to be based on twice the buckling length used in the fire design situation.
- $A_s / (A_c+A_s)$ higher than 6 % or lower than 1 %, should not be taken into account.

R_d has to be based on twice the buckling length used in the fire design situation

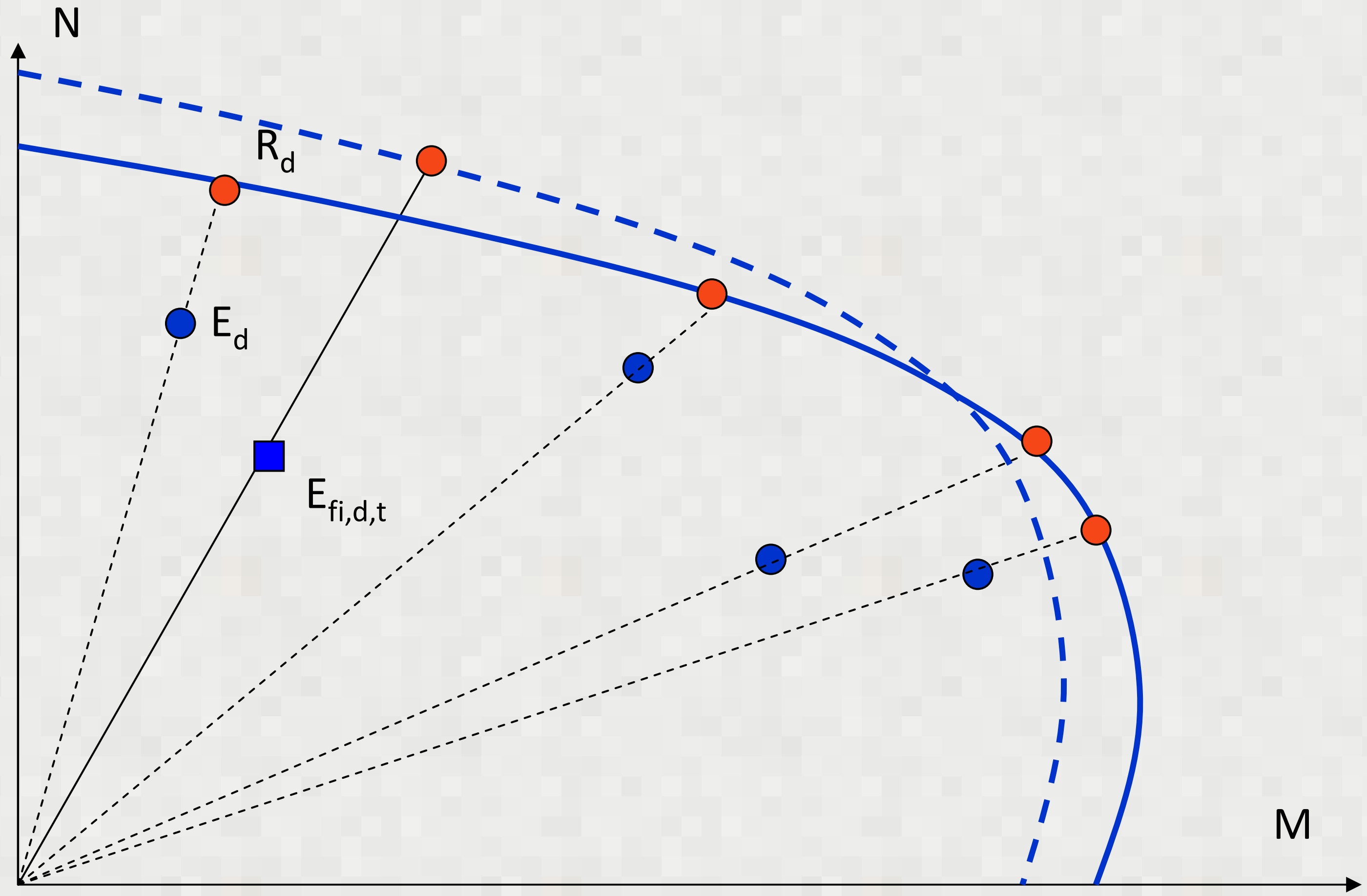


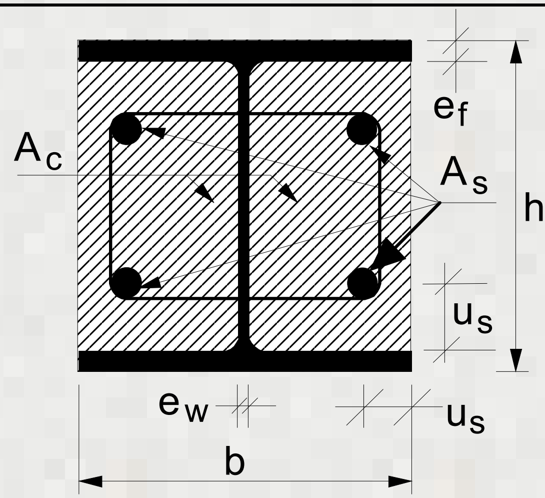
b) Deformation mode at room temperature
c) Deformation mode at elevated temperature

A_s / (A_c+A_s) higher than 6 % or lower than 1 %, should not be taken into account



Main parameter: $\eta_{fi,t} = E_{fi,d,t} / R_d$





		Standard Fire Resistance			
		R30	R60	R90	R120
	Minimum ratio of web to flange thickness e_w/e_f	*0,5*	*0,5*	*0,5*	*0,5*
1	Minimum cross-sectional dimensions for load level $\eta_{fi,t} \leq$ **0,28**				
1.1	minimum dimensions h and b [mm]	*160*	*200*	*300*	*400*
1.2	minimum axis distance of reinforcing bars u_s [mm]	-	*50*	*50*	*70*
1.3	minimum ratio of reinforcement $A_s/(A_c+A_s)$ in %	-	*4*	*3*	*4*
2	Minimum cross-sectional dimensions for load level $\eta_{fi,t} \leq$ **0,47**				
2.1	minimum dimensions h and b [mm]	*160*	*300*	*400*	-
2.2	minimum axis distance of reinforcing bars u_s [mm]	-	*50*	*70*	-
2.3	minimum ratio of reinforcement $A_s/(A_c+A_s)$ in %	-	*4*	*4*	-
3	Minimum cross-sectional dimensions for load level $\eta_{fi,t} \leq$ **0,66**				
3.1	minimum dimensions h and b [mm]	*160*	*400*	-	-
3.2	minimum axis distance of reinforcing bars u_s [mm]	*40*	*70*	-	-
3.3	minimum ratio of reinforcement $A_s/(A_c+A_s)$ in %	*1*	*4*	-	-

5.2.Simple calculation method: 4.3, 4.3.5, 4.3.5.2 & Annex G

- Valid only for the standard fire exposure (all sides)

- Valid only for braced frames

- The buckling length used in the fire design situation may be smaller than at room temperature.

- A_s / (A_c+A_s) ∈ [1%;6%]
- $l_\theta \leq 13.5$ b
- $l_\theta \leq 10.0$ b
  - pour R60 si b ∈ [230mm;300mm[ou si h/b > 3
  - pour R90 et R120 si h/b > 3
- h ∈ [230mm;1100mm]
- b ∈ [230mm;500mm]
- b et h ≥ 300mm pour R90 et R120
- R ≤ 120 min.

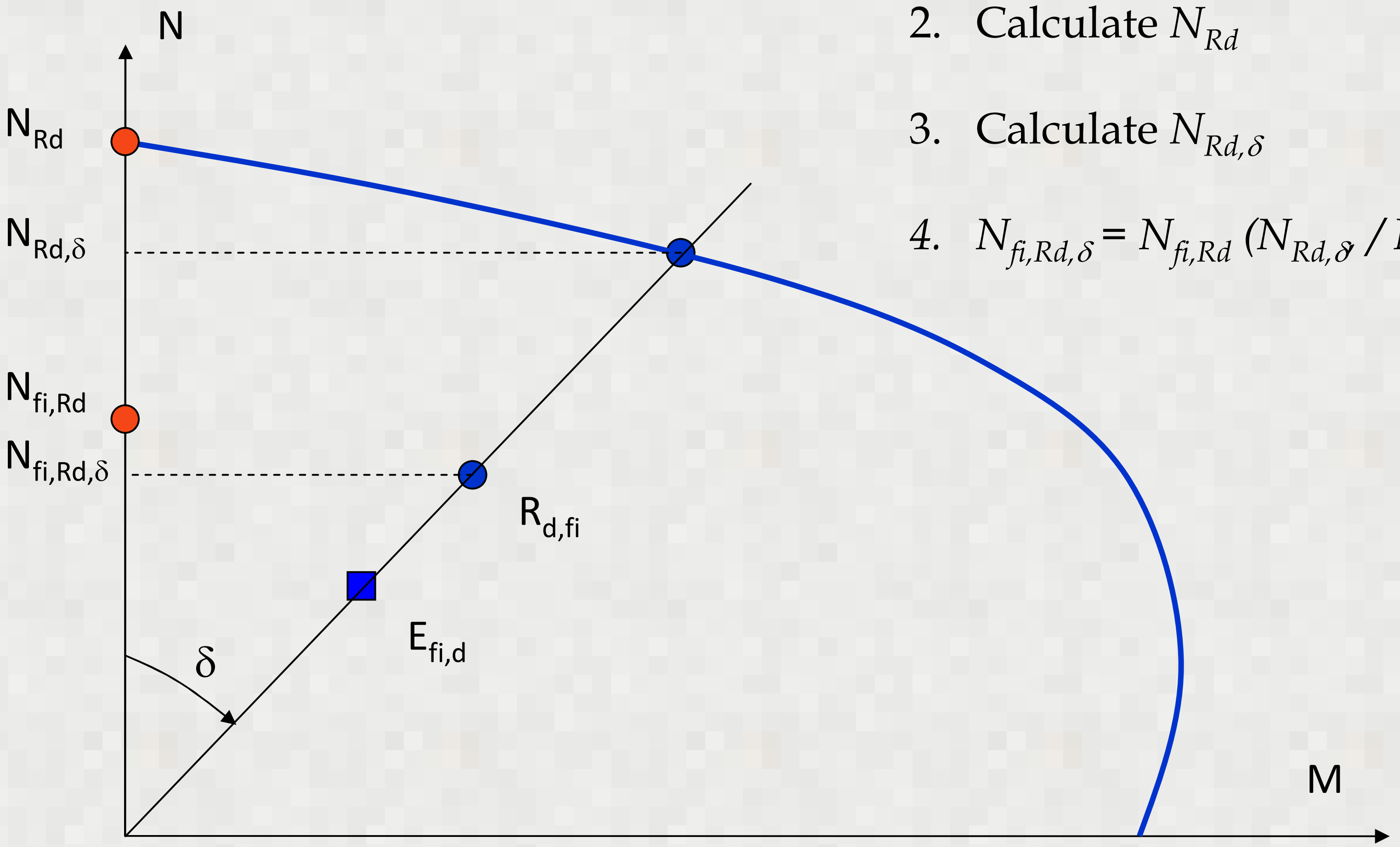
Principle of the simple design method:

1. Calculate $N_{fi,Rd} = \min(N_{fi,Rd,z};N_{fi,Rd,y})$
    1. $N_{fi,Rd,z} = \chi_z N_{fi,pl,Rd}$ (see annex G)
    2. $N_{fi,Rd,y} = \chi_y N_{fi,pl,Rd}$

2. Calculate N_{Rd}

3. Calculate $N_{Rd,\delta}$

4. $N_{fi,Rd,\delta} = N_{fi,Rd} (N_{Rd,\delta} / N_{Rd})$

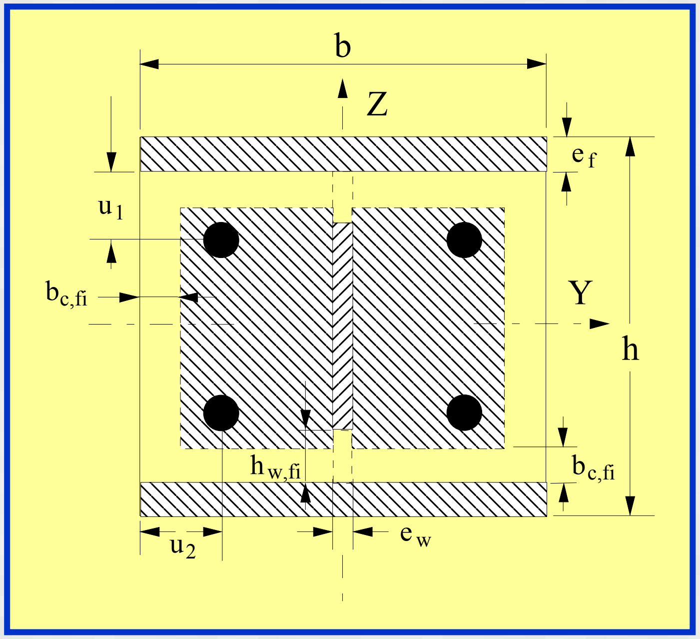


Principle of the simple design method:

1. Calculate $N_{fi,Rd} = \min(N_{fi,Rd,z};N_{fi,Rd,y})$
   1. $N_{fiRd,z} = \chi_z N_{fi,pl,Rd}$ (see annex G)
   2. $N_{fiRd,y} = \chi_y N_{fi,pl,Rd}$
2. Calculate N_{Rd}
3. Calculate $N_{Rd,\delta}$
4. $N_{fi,Rd,\delta} = N_{fi,Rd} \, (N_{Rd,\delta} \, / \, N_{Rd})$

How to calculate $N_{fi,Rd}$?

Annex G: reduced cross-section and reduced properties



Flanges

(1) The average flange temperature may be determined from:

$\theta_{f,t} = \theta_{o,t} + k_t \, (A_m/V)$,

Standard Fire Resistance	$\theta_{o,t}$ [°C]	k_t [m°C]
R30	550	9,65
R60	680	9,55
R90	805	6,15
R120	900	4,65

Web

(1) The part of the web with the height $h_{w,fi}$ and starting at the inner edge of the flange should be neglected (see figure G.1). This part is determined from:

$h_{w,fi}$ = 0,5(h-2e_f)$\left(1 - \sqrt{1 - 0,16\left(H_t / h\right)}\right)$ where H_t is given in table G.2.

Table G.2

Standard Fire Resistance	H_t [mm]
R 30	350
R 60	770
R 90	1100
R 120	1250

(2) The maximum stress level is obtained from:

$f_{amax,w,t} = f_{ay,w,20°C} \sqrt{1 - \left(0,16 H_t / h\right)}$

Concrete

(1) An exterior layer of concrete with a thickness $b_{c,fi}$ should be neglected in the calculation (see figure G.1). The thickness $b_{c,fi}$ is given in table G.3, with A_m/V, the section factor in m^{-1} of the entire composite cross-section.

Table G.3

Standard Fire Resistance	$b_{c,fi}$ [mm]
R30	4,0
R60	15,0
R90	$0{,}5\,(A_m/V) + 22{,}5$
R120	$2{,}0\,(A_m/V) + 24{,}0$

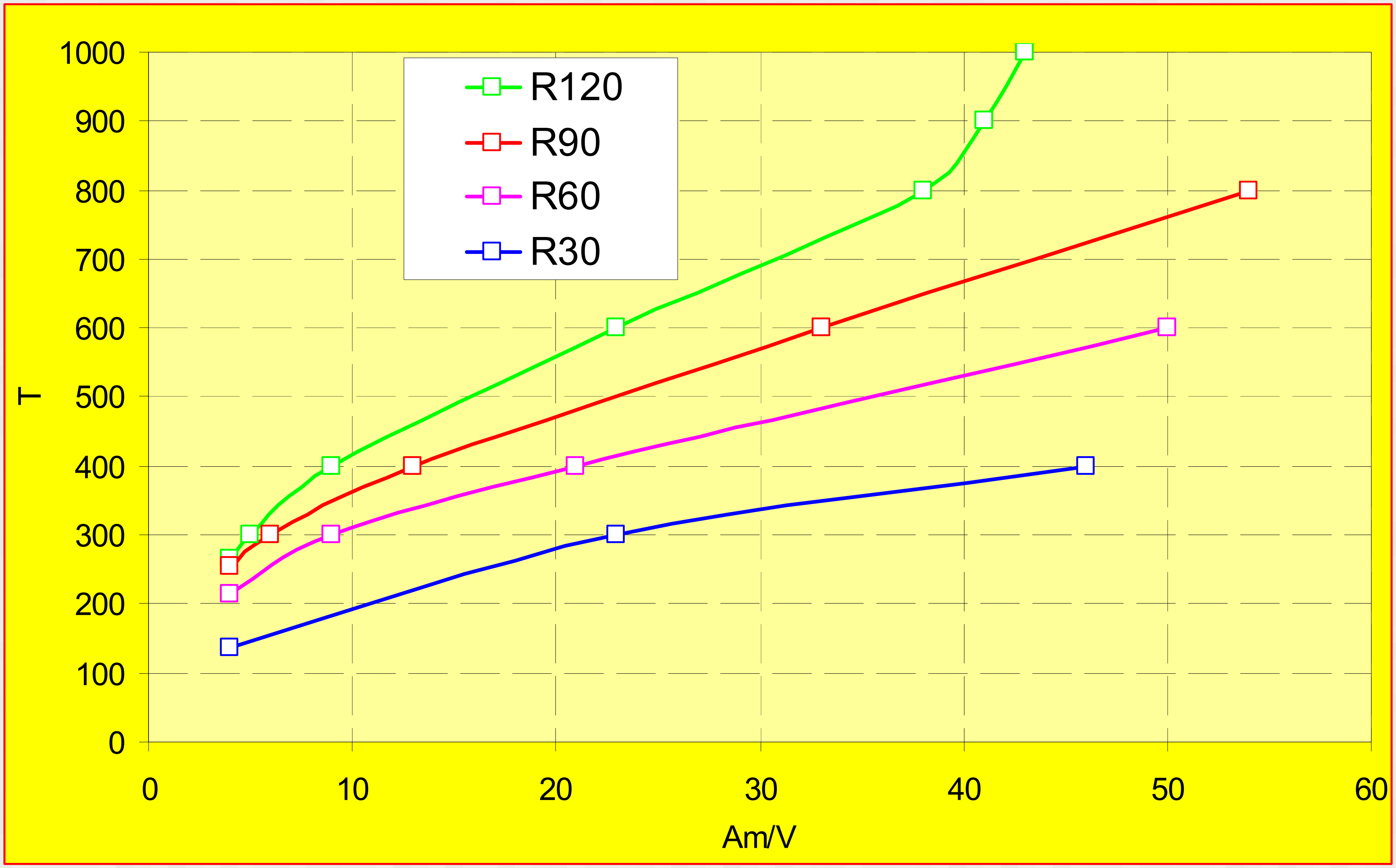
(2) The average temperature in concrete $\theta_{c,t}$ is given in table G.4 in function of the section factor A_m/V of the entire composite cross-section and for the standard fire resistance classes.

Table G.4

R30		R60		R90		R120	
A_m/V $[m^{-1}]$	$\theta_{c,t}$ [°C]	A_m/V $[m^{-1}]$	$\theta_{c,t}$ [°C]	A_m/V $[m^{-1}]$	$\theta_{c,t}$ [°C]	A_m/V $[m^{-1}]$	$\theta_{c,t}$ [°C]
4	136	4	214	4	256	4	265
23	300	9	300	6	300	5	300
46	400	21	400	13	400	9	400
-	-	50	600	33	600	23	600
-	-	-	-	54	800	38	800
-	-	-	-	-	-	41	900
-	-	-	-	-	-	43	1000

(3) On behalf of the temperature $\theta = \theta_{c,t}$ the secant modulus of concrete is obtained from:

$E_{c,sec,\theta} = f_{c,\theta} / \varepsilon_{cu,\theta} = f_{c,20°C}\ k_{c,\theta} / \varepsilon_{cu,\theta}$ with $k_{c,\theta}$ and $\varepsilon_{cu,\theta}$ following table 3.3 of 3.2.2



R120
R90
R60
R30
T
1000
900
800
700
600
500
400
300
200
100
0
0
10
20
30
40
50
60
Am/V

Table G4

Reinforcing bars

Table G.5: Reduction factor $k_{y,t}$ for the yield point $f_{sy,20°C}$ of the reinforcing bars

Standard Fire Resistance \ u[mm]	40	45	50	55	60
R30	1	1	1	1	1
R60	0,789	0,883	0,976	1	1
R90	0,314	0,434	0,572	0,696	0,822
R120	0,170	0,223	0,288	0,367	0,436

Table G.6: Reduction factor $k_{E,t}$ for the modulus of elasticity $E_{s,20°C}$ of the reinforcing bars

Standard Fire Resistance \ u[mm]	40	45	50	55	60
R30	0,830	0,865	0,888	0,914	0,935
R60	0,604	0,647	0,689	0,729	0,763
R90	0,193	0,283	0,406	0,522	0,619
R120	0,110	0,128	0,173	0,233	0,285

(2) The geometrical average u of the axis distances u_1 and u_2 is obtained from:

$$u = \sqrt{u_1 \cdot u_2}$$

$$N_{fi,pl.Rd} = N_{fi,pl.Rd,f} + N_{fi,pl.Rd,w} + N_{fi,pl.Rd,c} + N_{fi,pl.Rd,s}$$

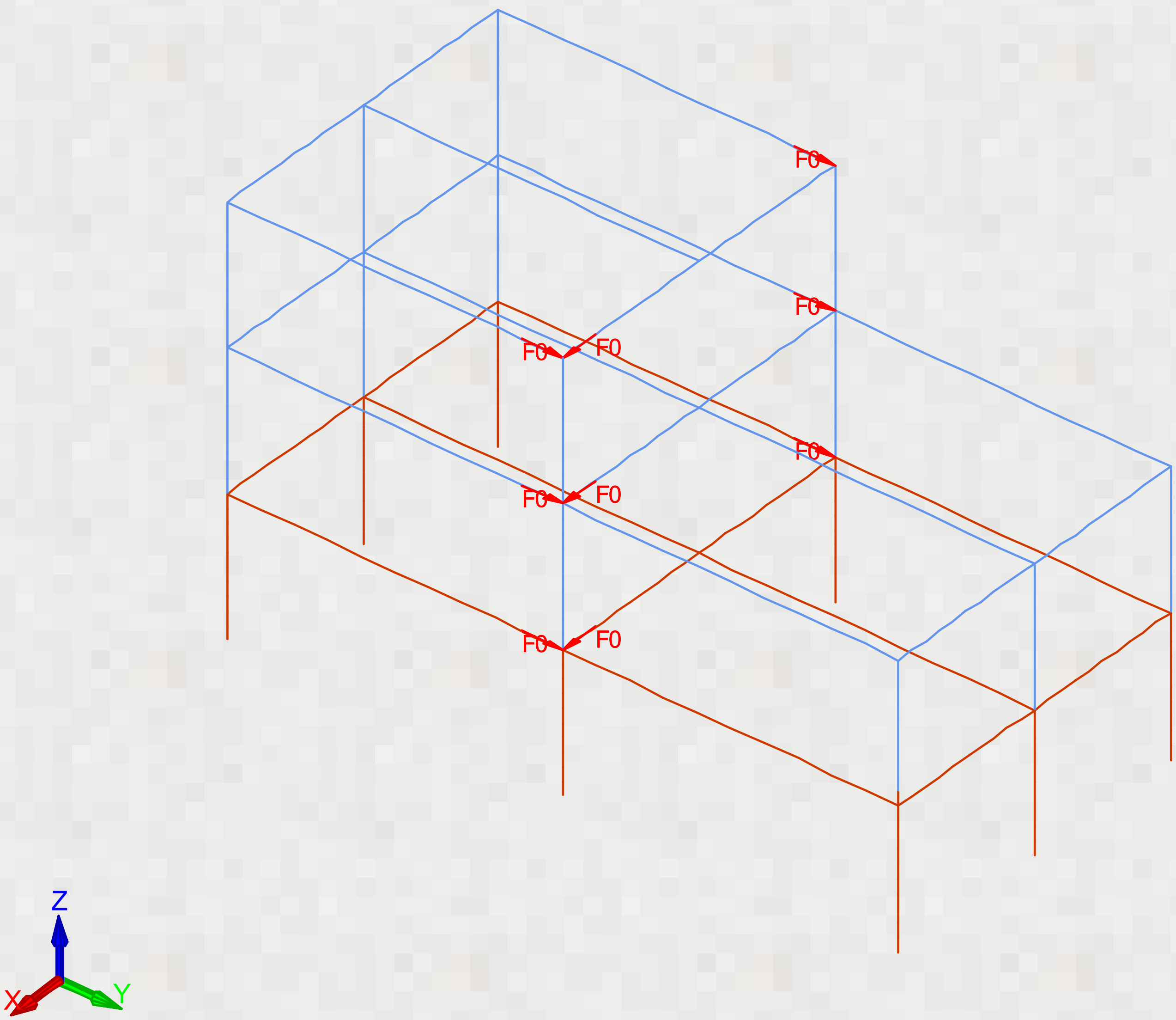
$$(EI)_{fi,eff,z} = \varphi_{f,\theta}\ (EI)_{fi,f,z} + \varphi_{w,\theta}\ (EI)_{fi,w,z} + \varphi_{c,\theta}\ (EI)_{fi,c,z} + \varphi_{s,\theta}\ (EI)_{fi,s,z}$$

where $\varphi_{i,\theta}$ is a reduction coefficient depending on the effect of thermal stresses. The values of $\varphi_{i,\theta}$ are given in table G.7.

Table G.7

Standard Fire Resistance	$\varphi_{f,\theta}$	$\varphi_{w,\theta}$	$\varphi_{c,\theta}$	$\varphi_{s,\theta}$
R30	1,0	1,0	0,8	1,0
R60	0,9	1,0	0,8	0,9
R90	0,8	1,0	0,8	0,8
R120	1,0	1,0	0,8	1,0

Buckling curve "***c***" of EN 1993-1-1



Diamond 2004 for SAFIR

FILE: bat
NODES: 1077
BEAMS: 520
TRUSSES: 0
SHELLS: 0
SOILS: 0

BEAMS PLOT
IMPOSED DOF PLOT

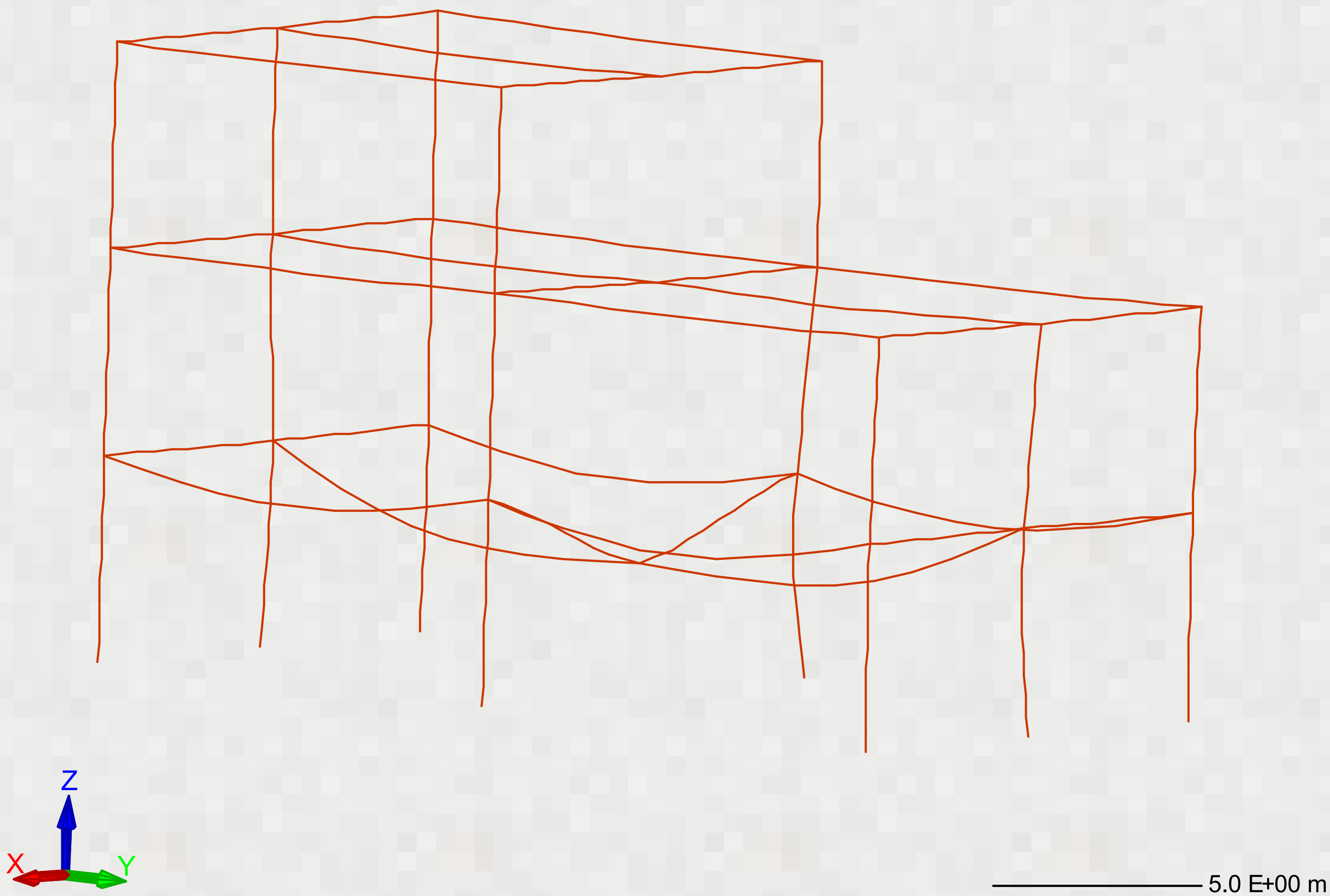
hea260fort.tem
hea260fortC.tem

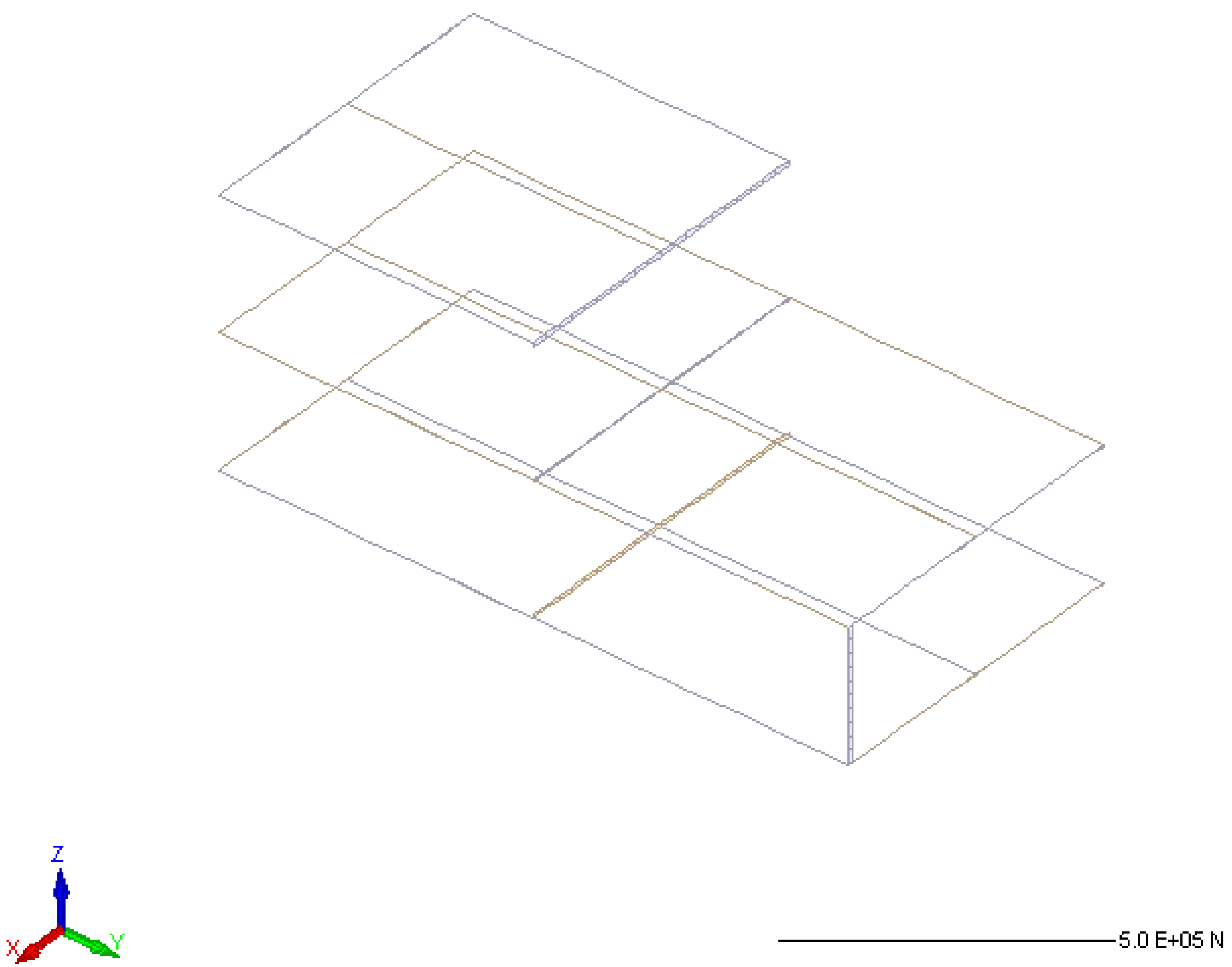
Diamond 2004 for SAFIR

FILE: bat
NODES: 1077
BEAMS: 520
TRUSSES: 0
SHELLS: 0
SOILS: 0

DISPLACEMENT PLOT (x 1)

TIME: 3129.25 sec





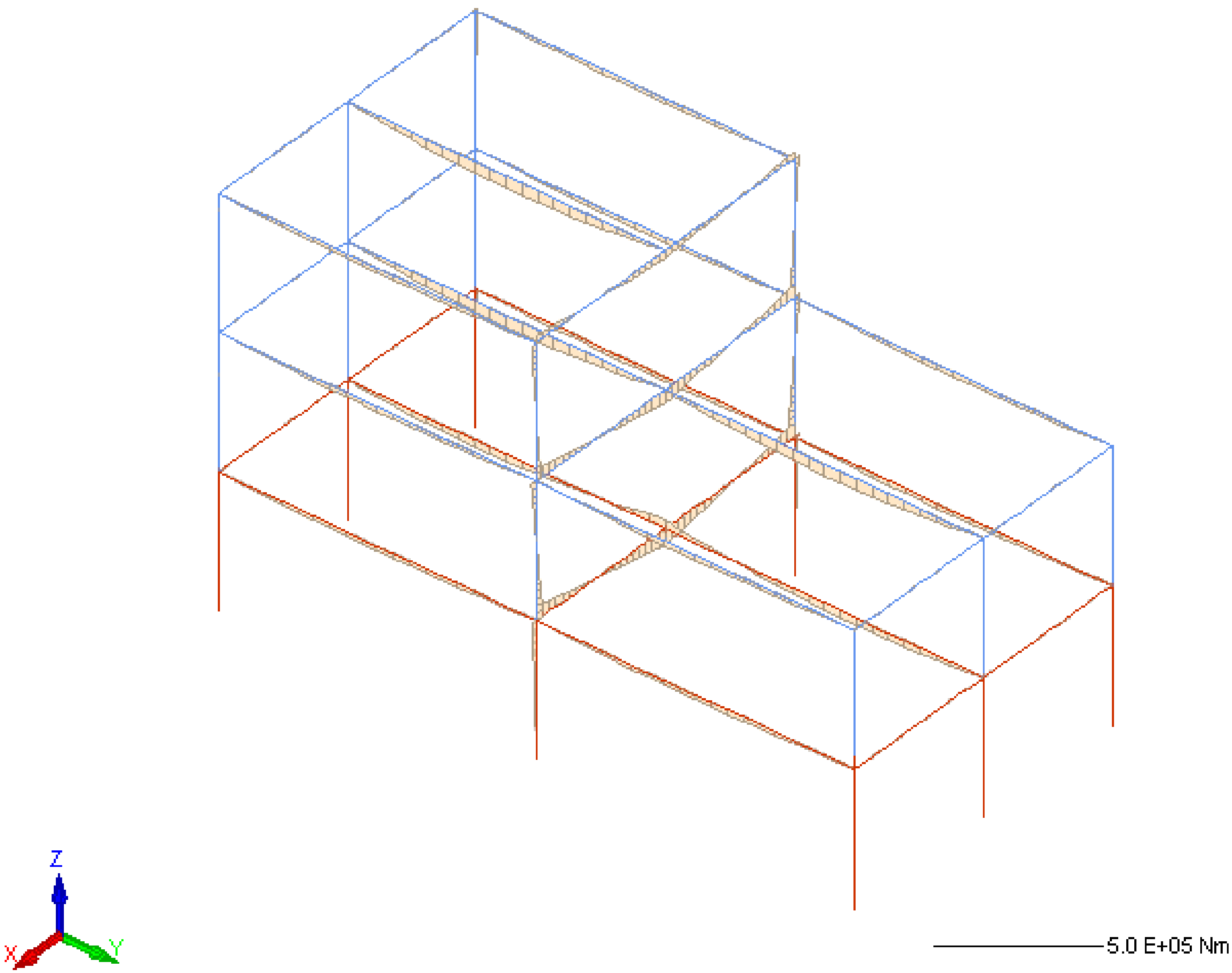
Diamond 2004 for SAFIR

FILE: bat
NODES: 1077
BEAMS: 520
TRUSSES: 0
SHELLS: 0
SOILS: 0

AXIAL FORCE PLOT

TIME: 4 sec

N < 0
N > 0

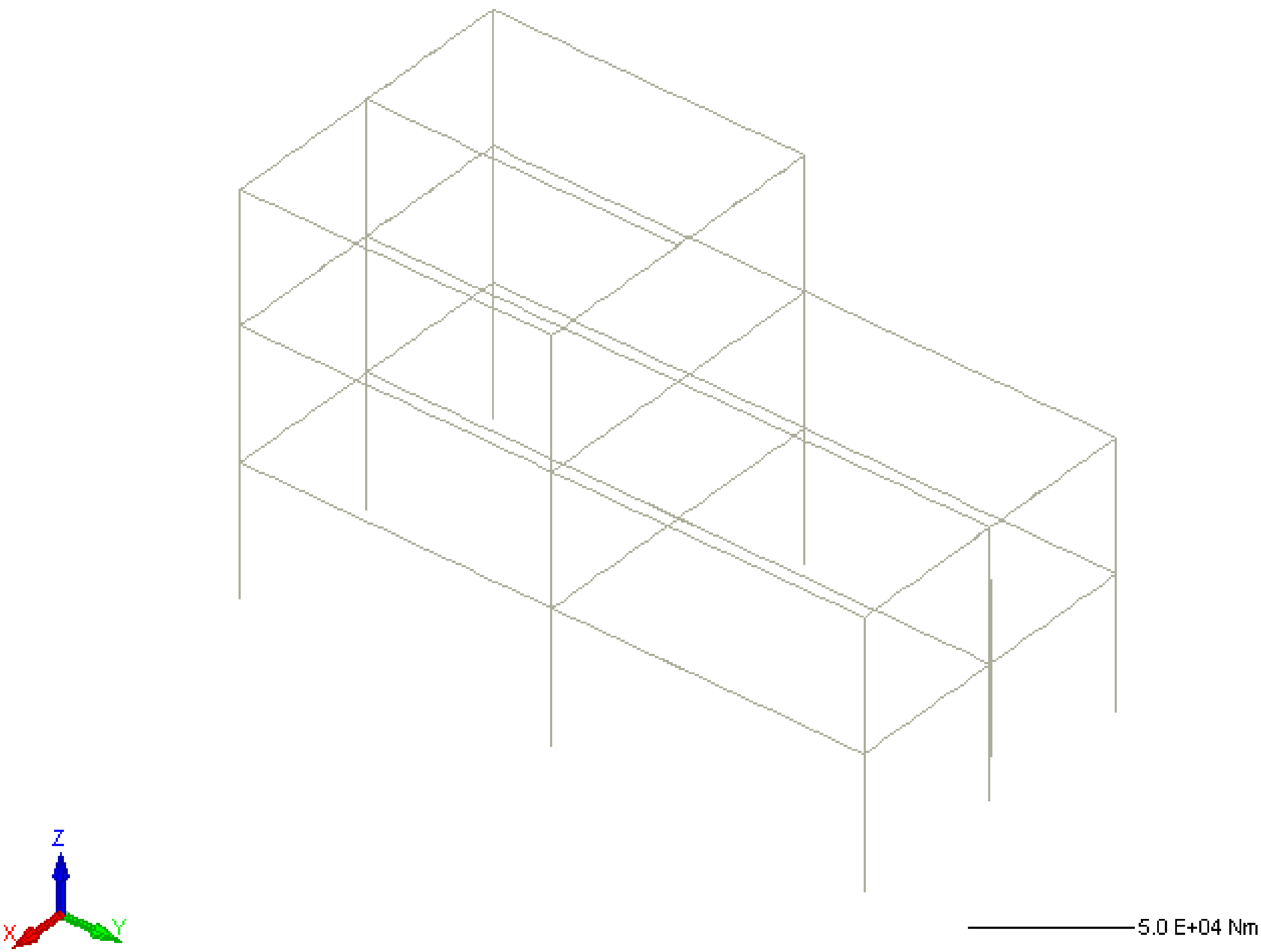


Diamond 2004 for SAFIR

FILE: bat
NODES: 1077
BEAMS: 520
TRUSSES: 0
SHELLS: 0
SOILS: 0

BEAMS PLOT
My BENDING MOMENT PLOT

TIME: 4 sec



Diamond 2004 for SAFIR

FILE: bat
NODES: 1077
BEAMS: 520
TRUSSES: 0
SHELLS: 0
SOILS: 0

Mz BENDING MOMENT PLOT

TIME: 4 sec