Fire resistance assessment of Composite structures

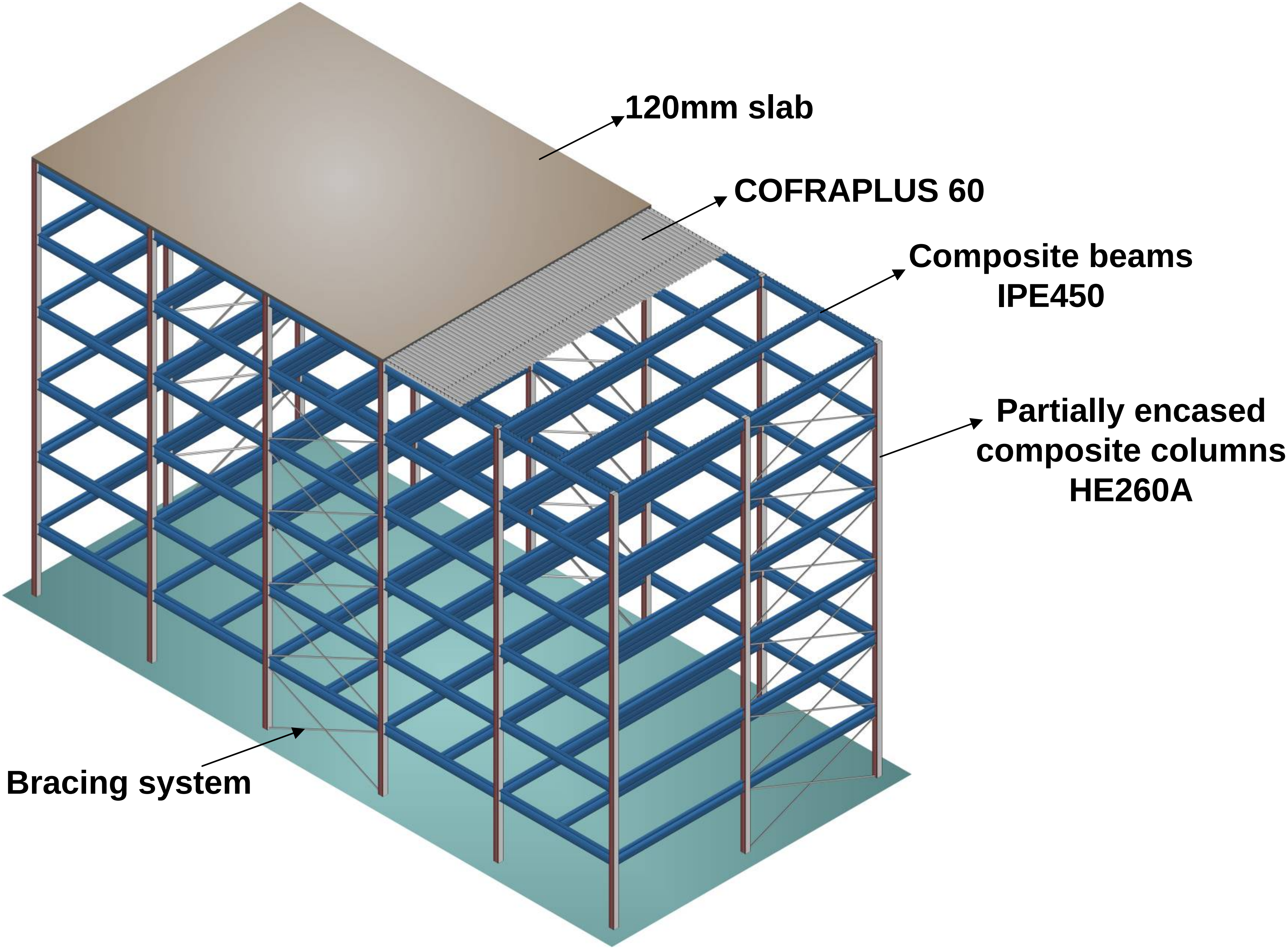
Basic design methods
Worked examples

CAJOT Louis-Guy

Chairman of CEN/TC250/SC4/EG - Fire Part

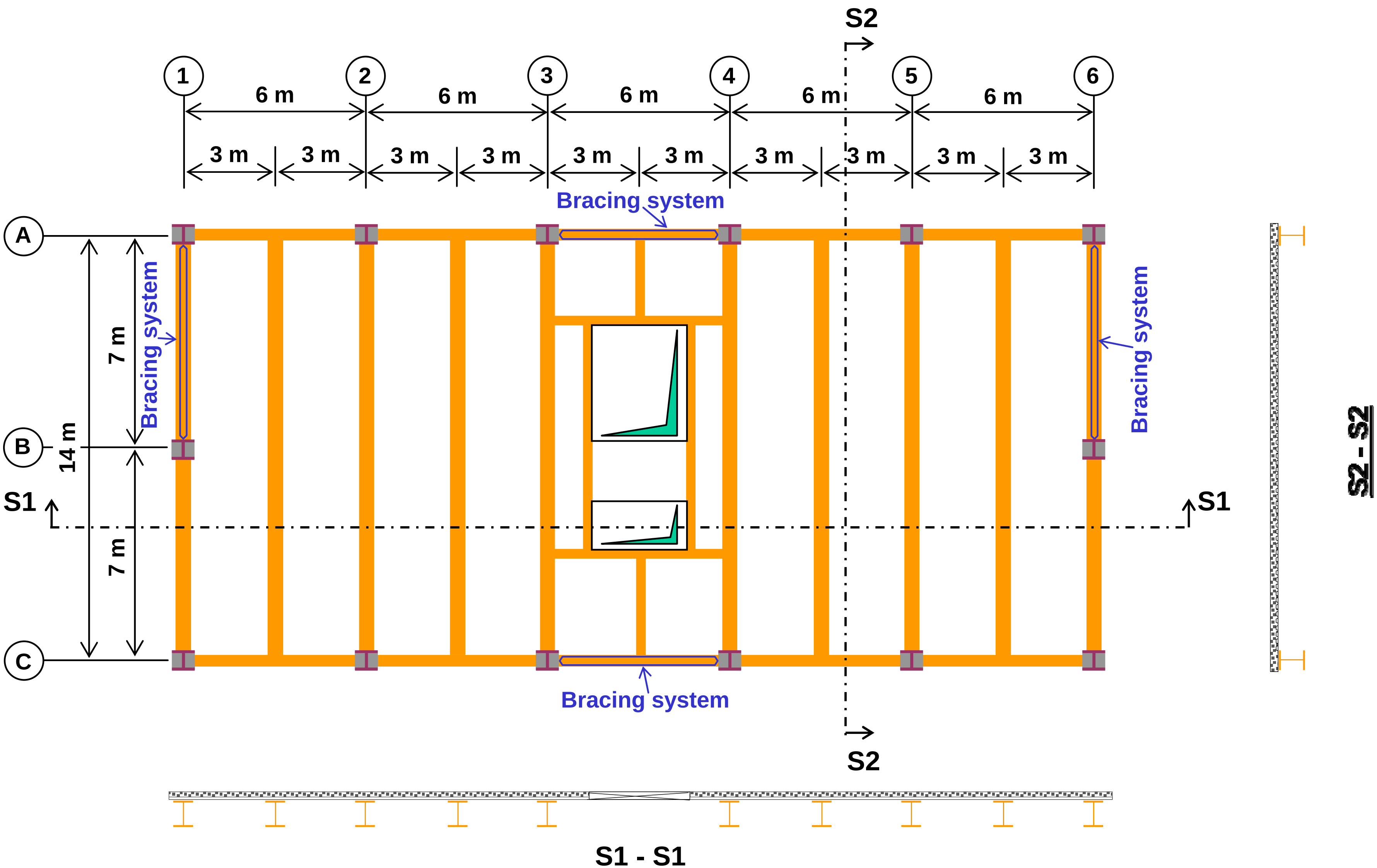
ArcelorMittal

The Building (R+5)



Structure grid of the composite building

Composite slab, composite beams, composite columns



Structure grid of the composite building

Actions (for all floor levels)

- ⇨ Self weight G1
  - ✓Composite slab unit weight : 2.12 kN/m²
  - ✓Steel structural members
- ⇨ Permanent load G2
  - ✓Finishing, embedded services, partitions : 1.50 kN/m²
- ⇨ Façade G3 : 2 kN/m
- ⇨ Characteristic values of variable loads and ψ factors

Type	q_k	ψ_1	ψ_2
Live load on floors	4.0 kN/m²	0.7	0.6
Snow on roof	1.7 kN/m²	0.2	0.0

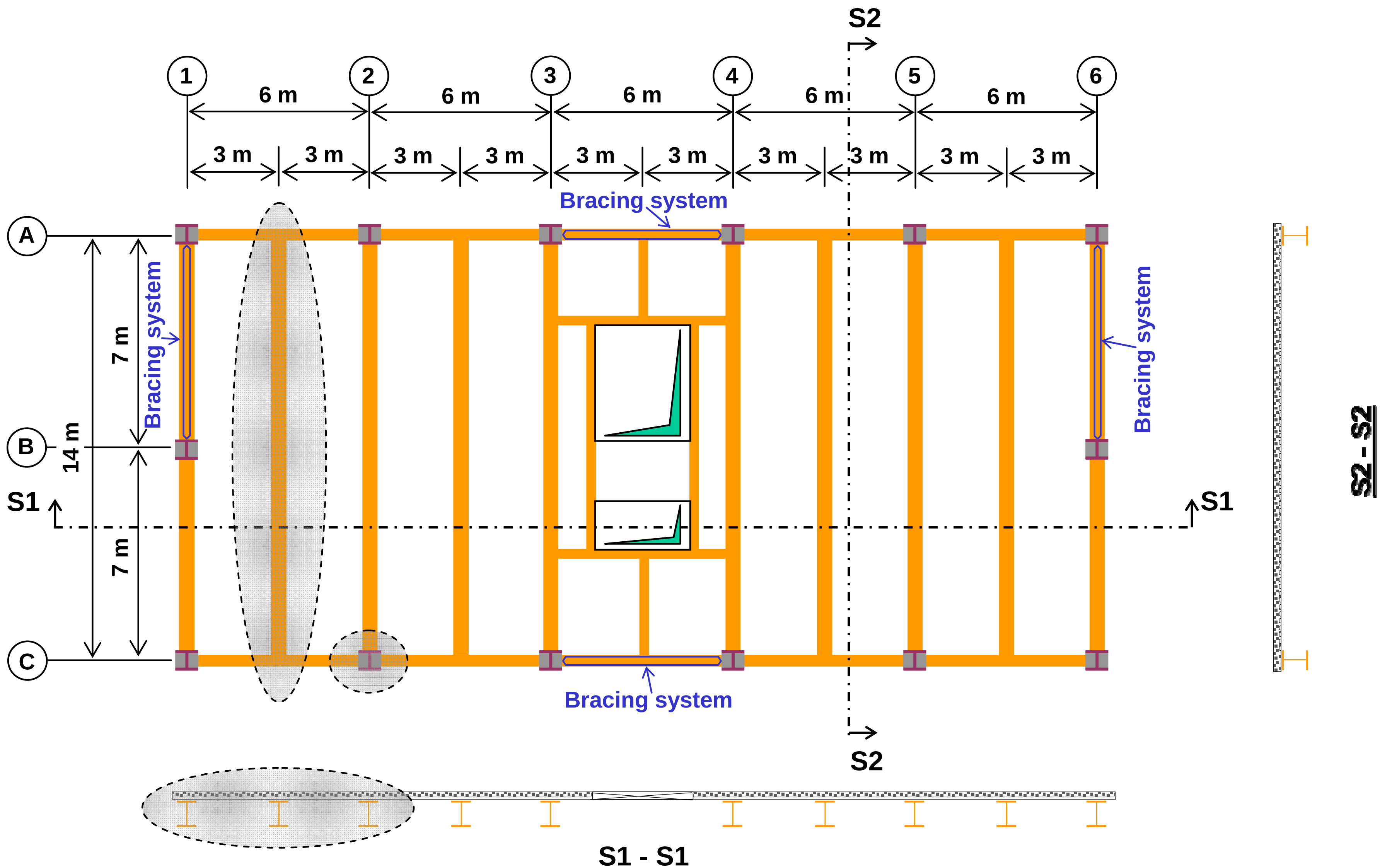
Structure grid of the composite building

Structural members

- ⇨ Composite slab
  - ✓ Total thickness: 12 cm
  - ✓ Steel deck: COFRAPLUS60
  - ✓ Thickness of steel deck: 0.75 mm
  - ✓ Continuous slab over 2 spans
- ⇨ Secondary beams
  - ✓ IPE450 linked with headed studs to the composite slab; fire protected steel sections
  - ✓ Alternative : Partially encased composite beams IPE450; fire protection obtained through partial encasement
- ⇨ Columns for ground level
  - ✓ Facade columns: Partially encased HEA260
  - ✓ Alternative: Fully encased HEB160

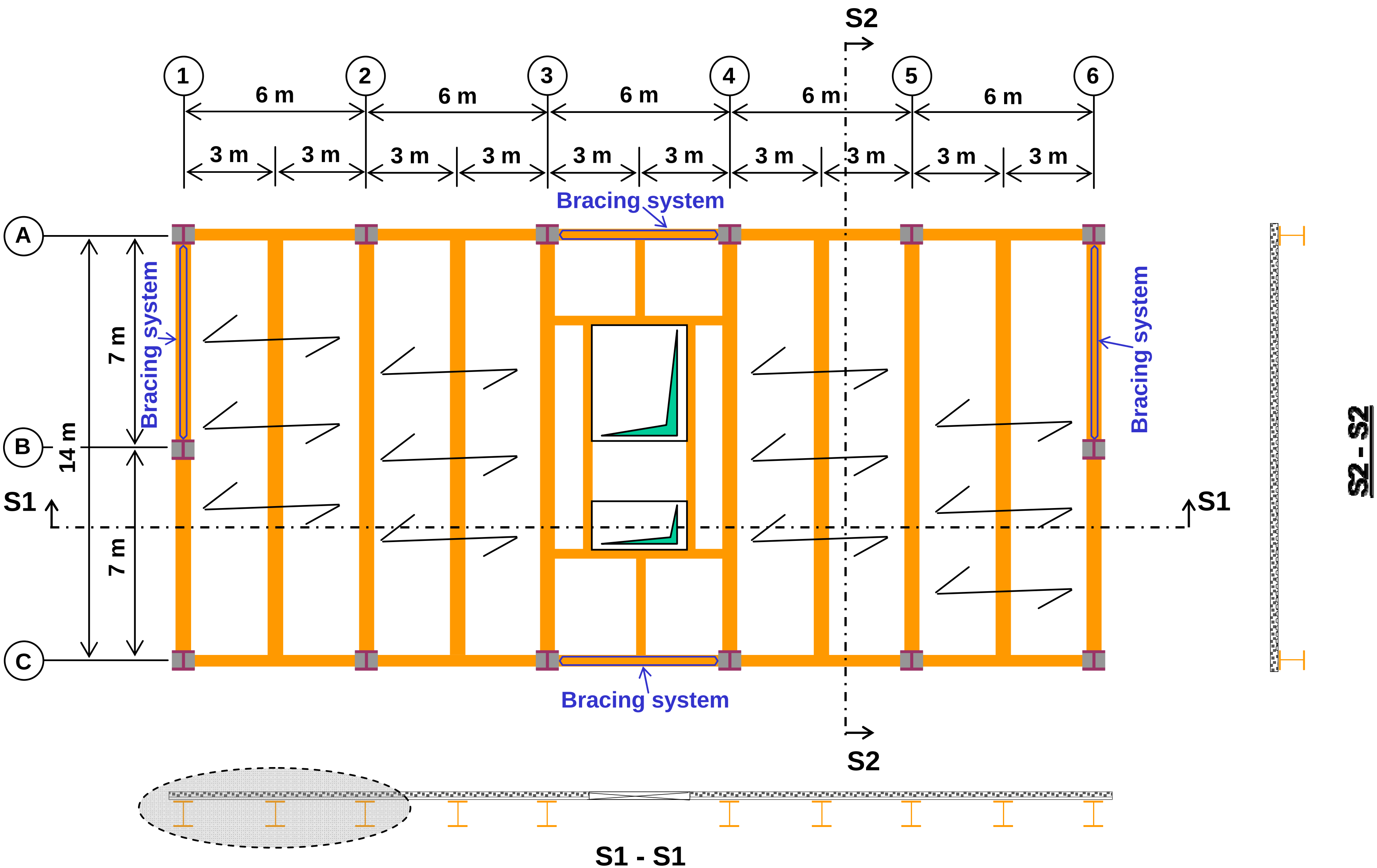
Structure grid of the composite building

Composite slab, composite beams, composite columns



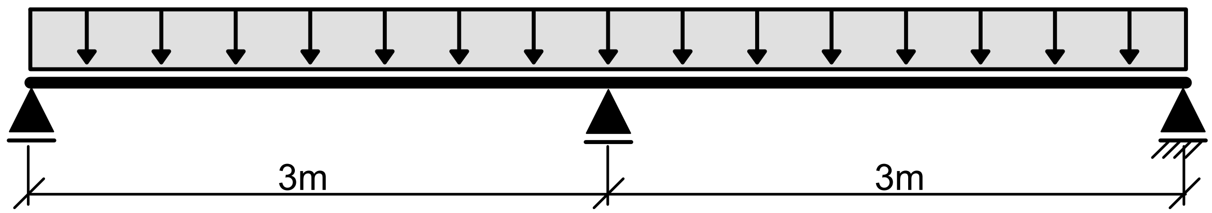
Structure grid of the composite building

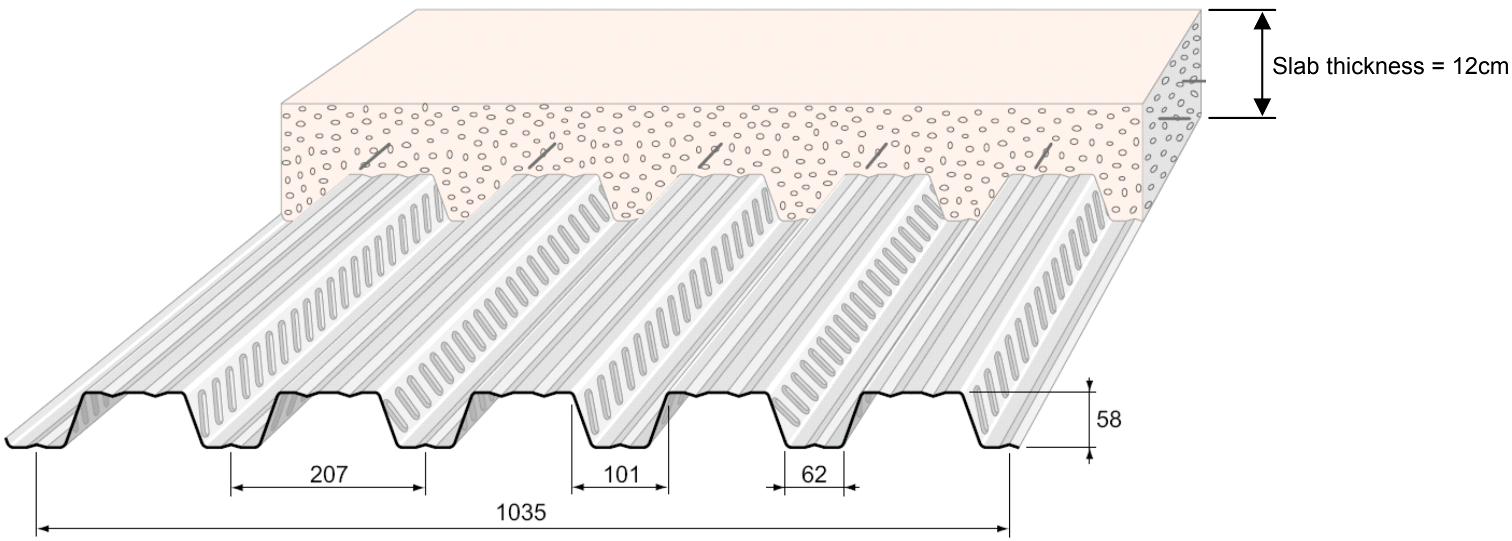
Composite slab continuous on 3 supports



Verification of the composite slab

Summary of data





Required fire resistance : R60

Verification of the composite slab

Material characteristics

Steel decking : $f_y = 350$ N/mm²

Concrete : C25/30 ; $f_c = 25$ N/mm²

Rebars : $f_y = 500$ N/mm²

Mesh ST25 ; A = 2,57 cm²/m

Ribs : 1 ϕ8 /rib

Loads

Permanent loads:

Steel decking : $g_{t,k} = 0{,}085$ kN/m²

Concrete : $g_{b,k} = 2{,}03$ kN/m²

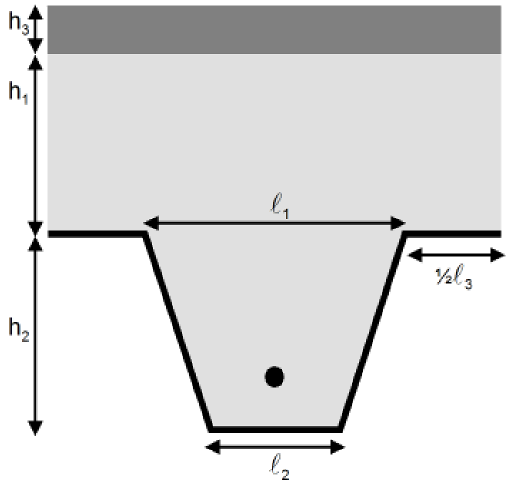
Permanent load : $g_{c,k} = 1{,}5$ kN/m²

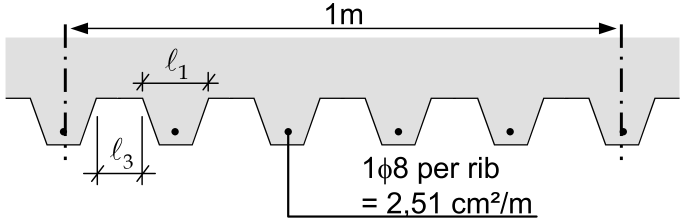
Variable load:

Variable load : $q_k = 4{,}0$ kN/m²

Verification of the composite slab

Geometric parameters





h_1 = 62 mm h_2 = 58 mm h_3 = 0

ℓ_1 = 101 mm ℓ_2 = 62 mm ℓ_3 = 106 mm

Application field

Trapezoidal steel decking profiles [mm]	Existing geometric parameters [mm]	Condition fulfilled ?
$80 \leq \ell_1 \leq 155$	$\ell_1 = 101$	OK
$32 \leq \ell_2 \leq 132$	$\ell_2 = 62$	OK
$40 \leq \ell_3 \leq 115$	$\ell_3 = 106$	OK
$50 \leq h_1 \leq 125$	$h_1 = 62$	OK
$50 \leq h_2 \leq 100$	$h_2 = 58$	OK

Verification of the composite slab

Fire resistance according to thermal insulation

$$t_i = a_0 + a_1 \cdot h_1' + a_2 \cdot \Phi + a_3 \cdot \frac{A}{L_r} + a_4 \cdot \frac{1}{\ell_3} + a_5 \cdot \frac{A}{L_r} \cdot \frac{1}{\ell_3}$$

$$t_i = (-28{,}8) + 1{,}55 \cdot h_1' + (-12{,}6) \cdot \Phi + 0{,}33 \cdot \frac{A}{L_r} + (-735) \cdot \frac{1}{\ell_3} + 48 \cdot \frac{A}{L_r} \cdot \frac{1}{\ell_3}$$

For Normal Weight Concrete

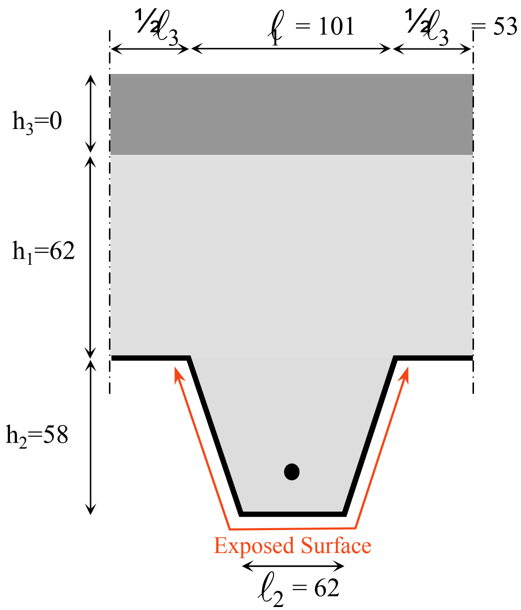
a_0 [min]	a_1 [min/mm]	a_2 [min]	a_3 [min/mm]	a_4 mm·min	a_5 [min]
-28,8	1,55	-12,6	0,33	-735	48

Verification of the composite slab

Fire resistance according to thermal insulation

$$t_i = a_0 + a_1 \cdot h_1' + a_2 \cdot \Phi + a_3 \cdot \frac{A}{L_r} + a_4 \cdot \frac{1}{\ell_3} + a_5 \cdot \frac{A}{L_r} \cdot \frac{1}{\ell_3}$$

$$t_i = (-28{,}8) + 1{,}55 \cdot 62 + (-12{,}6) \cdot \Phi + 0{,}33 \cdot \frac{A}{L_r} + (-735) \cdot \frac{1}{106} + 48 \cdot \frac{A}{L_r} \cdot \frac{1}{106}$$



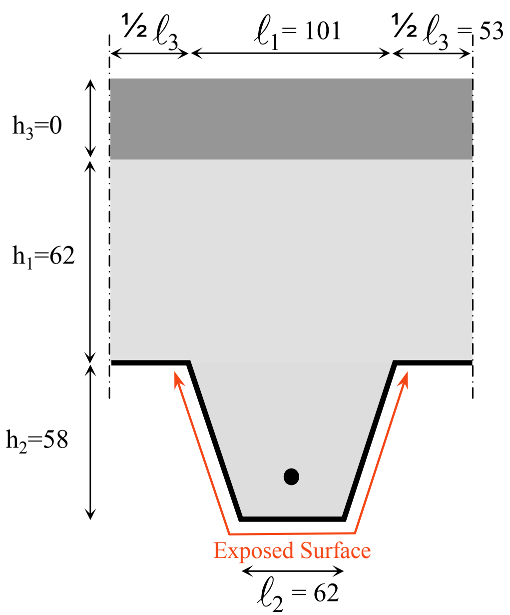
$$h_1' = h_1 + h_3 = 62\,\text{mm}$$ (h_3 = thickness of the screed)

Verification of the composite slab

Fire resistance according to thermal insulation

$$t_i = a_0 + a_1 \cdot h_1' + a_2 \cdot \Phi + a_3 \cdot \frac{A}{L_r} + a_4 \cdot \frac{1}{\ell_3} + a_5 \cdot \frac{A}{L_r} \cdot \frac{1}{\ell_3}$$

$$t_i = (-28{,}8) + 1{,}55 \cdot 62 + (-12{,}6) \cdot \Phi + 0{,}33 \cdot 25{,}6 + (-735) \cdot \frac{1}{106} + 48 \cdot 25{,}6 \cdot \frac{1}{106}$$



Rib geometry factor

$$\frac{A}{L_r} = \frac{h_2 \cdot \left(\frac{\ell_1 + \ell_2}{2} \right)}{\ell_2 + 2 \cdot \sqrt{h_2^2 + \left(\frac{\ell_1 - \ell_2}{2} \right)^2}} = 25{,}6\,\text{mm}$$

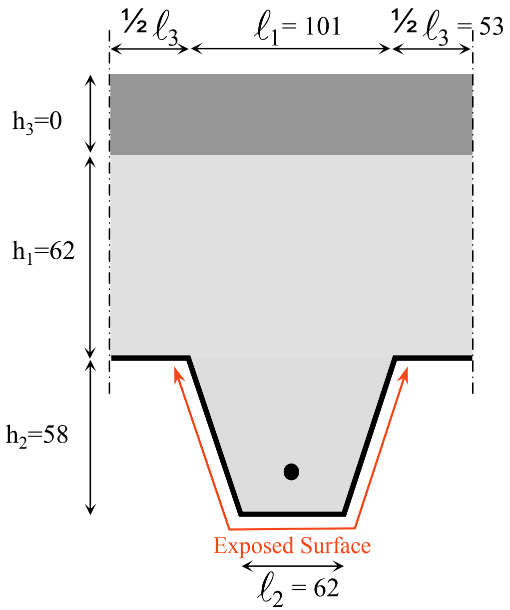
Verification of the composite slab

Fire resistance according to thermal insulation

$$t_i = a_0 + a_1 \cdot h_1' + a_2 \cdot \Phi + a_3 \cdot \frac{A}{L_r} + a_4 \cdot \frac{1}{\ell_3} + a_5 \cdot \frac{A}{L_r} \cdot \frac{1}{\ell_3}$$

$$t_i = (-28{,}8) + 1{,}55 \cdot 62 + (-12{,}6) \cdot 0{,}727 + 0{,}33 \cdot 25{,}6 + (-735) \cdot \frac{1}{106} + 48 \cdot 25{,}6 \cdot \frac{1}{106}$$

$$t_i = 71\text{min}$$



View factor

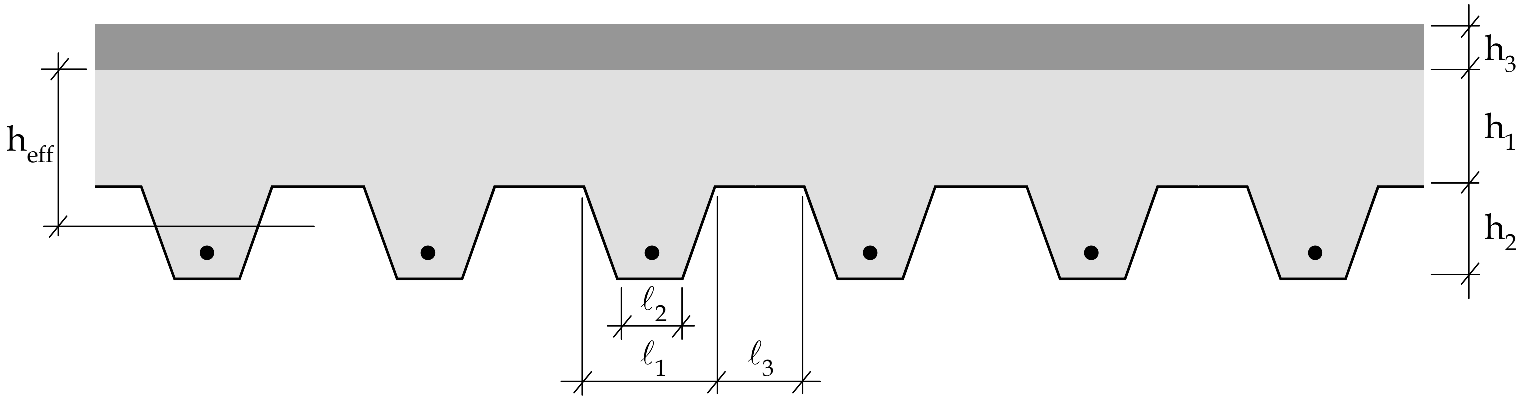
$$\Phi = \left[\sqrt{h_2^2 + \left(\ell_3 + \frac{\ell_1 - \ell_2}{2} \right)^2} - \sqrt{h_2^2 + \left(\frac{\ell_1 - \ell_2}{2} \right)^2} \right] \Big/ \ell_3 = 0{,}727$$

Verification of the composite slab

Simplified method (EN1994-1-2/Annex D/§D.4)

For $h_2/h_1 \leq 1{,}5$ and $h_1 > 40$mm

$$h_{eff} = h_1 + 0{,}5 \cdot h_2 \cdot \left(\frac{\ell_1 + \ell_2}{\ell_1 + \ell_3} \right) = 62 + 0{,}5 \cdot 58 \cdot \left(\frac{101 + 62}{101 + 106} \right) = 85\,\text{mm}$$



$h_1 = 62$ mm
$h_2 = 58$ mm
$\ell_1 = 101$ mm
$\ell_2 = 62$ mm
$\ell_3 = 106$ mm

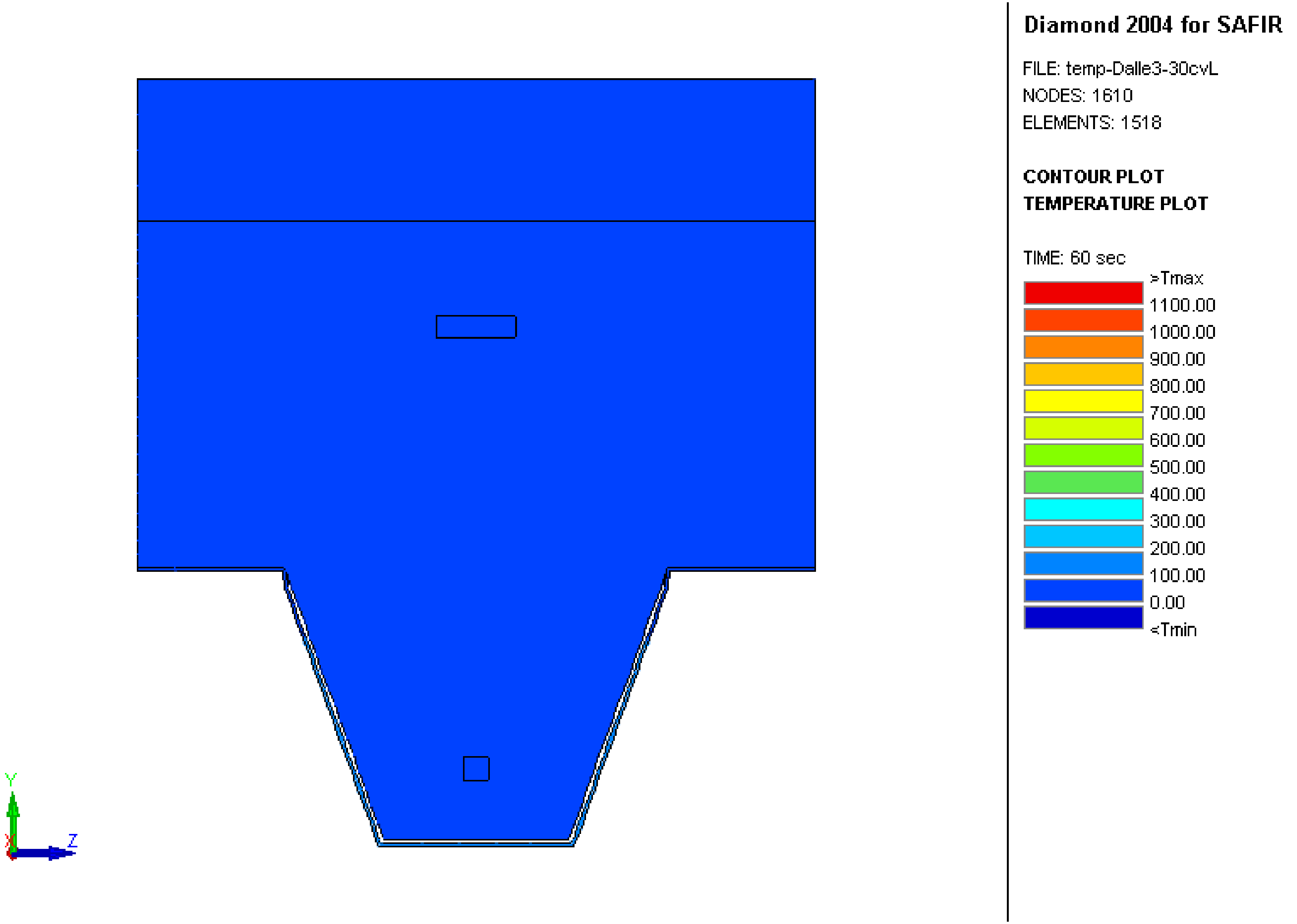
Minimal effective thickness of the slab h_{eff} to satisfy the thermal insulation criteria

Standard fire resistance	h_{eff} [mm]
I 30	$60-h_3$
I 60	$80-h_3$
I 90	$100-h_3$
I 120	$120-h_3$
I 180	$150-h_3$
I 240	$175-h_3$

$h_3 = 0$; $h_{eff} = 85$mm → **I 60**

Verification of the composite slab

Temperature evolution in the section as a function of the time



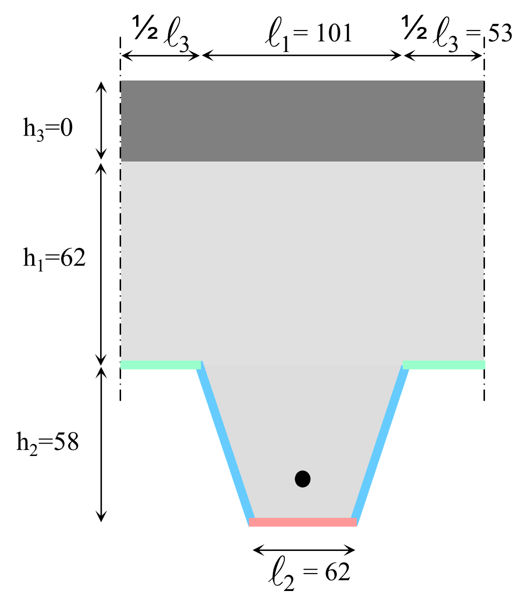
Verification of the composite slab

European Commission

Calculation of the sagging moment resistance $M^{+}_{fi,t,Rd}$

Temperature distribution in the steel decking : $\theta_a = b_0 + b_1 \cdot \frac{1}{\ell_3} + b_2 \cdot \frac{A}{L_r} + b_3 \cdot \Phi + b_4 \cdot \Phi^2$

Concrete	Fire resistance [min]	Part of the Steel decking	b_0 [°C]	b_1 [°C·mm]	b_2 [°C/mm]	b_3 [°C]	b_4 [°C]
Normal 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



For the different parts of the steel decking, the temperatures at 60 minutes are :

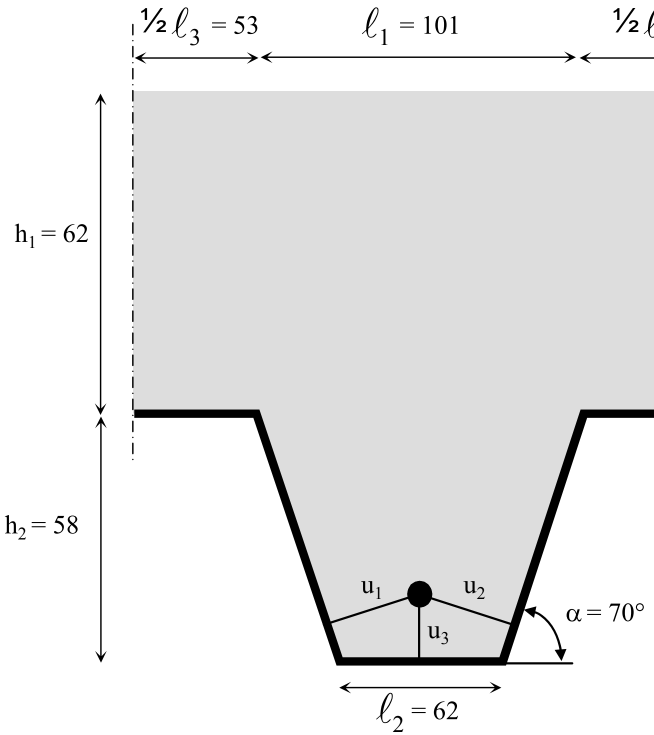
Upper Flange $\theta_a = 951 - 1197 \cdot \frac{1}{106} - 2{,}32 \cdot 25{,}6 + 86{,}4 \cdot 0{,}727 - 150{,}7 \cdot 0{,}727^2 = 863\ °C$

Web $\theta_a = 661 - 833 \cdot \frac{1}{106} - 2{,}96 \cdot 25{,}6 + 537{,}7 \cdot 0{,}727 - 351{,}9 \cdot 0{,}727^2 = 782\ °C$

Lower Flange $\theta_a = 340 - 3269 \cdot \frac{1}{106} - 2{,}62 \cdot 25{,}6 + 1148{,}4 \cdot 0{,}727 - 679{,}8 \cdot 0{,}727^2 = 718\ °C$

Verification of the composite slab

Temperature of the reinforcing bar in the rib : $\theta_s = c_0 + c_1 \cdot \frac{u_3}{h_2} + c_2 \cdot z + c_3 \cdot \frac{A}{L_r} + c_4 \cdot \alpha + c_5 \cdot \frac{1}{\ell_3}$



with $\frac{1}{z} = \frac{1}{\sqrt{u_1}} + \frac{1}{\sqrt{u_2}} + \frac{1}{\sqrt{u_3}} = \frac{1}{\sqrt{35{,}8}} + \frac{1}{\sqrt{35{,}8}} + \frac{1}{\sqrt{20}}$

$\rightarrow z = 1{,}79\ \mathrm{mm}^{0{,}5}$

Concrete	Fire resistance [min]	c_0 [°C]	c_1 [°C]	c_2 [°C/mm$^{0.5}$]	c_3 [°C/mm]	c_4 [°C/°]	c_5 [°C]
Normal 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

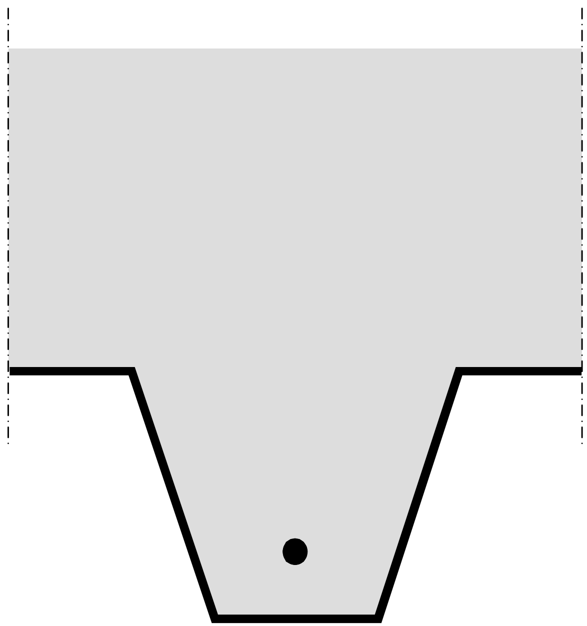
u_1 = 35,8mm
u_2 = 35,8mm
u_3 = 20mm
(axis distances)

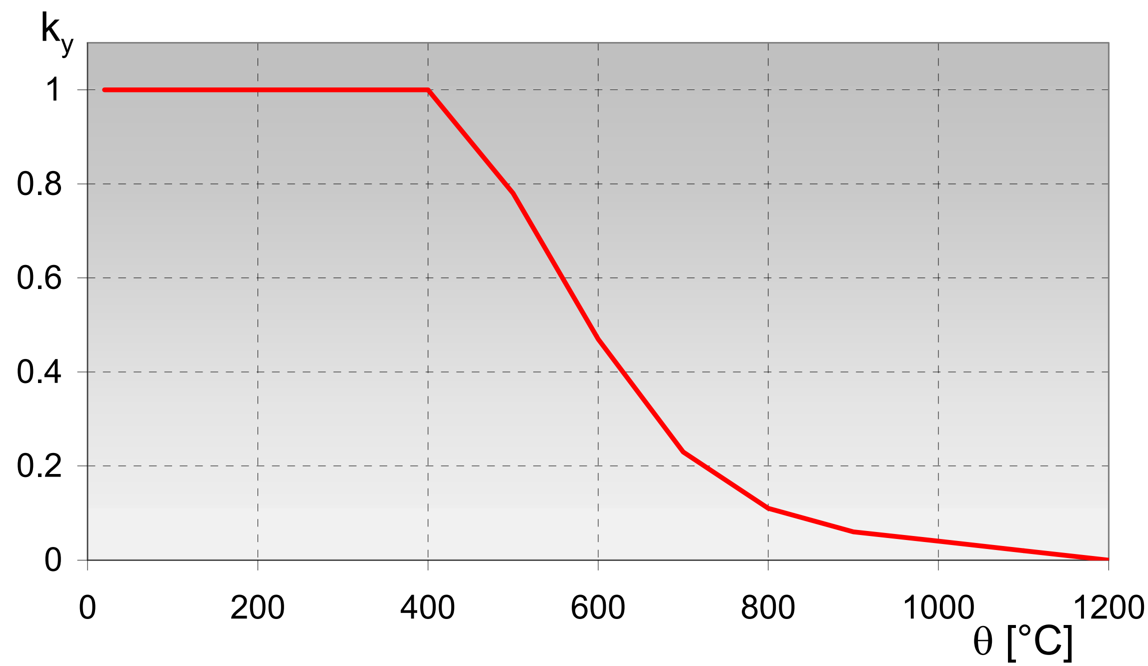
$$\theta_s = 1191 - 250 \cdot \frac{20}{58} - 240 \cdot 1{,}79 - 5{,}01 \cdot 25{,}6 + 1{,}04 \cdot 71{,}4 - 925 \cdot \frac{1}{106} = 612\,°C$$

Verification of the composite slab

Bearing capacity of the different parts of the steel decking and the reinforcing bar :

	Temperature θ_i **[°C]**	**Reduction factor** $k_{y,i}$ **[-]**	**Partial area** A_i **[cm²]**	$f_{y,i,\theta}$ **[kN/cm²]**	F_i **[kN]**
Lower flange	863	0,078	0,465	2,74	1,274
Web	782	0,131	0,918	4,60	4,221
Upper flange	718	0,209	0,795	7,31	5,813
Rebar in the rib	612	0,367	0,503	18,34	9,22

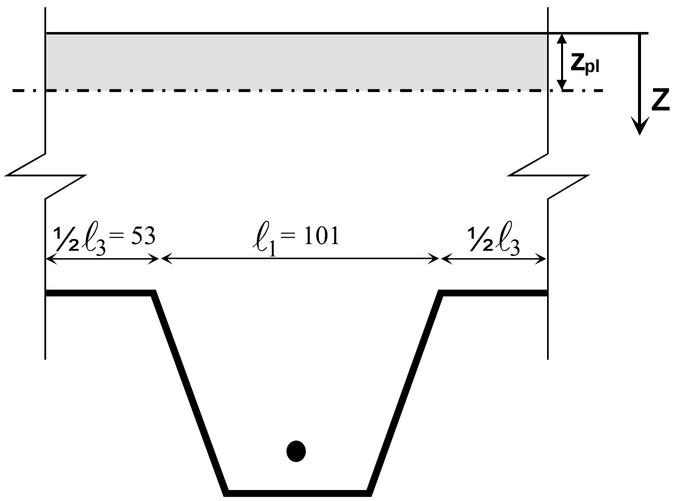




Verification of the composite slab

Positive moment of the composite slab

Determination of the plastic neutral axis :



Equilibrium of the horizontal forces

$$\Rightarrow \sum F_i = \alpha_{slab} \cdot (\ell_1 + \ell_3) \cdot z_{pl} \cdot f_c$$

$$z_{pl} = \frac{\sum F_i}{\alpha_{slab} \cdot (\ell_1 + \ell_3) \cdot f_c} = \frac{1{,}274 + 4{,}221 + 5{,}813 + 9{,}22}{0{,}85 \cdot (101 + 106) \cdot 25 \cdot 10^{-3}} = 4{,}7\,\text{mm}$$

Determination of the positive moment resistance of the composite slab :

	F_i [kN]	z_i [cm]	M_i [kNcm]
Lower flange	1,274	11,96	15,245
Web	4,221	9,10	38,410
Upper flange	5,813	6,16	35,820
Reinforcing bar in the rib	9,22	10,0	92,196
Concrete	-20,527	0,23	-4,79

for a rib width of 207mm,

$$\sum M_i = 176{,}9\,\text{kNcm}$$

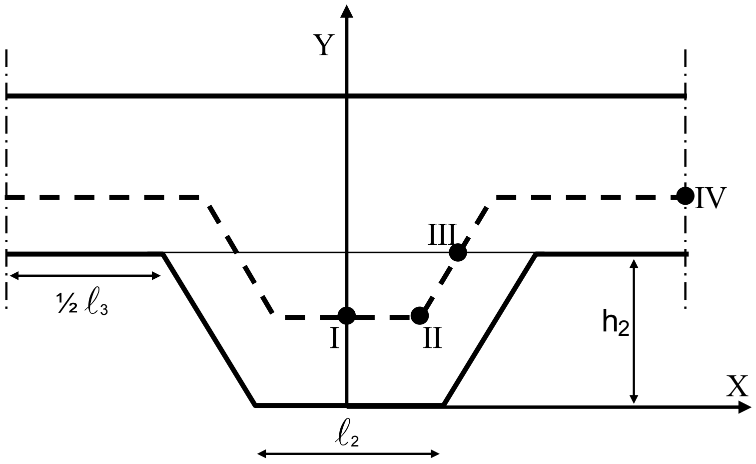
Than, for a slab width equal to 1m,

$$M^{+}_{fi,Rd} = \frac{\sum M_i}{\text{rib width}} = \frac{1{,}769}{0{,}207} = 8{,}5\,\text{kNm/m}$$

Verification of the composite slab

Calculation of the hogging moment resistance $M^{-}_{fi,t,Rd}$

The hogging moment resistance of the slab is calculated by considering a reduced cross section established on the basis of the isotherm for the limit temperature θ_{lim} schematised by means of 4 characteristic points.



$$\theta_{lim} = d_0 + d_1 \cdot N_s + d_2 \cdot \frac{A}{L_r} + d_3 \cdot \Phi + d_4 \cdot \frac{1}{\ell_3}$$

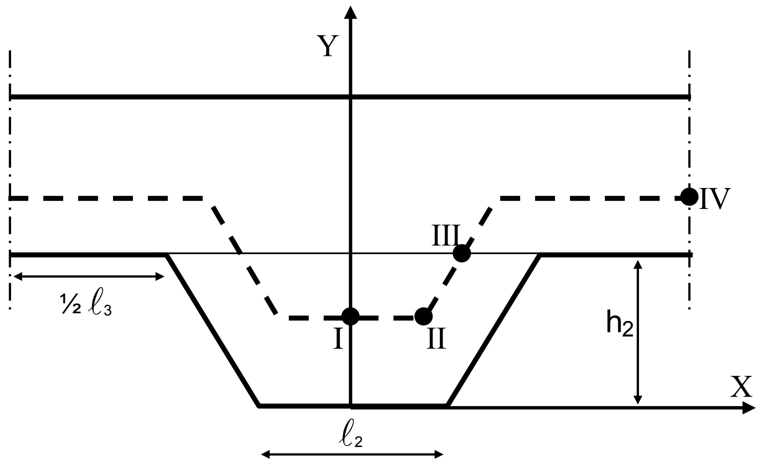
(N_s = 26,6 kN is the normal force in the upper reinforcing bar)

	Fire resistance [min]	d_0 [°C]	d_1 [°C].N	d_2 [°C].mm	d_3 [°C]	d_4 [°C].mm
Normal weight concrete	60	867	$-1{,}9.10^{-4}$	-8,75	-123	-1378
	90	1055	$-2{,}2.10^{-4}$	-9,91	-154	-1990
	120	1144	$-2{,}2.10^{-4}$	-9,71	-166	-2155

$$\theta_{lim} = 867 - 1{,}9 \cdot 10^{-4} \cdot 26600 - 8{,}75 \cdot 25{,}6 - 123 \cdot 0{,}727 - 1378 \cdot \frac{1}{106} = 535°C$$

Verification of the composite slab

Determination of the points of the isotherm :



The parameter z of the formula D.9 is obtained from the equation for the determination of the temperature of the reinforcing bar, assuming that u_3/h_2 = 0,75 and $\theta_s = \theta_{lim}$

$$\theta_{lim} = c_0 + c_1 \cdot \frac{u_3}{h_2} + c_2 \cdot z + c_3 \cdot \frac{A}{L_r} + c_4 \cdot \alpha + c_5 \cdot \frac{1}{\ell_3}$$

$$\Rightarrow z = \frac{\theta_{lim} - c_0 - 0{,}75 \cdot c_1 - 25{,}6 \cdot c_3 - 71{,}4 \cdot c_4 - c_5 \cdot \frac{1}{106}}{c_2} = \frac{535 - 1191 + 0{,}75 \cdot 250 + 25{,}6 \cdot 5{,}01 - 71{,}4 \cdot 1{,}04 + 925 \cdot \frac{1}{106}}{(-240)} = 1{,}69\,\text{mm}^{0{,}5}$$

Verification of the composite slab

Coordinates of the points of the isotherm :

The coordinates of the 4 characteristic points are determined by the following formulae :

$$Y_I = Y_{II} = \frac{1}{\left(\frac{1}{z} - \frac{4}{\sqrt{\ell_1 + \ell_3}}\right)^2}$$

$$Y_{III} = h_2$$

$$Y_{IV} = h_2 + b$$

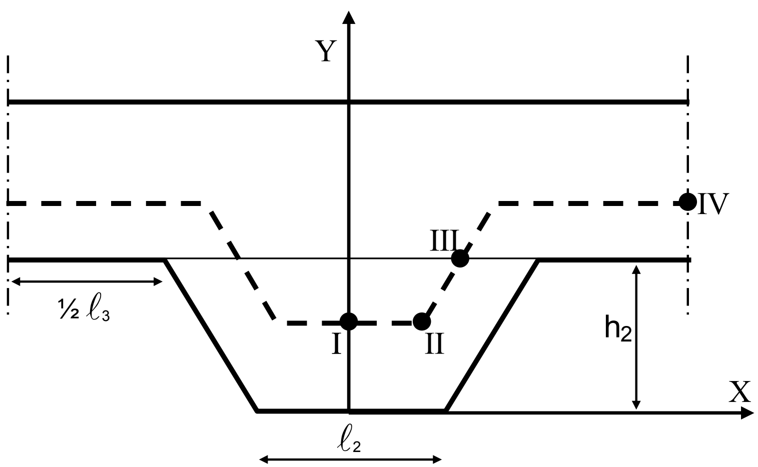
$$X_I = 0$$

$$X_{II} = \frac{1}{2}\ell_2 + \frac{Y_I}{\sin\alpha}\cdot(\cos\alpha - 1)$$

$$X_{III} = \frac{1}{2}\ell_1 - \frac{b}{\sin\alpha}$$

$$X_{IV} = \frac{1}{2}(\ell_1 + \ell_3)$$

Points	Coordinates [mm]	
	X	Y
I	0,0	10,1
II	23,7	10,1
III	42,1	58,0
IV	103,5	66,0



with

$$b = \frac{1}{2}\ell_1 \sin\alpha\left(1 - \frac{\sqrt{a^2 - 4a + c}}{a}\right)$$

$$a = \left(\frac{1}{z} - \frac{1}{\sqrt{h_2}}\right)^2 \ell_1 \sin\alpha$$

$$\alpha = \arctan\left(\frac{2\,h_2}{\ell_1 - \ell_2}\right)$$

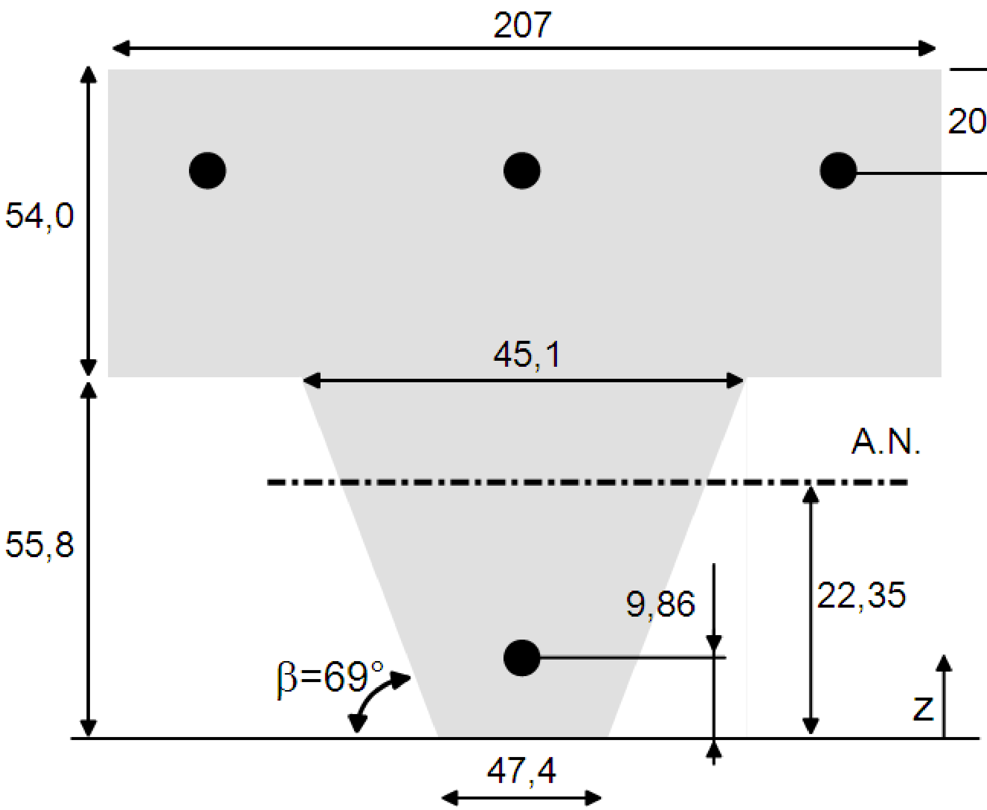
$$c = -8\left(1 + \sqrt{1+a}\right) \text{si } a \geq 8$$

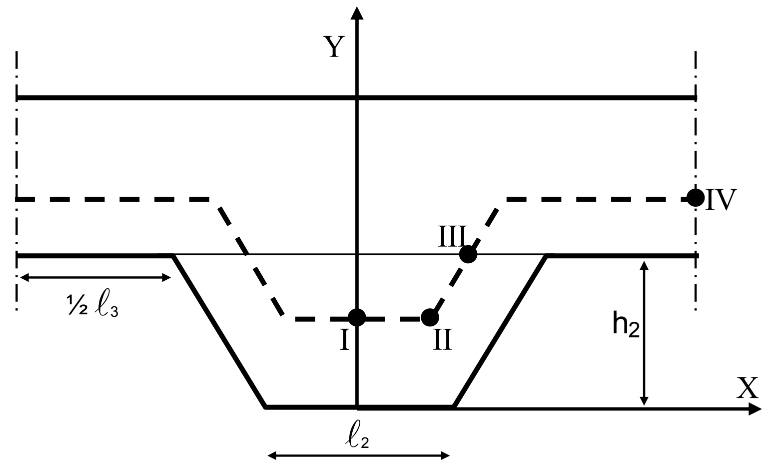
$$c = +8\left(1 + \sqrt{1+a}\right) \text{si } a < 8$$

Verification of the composite slab

European Commission

Determination of the plastic neutral axis :





	Temperature θ_i [°C]	Reduction factor $k_{y,i}$ [-]	Partial area A_i [cm²]	$f_{y,i,\theta}$ [kN/cm²]	F_i [kN]
Mesh ST25	$\theta < \theta_{lim}$	1,0	0,532	50	26,60

The horizontal equilibrium gives : $\sum F_i = \left(\frac{1}{tg\beta} \cdot z_{pl}^2 + 47{,}4 \cdot z_{pl} \right) \cdot 0{,}85 \cdot f_c$

=> z_{pl} = 22,35mm

Verification of the composite slab

Moment resistance of each part of the rib :

	F_i [kN]	z_i [cm]	M_i [kNcm]
Mesh ST25	26,60	8,9	239,01
Concrete rib	-26,60	4,5	-120,32

The negative moment resistance of the composite slab, for a rib width of 207mm is given by :

$$\sum M_i = 118{,}7\,\text{kNcm}$$

Than, for a slab width equal to 1m,

$$M^{-}_{fi,Rd} = \frac{\sum M_i}{\text{rib width}} = \frac{1{,}187}{0{,}207} = 5{,}734\,\text{kNm/m}$$

Verification of the composite slab

Bearing capacity of the continuous slab

The applied load is :

$E_{fi,d} = G_k + \psi_{1,1}\, Q_{k,1}$ ⟹ $p_{fi,d} = 1,0 * (0,085 + 2,03 + 1,5) + 0,6 * 4 = 6,02$ kN/m²

For a slab width equal to 1m, the bearing capacity may be deduced from the sagging and hogging moment by the following relation :

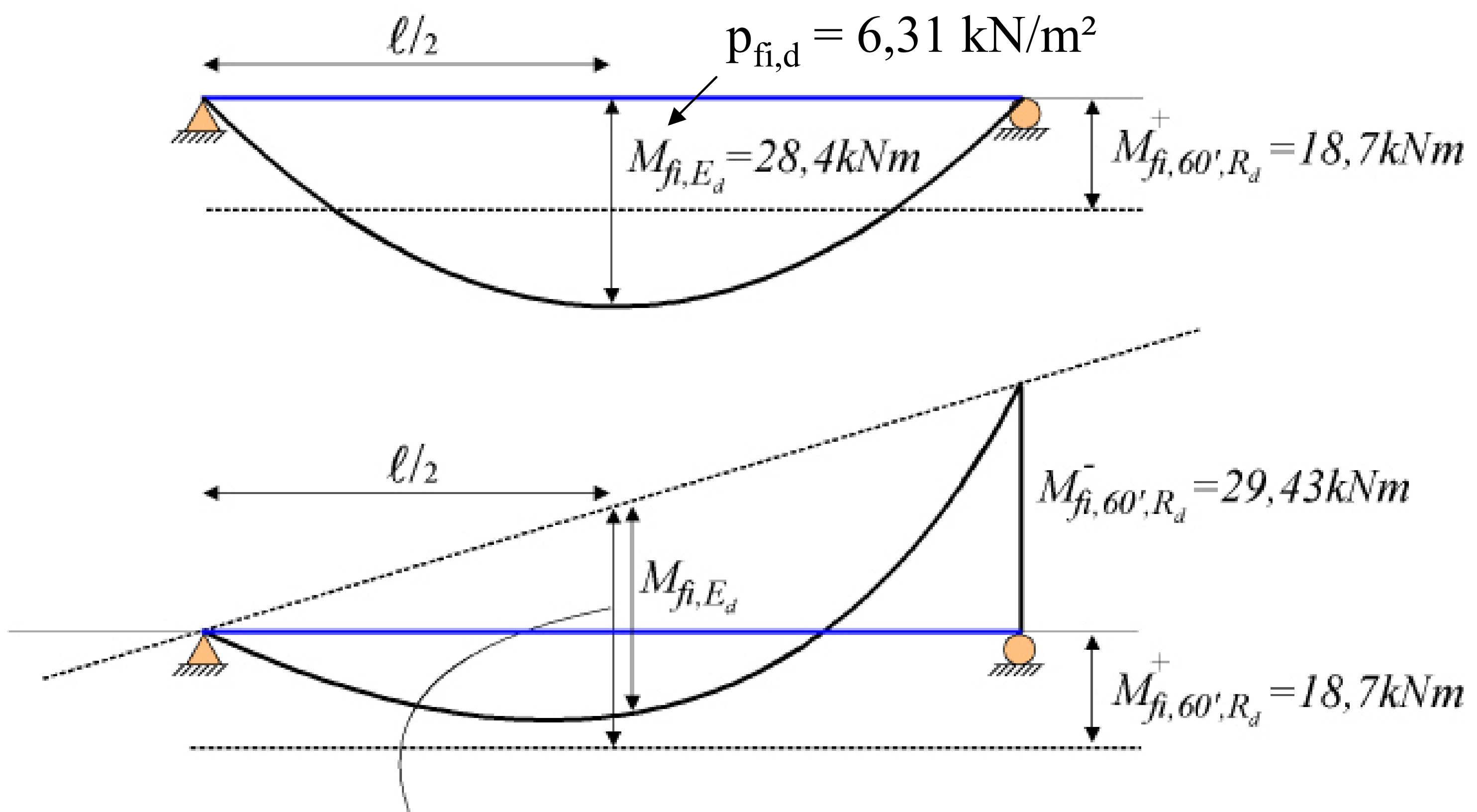
$$p_{fi,Rd} = \frac{2M^{-}_{fi,Rd} + 4M^{+}_{fi,Rd}}{\ell^2} + \frac{2}{\ell^2} \cdot \sqrt{\left(M^{-}_{fi,Rd} + 2M^{+}_{fi,Rd}\right)^2 - {M^{-}_{fi,Rd}}^2}$$

$$p_{fi,Rd} = \frac{2 \cdot 5{,}734 + 4 \cdot 8{,}5}{3^2} + \frac{2}{3^2} \cdot \sqrt{\left(5{,}734 + 2 \cdot 8{,}5\right)^2 - 5{,}734^2}$$

$p_{fi,Rd} = 9,98$ kN/m² > $p_{fi,d} = 6,02$ kN/m²

The <u>continuous</u> slab has a fire resistance of 60 minutes

Bearing capacity of the continuous slab : Simplified formula



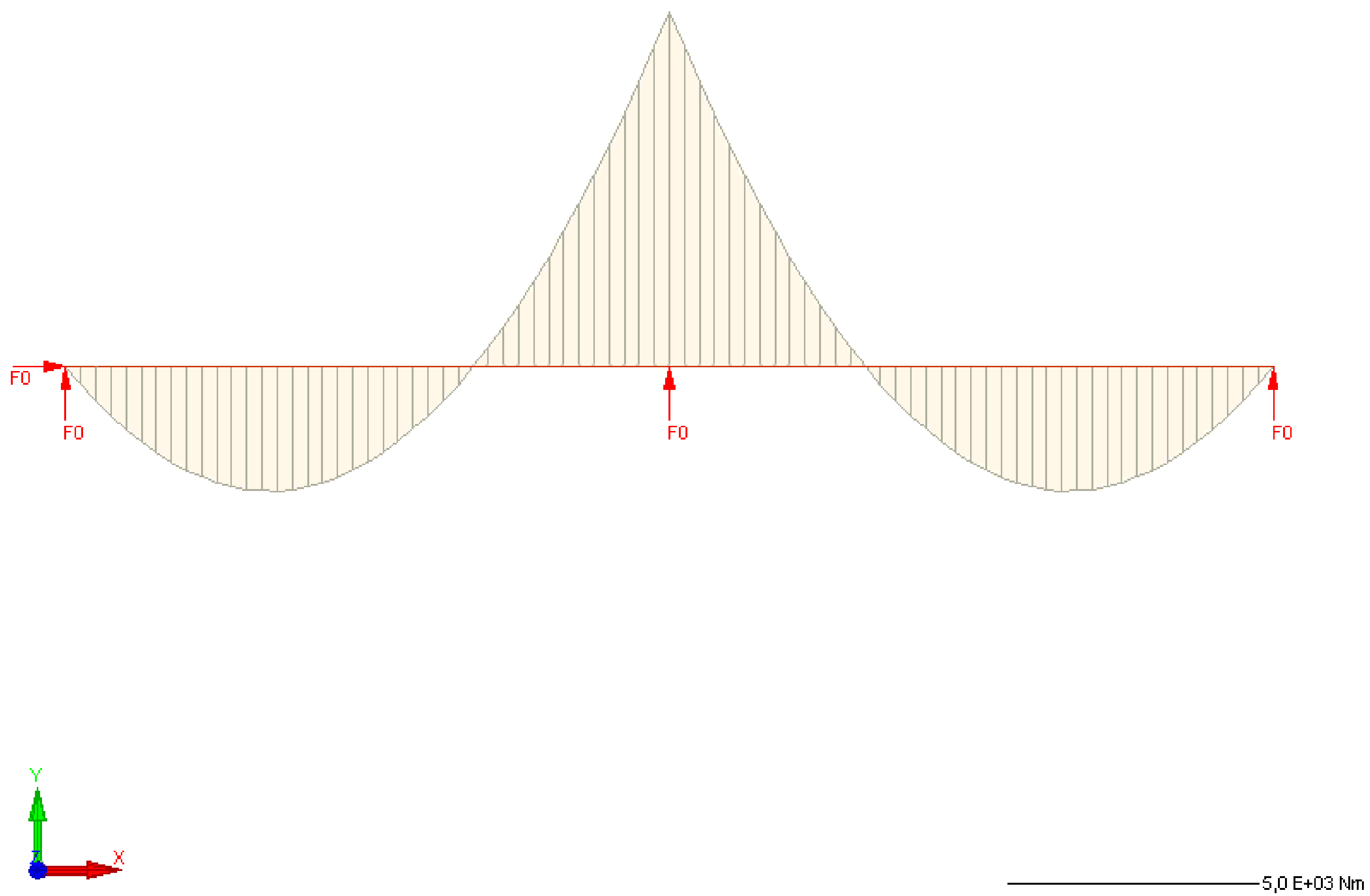
$$\frac{M^-_{fi,60',R_d}}{2} + M^+_{fi,60',R_d} \geqslant M_{fi,E_d}$$

33,4 kNm ≥ 28,4 kNm

$p_{fi,Rd}$ = 7,33 kN/m² ≥ $p_{fi,d}$ = 6,31 kN/m²

Verification of the composite slab

Evolution du moment fléchissant en fonction du temps



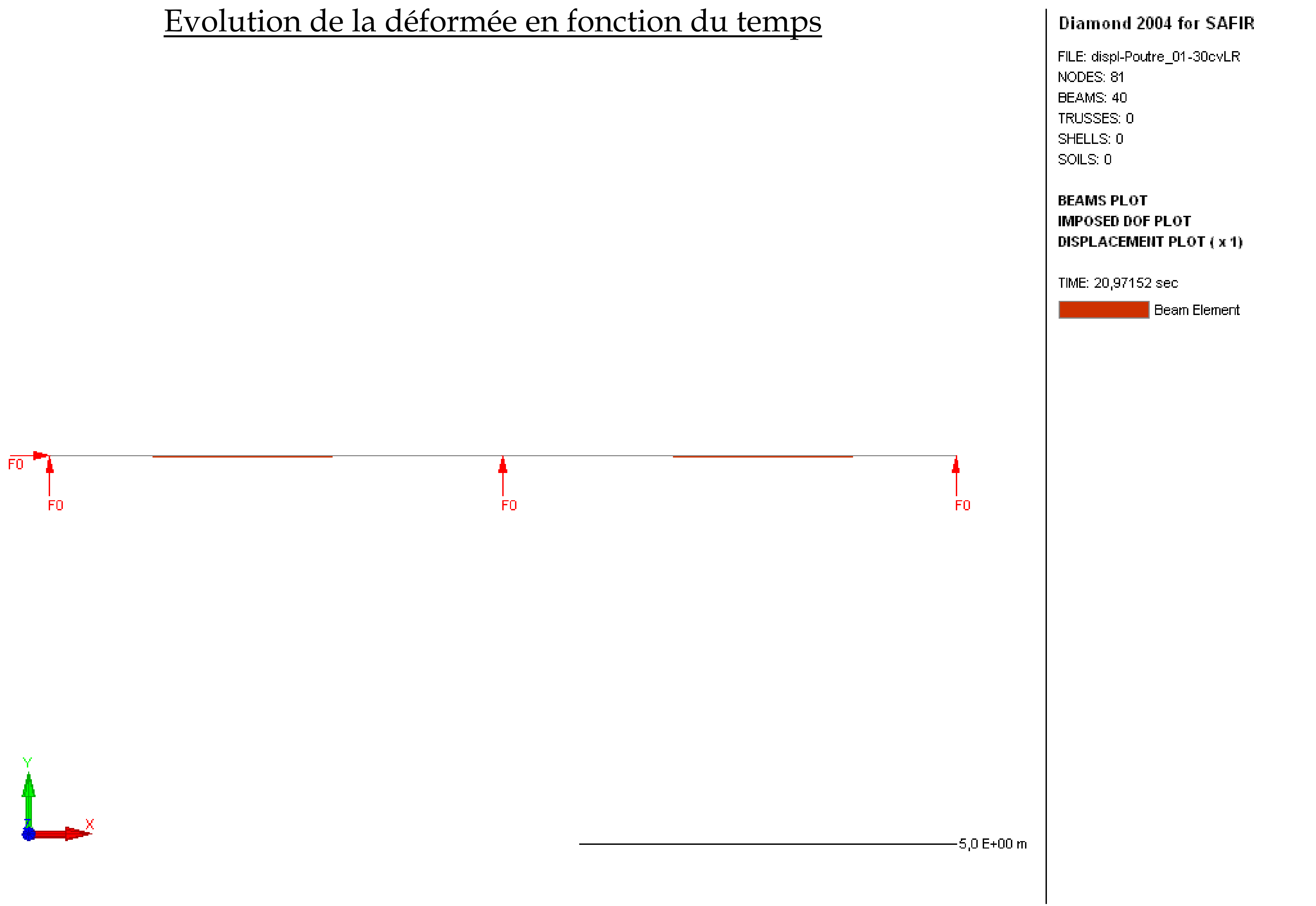
Diamond 2004 for SAFIR

FILE: displ-Poutre_01-30cvLR
NODES: 81
BEAMS: 40
TRUSSES: 0
SHELLS: 0
SOILS: 0

BEAMS PLOT
IMPOSED DOF PLOT
BENDING MOMENT PLOT

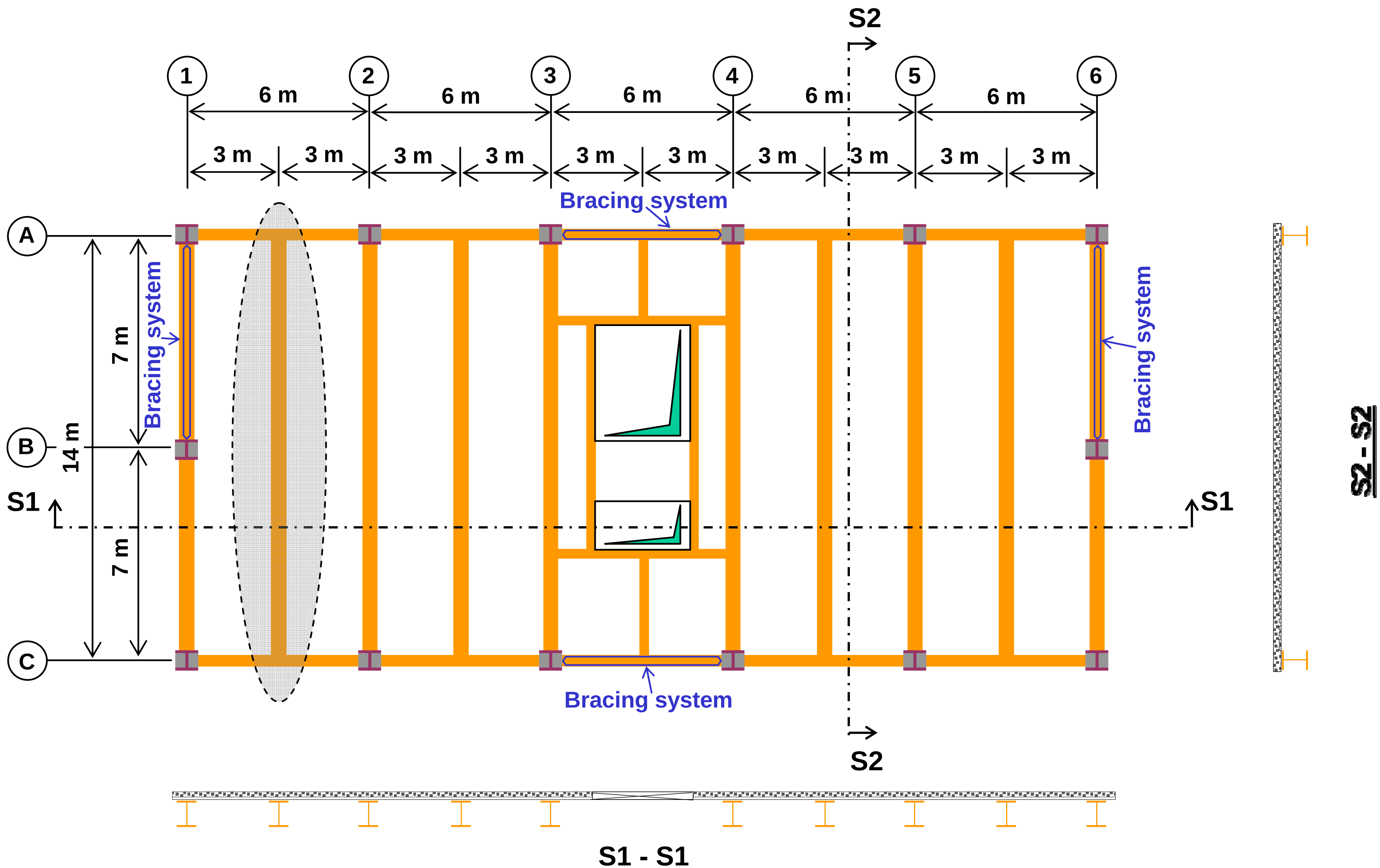
TIME: 20,97152 sec

Verification of the composite slabv



Structure grid of the composite building

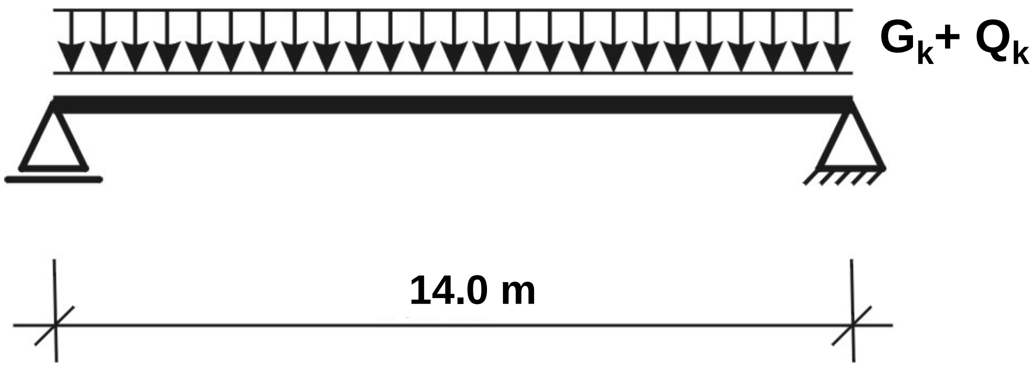
Secondary composite beams

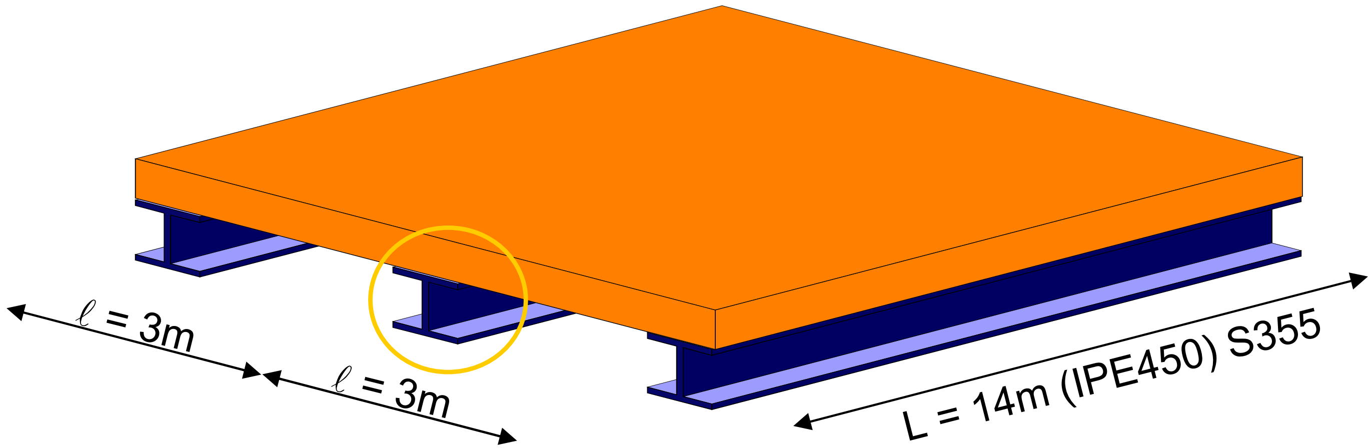


Verification of the composite beam

Data

- ⇨ Composite beam on 2 supports
- ⇨ Continuous slab on 3 supports
- ⇨ Beam span = 14m
- ⇨ Distance between beams = 3m
- ⇨ Required fire resistance R60





Verification of the composite beam

Geometrical characteristics and material properties

Beam : IPE450

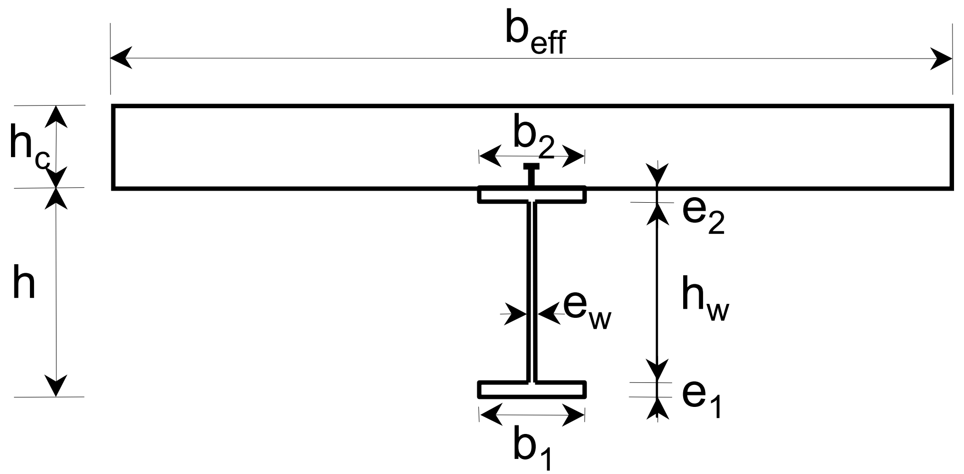
$h = 450$ mm

$b = b_1 = b_2 = 190$ mm

$e_w = 9,4$ mm

$e_f = e_1 = e_2 = 14,6$ mm

$f_y = 355$ N/mm²



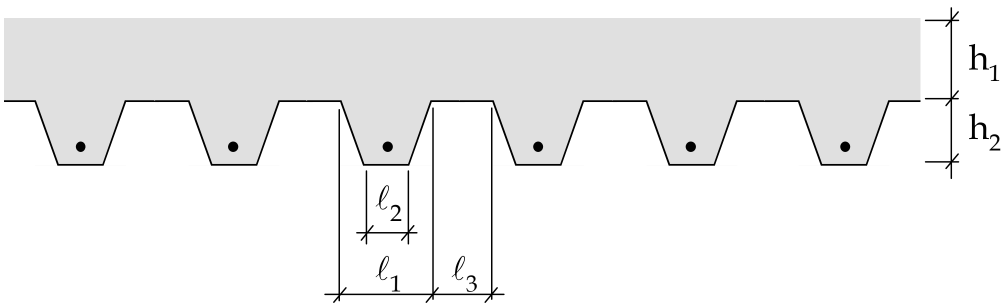
Steel decking : $f_y = 350$ N/mm²

Concrete : $h_c = 120$ mm

$b_{eff} = 3000$ mm

C25/30

$f_c = 25$ N/mm²



Connectors : $f_u = 450$ N/mm²

Number = 136

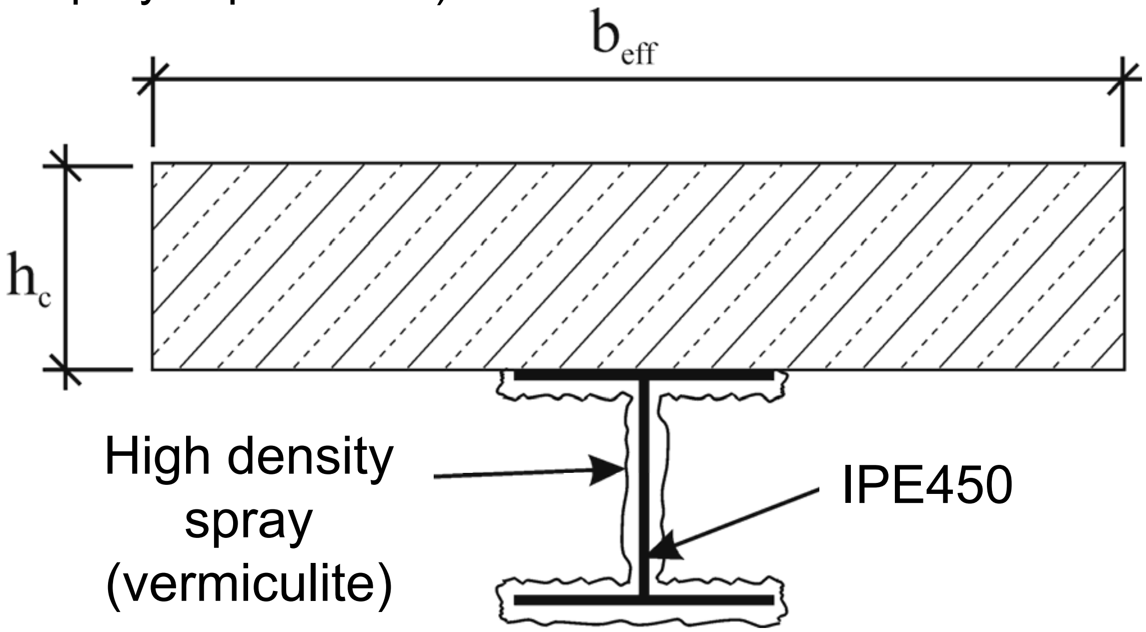
Diameter = 19 mm

$h_1 = 62$ mm $h_2 = 58$ mm

$\ell_1 = 101$ mm $\ell_2 = 62$ mm $\ell_3 = 106$ mm

Geometrical characteristics and material properties

Fire protection (use of sprayed protection)



Fire protection material characteristics :

Thickness : d_p= 15 mm

Thermal conductivity : λ_p= 0,12 W/(m·K)

Specific heat : c_p=1100 J/(kg·K)

Density: ρ_p= 550 kg/m³

Verification of the composite beam

Loads

Permanents loads:

Steel decking : $g_{t,k}$ = 0,085 kN/m²

Concrete : $g_{b,k}$ = 2,03 kN/m²

Permanent load : $g_{c,k}$ = 1,5 kN/m²

Self weight of the profile : $G_{a,k}$ = 0,776 kN/ml

Variable load:

Variable load : q_k = 4,0 kN/m²

Verification of the composite beam

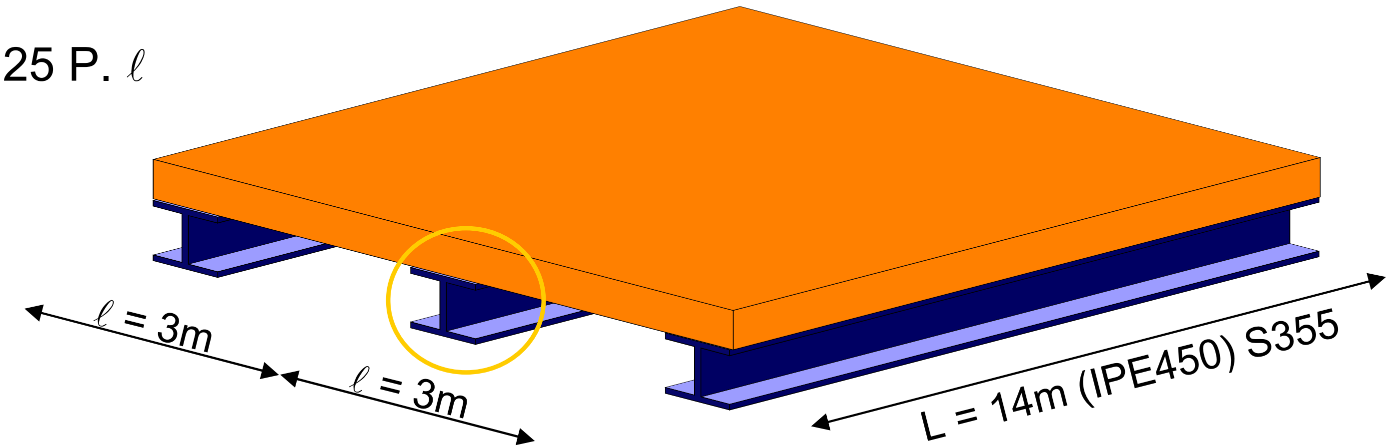
Determination of the load level in fire situation

$$\eta_{fi,t} = \frac{E_{fi,d,t}}{R_d} = \frac{M_{fi,Ed}}{M_{Rd}}$$

Combination of the mechanical actions: $E_d = E\left\{\sum_{j\geq1} G_{k,j} + P + A_d + (\psi_{1,1} \text{ ou } \psi_{2,1})Q_{k,1} + \sum_{i\succ1} \psi_{2,i}Q_{k,i}\right\}$

System considered:

Central support : reaction = 1,25 P. ℓ



$$F_{fi,d} = 1{,}25 \cdot \left[(g_{k,1} + \psi_{2,1} \cdot q_{k,1}) \cdot \ell\right] + G_{a,k} = 1{,}25 \cdot \left[(3{,}62 + 0{,}6 \cdot 4) \cdot 3\right] + 0{,}776 = 23{,}332 \,\text{kN/m}$$

Calculated moment:

$$M_{fi,Ed} = \frac{F_{fi,d} \cdot L^2}{8} = \frac{23{,}332 \cdot 14^2}{8} = 571{,}6 \,\text{kNm}$$

$$M_{Rd} = 1065{,}3 \text{ kNm} \quad \text{(see Calculation Note)}$$

$$\Rightarrow \quad \eta_{fi,t} = \frac{571{,}6}{1065{,}3} = 0{,}537$$

Verification of the composite beam

Critical temperature method

Determination of the critical T°

For R60 → $\eta_{fi,t} = f_{ay,\theta cr} / f_{ay}$

→ $\frac{f_{ay,\theta cr}}{f_{ay}} = 0{,}537$

→ $\theta_{cr} = 578°C$

Resistance

% of the nominal value

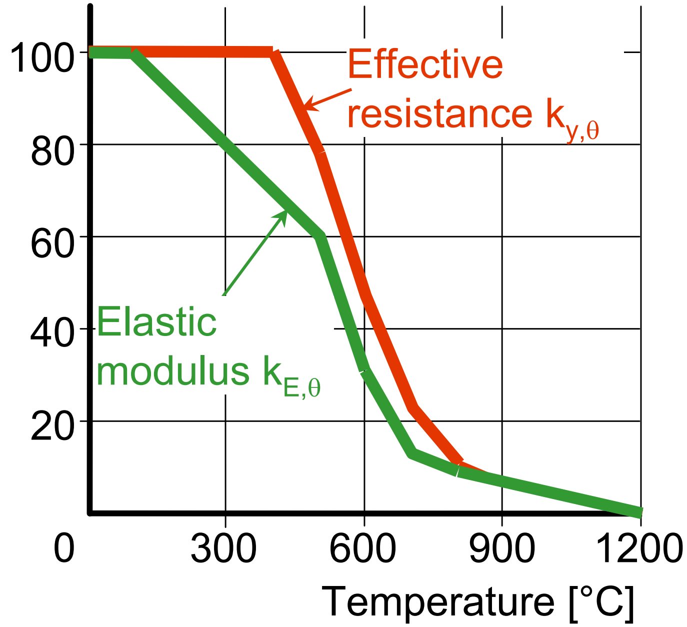


Table 3.2: Reduction factors k_θ for stress-strain relationships of structural steel at elevated temperatures.

Steel Temperature θ_a[°C]	$k_{E,\theta}$	$k_{p,\theta}$	$k_{y,\theta}$	$k_{u,\theta}$
20	1,00	1,00	1,00	1,25
100	1,00	1,00	1,00	1,25
200	0,90	0,807	1,00	1,25
300	0,80	0,613	1,00	1,25
400	0,70	0,420	1,00	
500	0,60	0,360	0,78	
600	0,31	0,180	0,47	
700	0,13	0,075	0,23	
800	0,09	0,050	0,11	
900	0,0675	0,0375	0,06	
1000	0,0450	0,0250	0,04	
1100	0,0225	0,0125	0,02	
1200	0	0	0	

Verification of the composite beam

Temperature calculation in the protected steel cross section

The increase of temperature of the various parts of the protected steel beam during the time interval may be determined by the following equation :

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

with :

C_a	specific heat of the steel ; varying according to the steel temperature [J/(kg.K)] (§3.3.1(4))
ρ_a	density of the steel [kg/m^3] (§ 3.4(1))
λ_p	thermal conductivity of the fire protection material [W/m°K]
d_p	thickness of the fire protection material [m]
$A_{p,i}$	is the area of the inner surface of the fire protection material per unit length of the part i of the steel member [m²/m]
V_i	volume of the part i of the steel cross section per unit length [m^3/m]
$A_{p,i}/V_i$	section factor of the part i of the insulated steel cross-section [m^{-1}]
Δt	time interval (less than 5sec) [s]

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

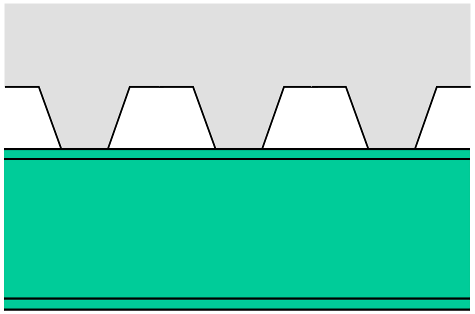
where

c_p specific heat of the fire protection material [J/kg°K]

ρ_p density of the fire protection material [kg/m^3]

Verification of the composite beam

Temperature calculation in the protected steel cross section

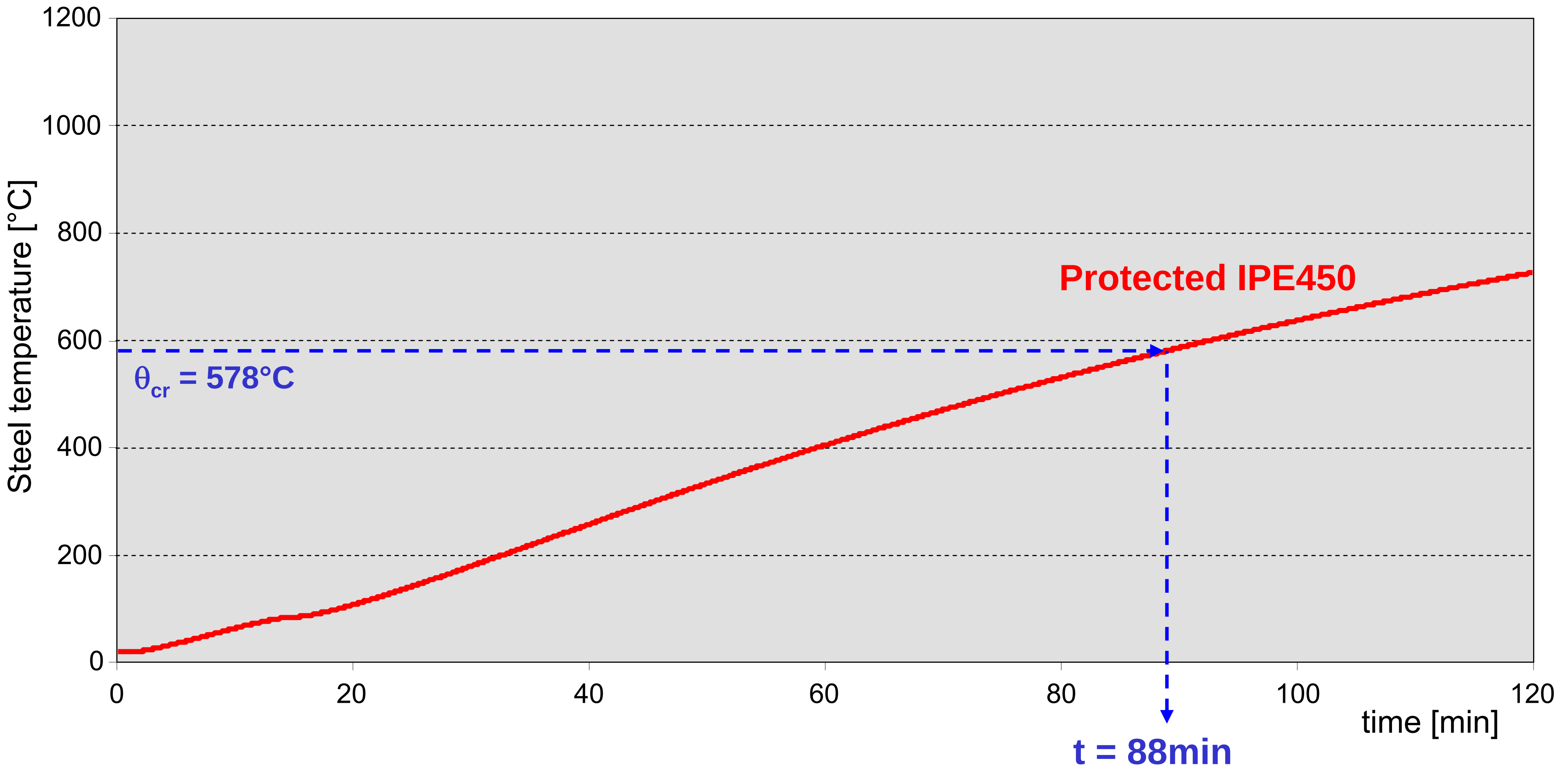


T° lower flange ≅ T° upper flange

	A_m/V [m^{-1}]	Steel temperature after 60' [°C]
Upper flange	147,5	480
Web	212,8	588
Lower flange	147,5	480

Verification of the composite beam

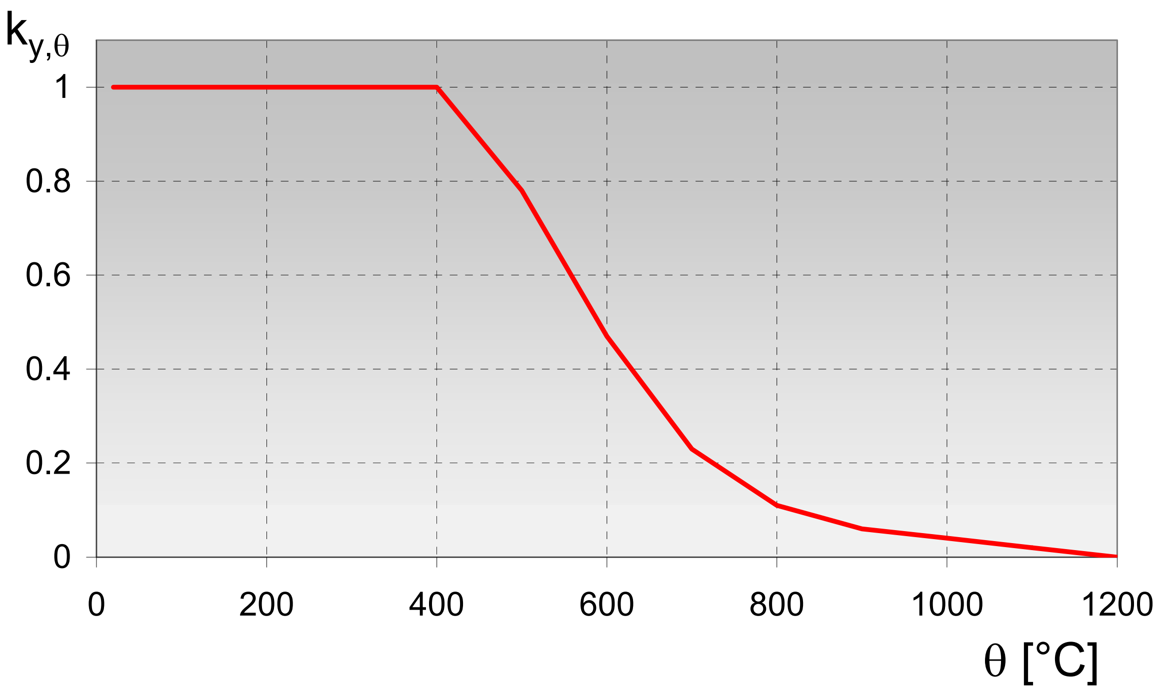
Determination of the failure time

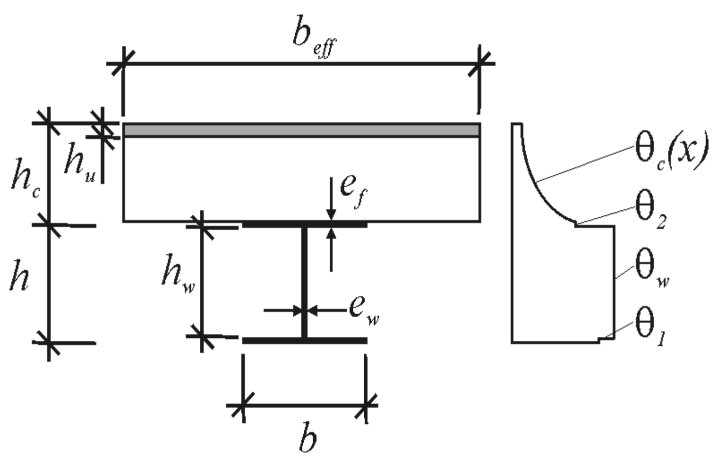


Verification of the composite beam

Verification of the resistance by the moment resistance method

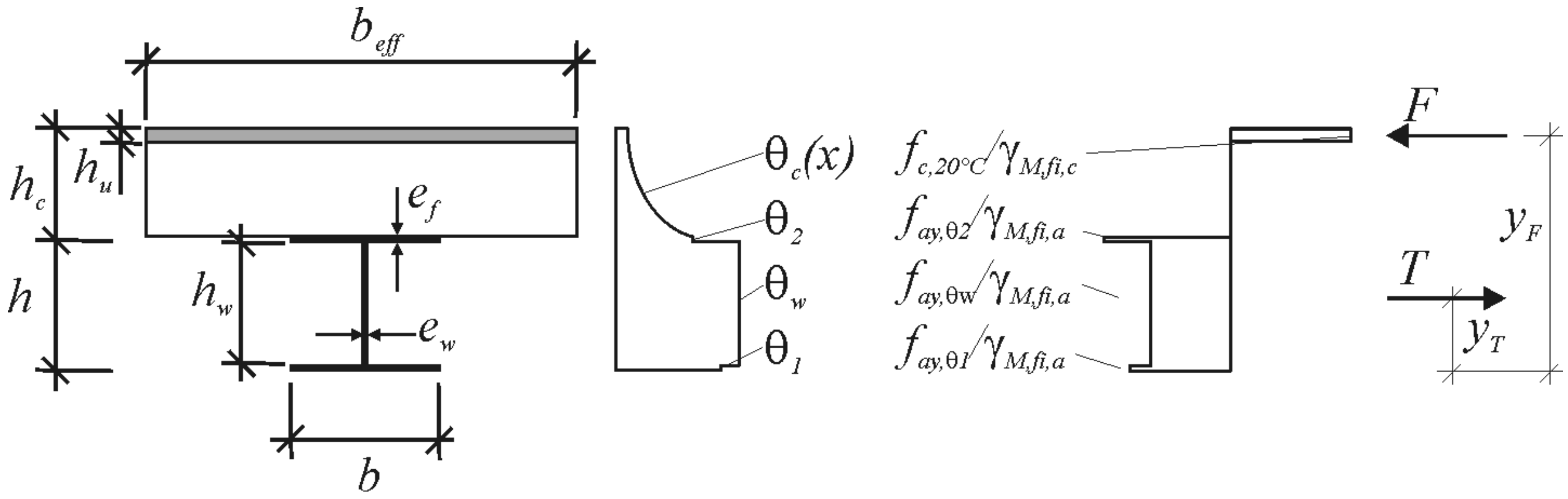
	$\theta_{a,max,30}$ [°C]	$k_{y,\theta}$ [-]	$f_{ay,\theta}$ [N/mm²]
Upper flange	480	0,824	292,5
Web	588	0,507	179,9
Lower flange	480	0,824	292,5





Verification of the composite beam

Determination of the tensile force in the profile



The steel profile is subjected to a tensile force T which could be calculated by the following way

$$T = \frac{f_{ay,\theta 1} \cdot b \cdot e_f + f_{ay,\theta w} \cdot h_w \cdot e_w + f_{ay,\theta 2} \cdot b \cdot e_f}{\gamma_{M,fi,a}} = 2334{,}096\,\text{kN}$$

The location of the tensile force (with regard to the bottom flange) is given by the following equation :

$$y_T = \frac{f_{ay,\theta 1} \cdot \left(\frac{b \cdot e_f^2}{2}\right) + f_{ay,\theta w}\left(h_w \cdot e_w\right)\left(e_f + \frac{h_w}{2}\right) + f_{ay,\theta 2}\left(b \cdot e_f\right)\left(h - \frac{e_f}{2}\right)}{T \cdot \gamma_{M,fi,a}} = 222{,}6\,\text{mm}$$

Verification of the composite beam

Limitation of the tensile force

$T \leq N \cdot P_{fi,Rd}$ with N is the number of connectors in the critical length of the beam

$P_{fi,Rd}$ is the design shear resistance of one connector in fire situation

$$P_{fi,Rd} = \min \begin{cases} P_{fi,Rd,1} = 0{,}8 \cdot k_{u,\theta} \cdot P'_{Rd,1} \\ P_{fi,Rd,2} = k_{c,\theta} \cdot P'_{Rd,2} \end{cases}$$

where $$P'_{Rd,1} = 0{,}8 \cdot \frac{f_u}{\gamma_{M,fi,v}} \cdot \frac{\pi . d^2}{4} = 0{,}8 \cdot \frac{450}{1{,}0} \cdot \frac{\pi . 19^2}{4} = 102\,kN$$

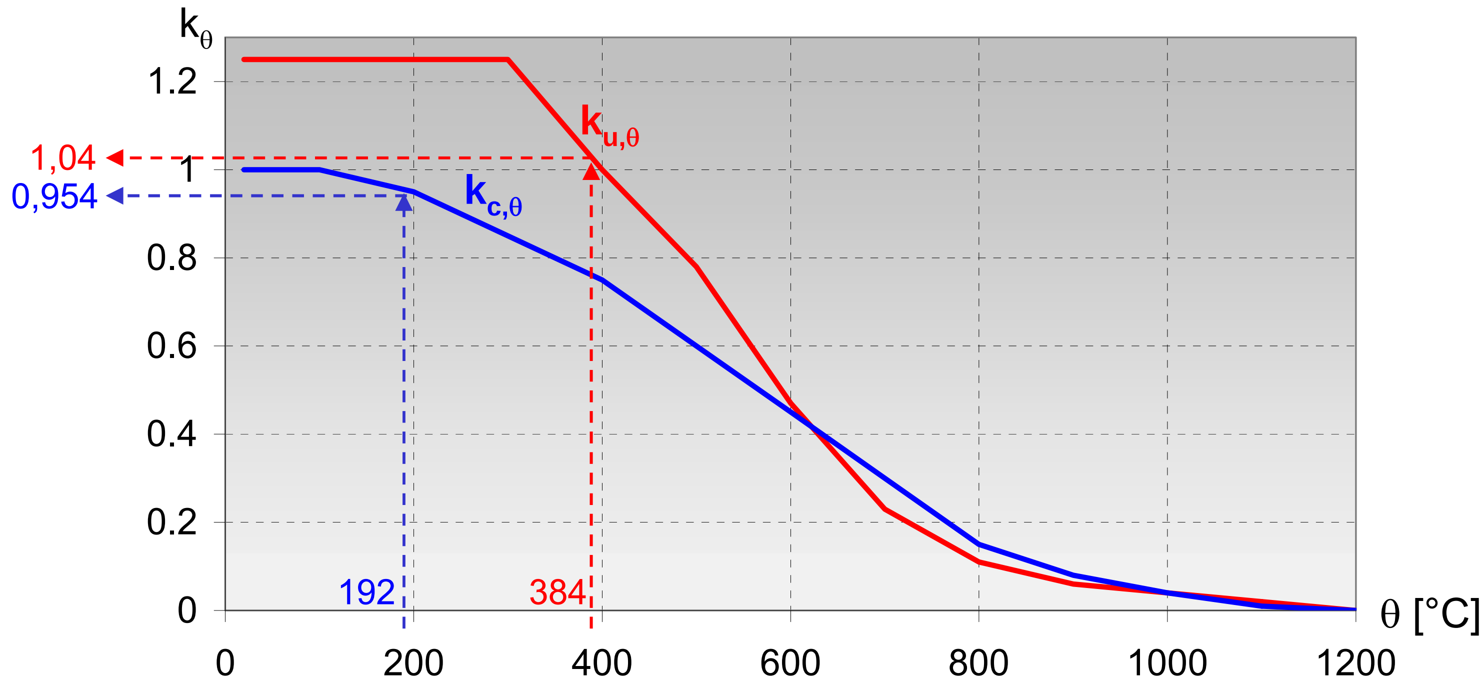
and $$P'_{Rd,2} = 0{,}29 \cdot \alpha \cdot d^2 \cdot \frac{\sqrt{f_c \cdot E_{cm}}}{\gamma_{M,fi,v}} = 0{,}29 \cdot 1{,}0 \cdot 19^2 \cdot \frac{\sqrt{25 \cdot 30500}}{1{,}0} = 91 kN$$

Verification of the composite beam

Determination of the reduction factors

θ_v (connectors) = 80% $\theta_{semelle}$ = 0,8 x 480 = 384°C $\Rightarrow k_{u,\theta} = 1{,}04$

θ_c (concrete) = 40% $\theta_{semelle}$ = 0,4 x 480 = 192°C $\Rightarrow k_{c,\theta} = 0{,}954$



$$P_{fi,Rd} = \min \begin{cases} P_{fi,Rd,1} = 0{,}8 \cdot k_{u,\theta} \cdot P'_{Rd,1} = 0{,}8 \cdot 1{,}04 \cdot 102 = 84{,}9\,kN \\ P_{fi,Rd,2} = k_{c,\theta} \cdot P'_{Rd,2} = 0{,}954 \cdot 91 = 87{,}21\,kN \end{cases}$$

Limit of the tensile force is fulfilled :

$T \leq N \cdot P_{fi,Rd}$ ⇒ $2334{,}096\,kN < 68 \cdot 84{,}9 = 5774\,kN$ ⇒ **OK**

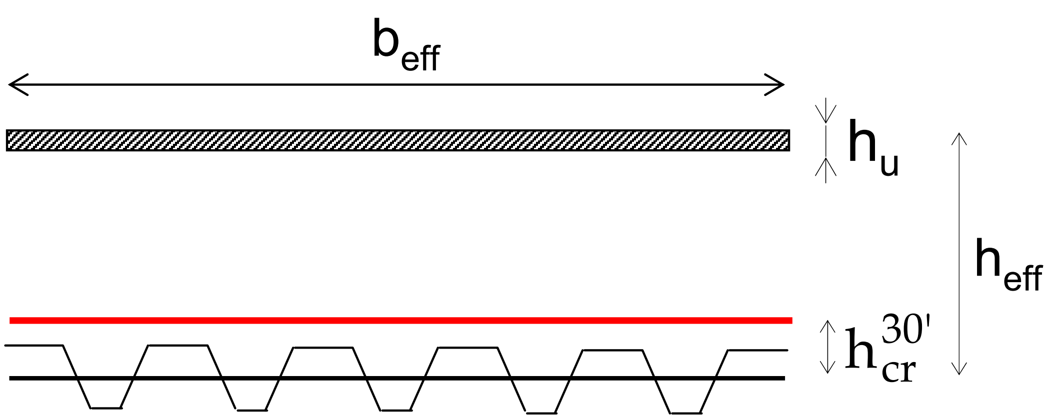
Determination of the compressive zone of the slab

Determination of the effective thickness of the slab $h_{eff} = 84{,}8\,cm$

Determination of the critical height $h_{cr}^{60'} = 50mm$

Determination of the thickness of the compressive zone of the concrete :

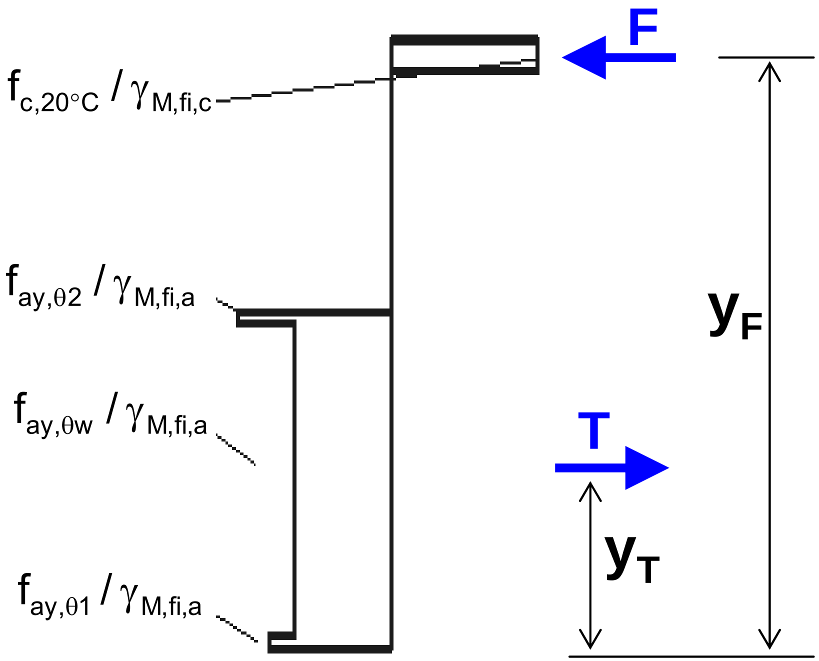
$$h_u = \frac{T}{b_{eff} \cdot f_c / \gamma_{M,fi,c}} = \frac{2334{,}096}{3000 \cdot 25 / 1{,}0} = 31{,}12\,mm$$

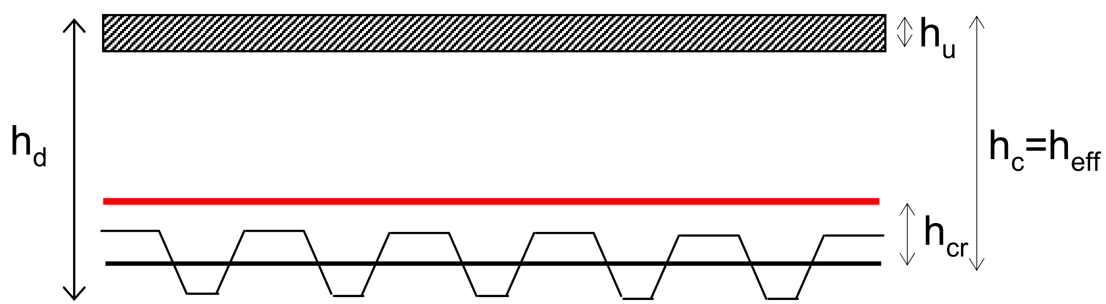


h_{cr} [mm]	Temperature [°C]	
	30'	60'
5	535	705
10	470	642
15	415	581
20	350	525
25	300	469
30	250	421
35	210	374
40	180	327
45	160	289
50	140	250

Verification of the composite beam

Determination of the moment resistance





Location of the force F :

$$y_F \approx h + h_c - (h_u/2) = 52\,cm$$

$$M_{fi,Rd} = T \cdot (y_F - y_T) = 692{,}4\,kNm$$

Verification : $\dfrac{M_{fi,Ed}}{M_{fi,Rd}} = \dfrac{571{,}6}{692{,}4} = 0{,}83 < 1$

The stability of the beam in fire situation is fulfilled for R60

Verification of the composite beam

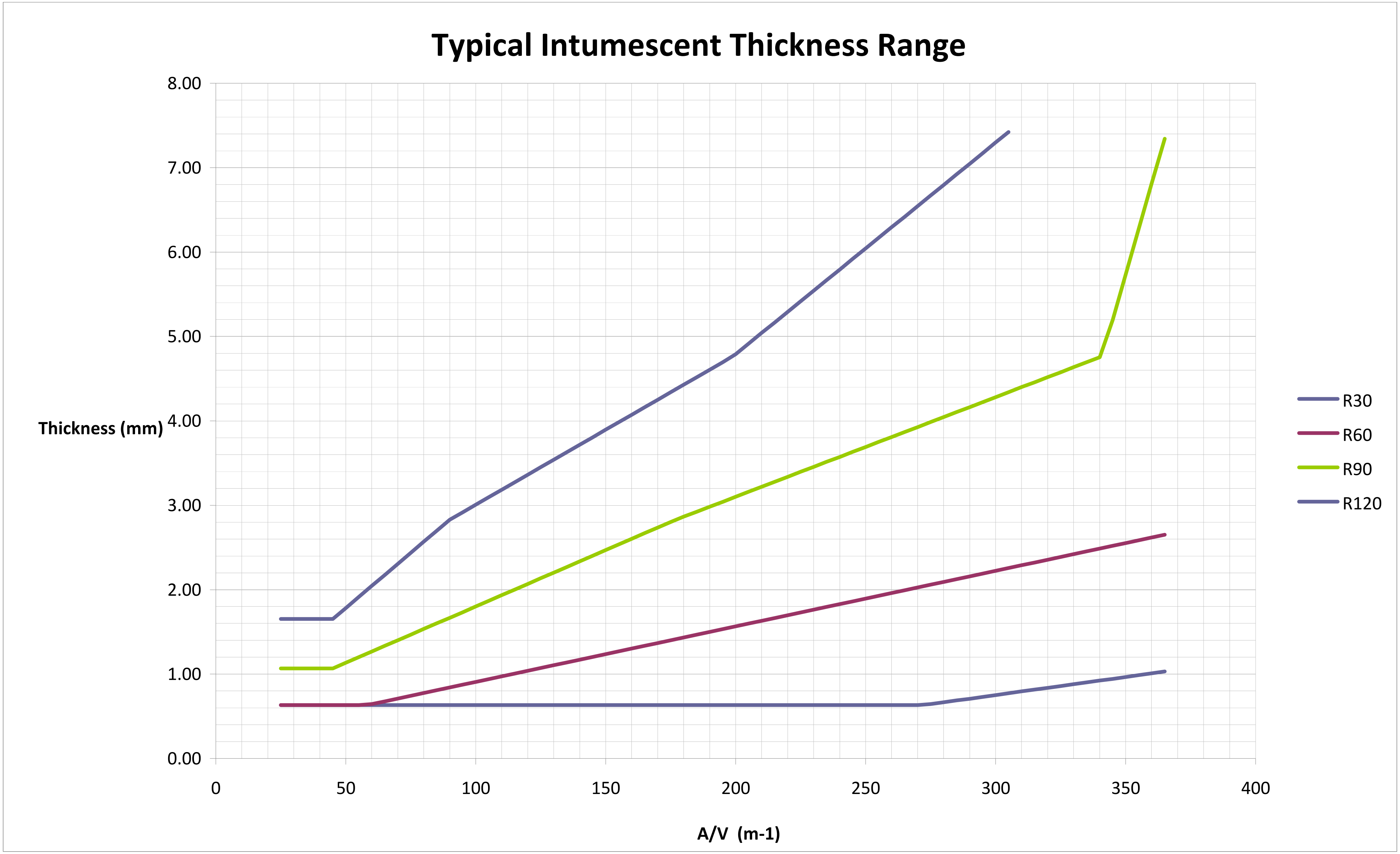
Verification of the shear resistance

$$V_{pl,fi,Rd} = A_v \cdot \frac{f_{ay,\theta}}{\sqrt{3} \cdot \gamma_{M,fi,a}}$$

$$V_{pl,fi,Rd} = 5090 \cdot \frac{292{,}5}{\sqrt{3} \cdot 1{,}0} = 859{,}5\,kN > V_{fi,Ed} = \frac{p \cdot \ell}{2} = 163{,}33\ kN$$ ⇨ **OK**

Verification of the composite beam

Alternative with reactive coating painted beam



Verification of the composite beam

European Commission

ABC calculation (Alternative AF solution without fire protection)

ArcelorMittal

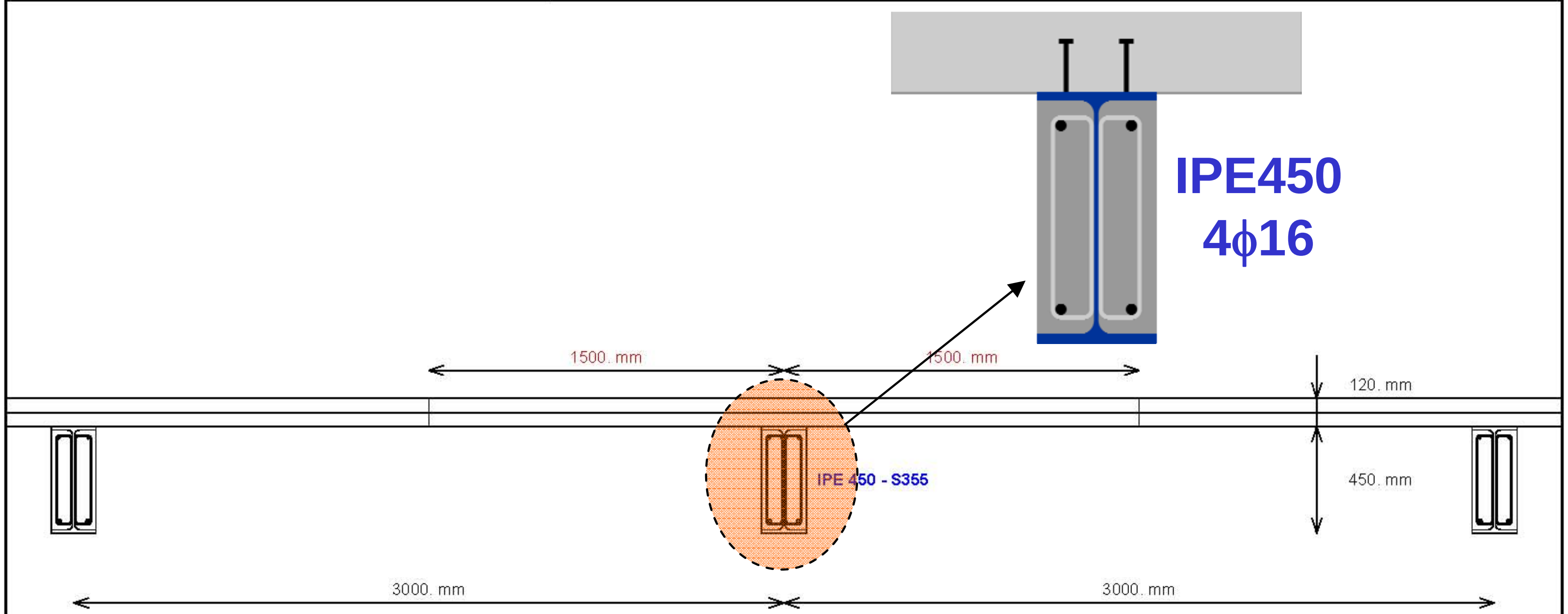
ArcelorMittal Beams Calculator v3.00 - b9

...

Nom de l'affaire :

Référence de l'affaire :

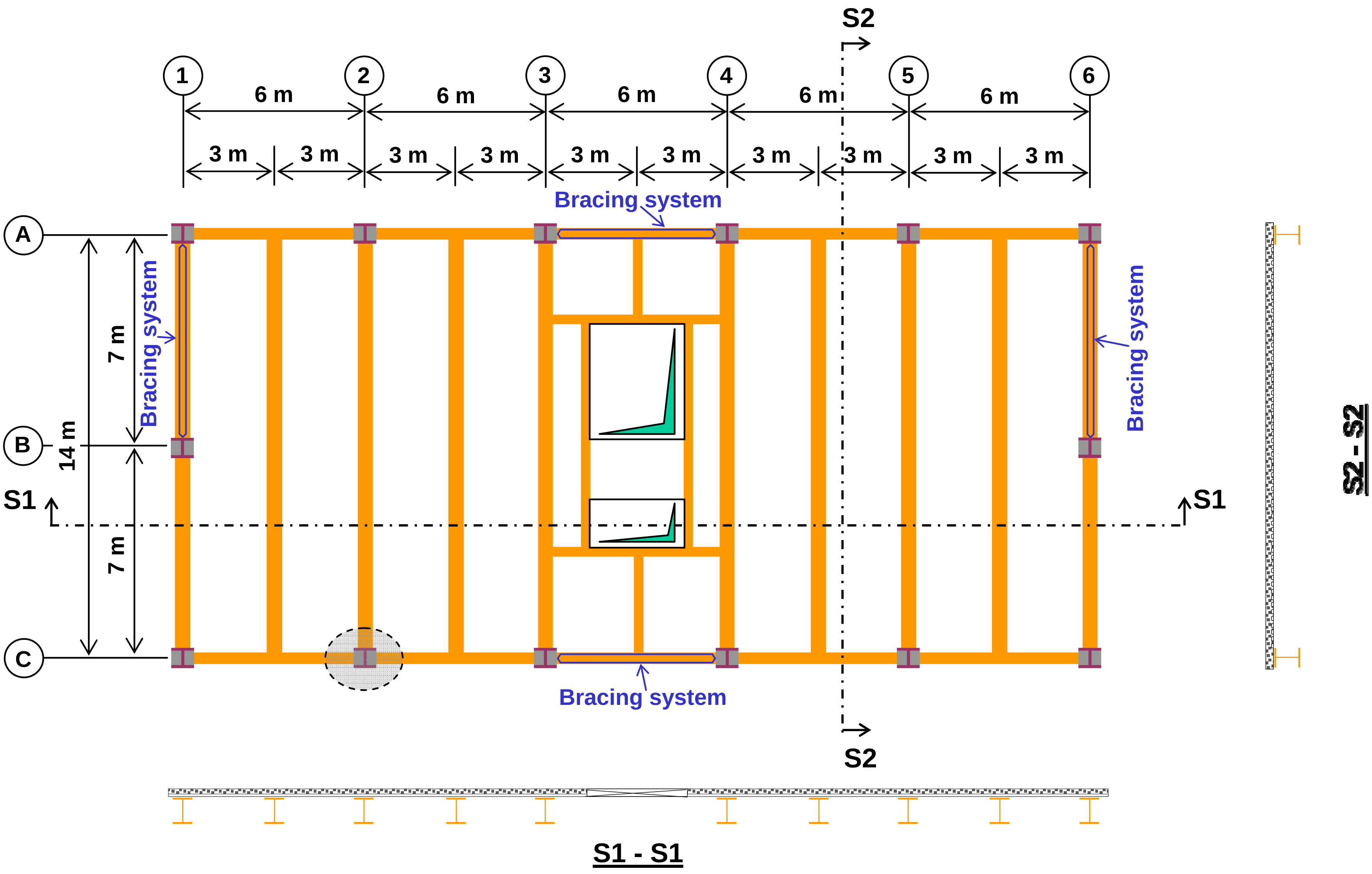
Repère de la poutre :



Date : 20-11-2012	Nom de l'utilisateur : ... D:\...\EUROCODE FIRE DESIGN WORKSHOP\POUTREISOSTATIC_IPE450AF.PMX	**Page 10 / 11**

Structure grid of the composite building

Composite columns



Verification of the composite column

Summary of data

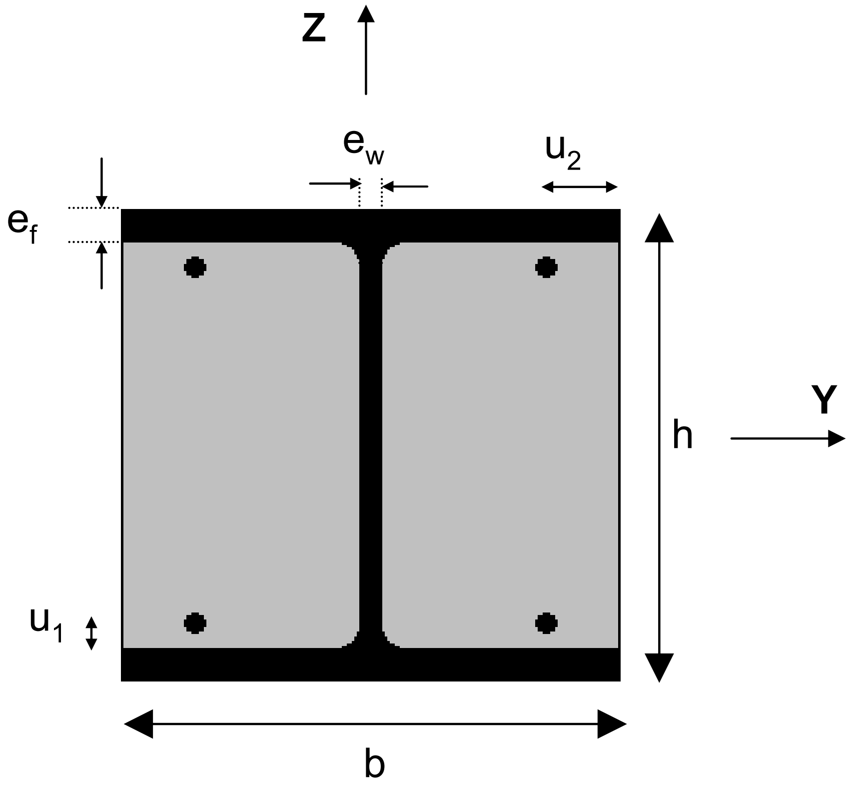
- Partially encased composite column
- Height = 3,40 m

Geometrical characteristics and material properties

Profile : HEA 260
$h = 250$ mm
$b = 260$ mm
$e_w = 7{,}5$ mm
$e_f = 12{,}5$ mm
$A_a = 8680$ mm²
$f_y = 460$ N/mm²

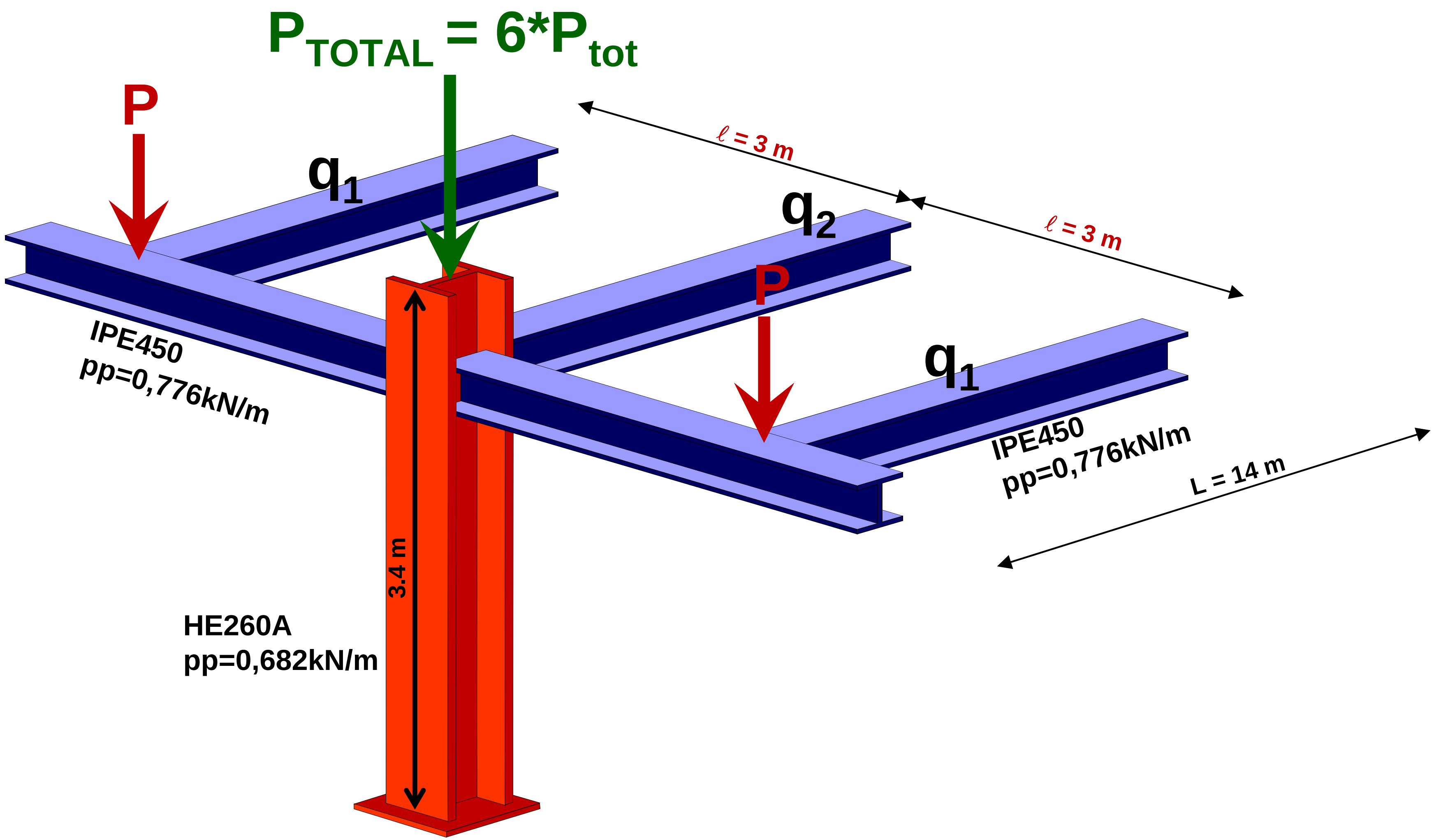
Concrete : C30/37 ; $f_c = 30$ N/mm²
$A_c = 53860$ mm²

Rebars : 4 ø 28 ; $A_s = 2463$ mm²
$u_1 = 52$mm ; $u_2 = 60$mm
$f_s = 500$ N/mm²



Verification of the composite column

Load



For one level : q_1 = 1,25 [(3,62+0,6*4,0)*3]+0,776 = 23,351 kN/m

q_2 = 0,750 [(3,62+0,6*4,0)*3]+0,776 = 14,321 kN/m

$\mathbf{P_{tot}}$ = (23,351*14/2)+ (14,321*14/2)+2*6+0,776*6+2,14*3,4 = 287,636 kN

P_{TOTAL} = 6 * 287,636 = 1726 kN

Verification of the composite column

Inertia

Profile : $I_z = 3668\ cm^4$

$I_y = 10450\ cm^4$

Rebars : $$I_{s,z} = 4 \cdot \left[\frac{\pi \cdot d^4}{64} + \frac{\pi \cdot d^2}{4} \left(\frac{b}{2} - u_2 \right)^2 \right] = 1324{,}6\ cm^4$$

$$I_{s,y} = 4 \cdot \left[\frac{\pi \cdot d^4}{64} + \frac{\pi \cdot d^2}{4} \left(\frac{h}{2} - u_1 \right)^2 \right] = 1218{,}9\ cm^4$$

Concrete : $$I_{c,z} = \frac{h.b^3}{12} - I_z - I_{s,z} = 31730\ cm^4$$

$$I_{c,y} = \frac{b.h^3}{12} - I_y - I_{s,y} = 22080\ cm^4$$

Verification of the composite column

Use of tabulated data ($0,28 < \eta_{fi,t} < 0,47$)

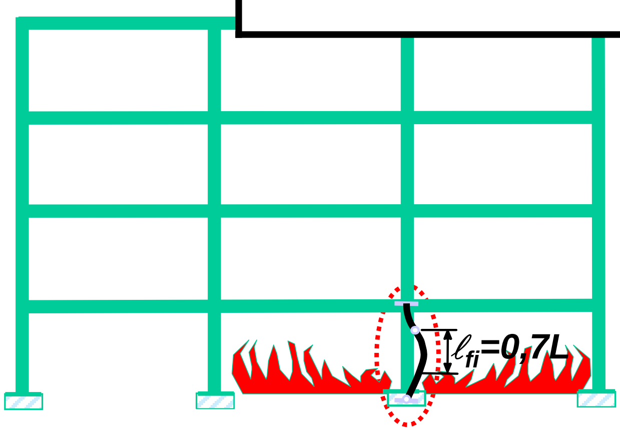
Allowed parameters R60	Existing parameters	Fulfilled conditions ?
$e_w / e_f > 0,5$	$7,5 / 12,5 = 0,6$	YES
h and b > 300 mm	h = 250 mm b = 260 mm	NO
u_1 and u_2 > 50 mm	$u_1 = 52mm$ $u_2 = 60mm$	YES
$\frac{A_s}{A_c + A_s} > 4\%$	$\frac{2463}{538,6 + 2463} = 4,4\%$	YES

Method not applicable

Verification of the composite column

Simplified method : Application field

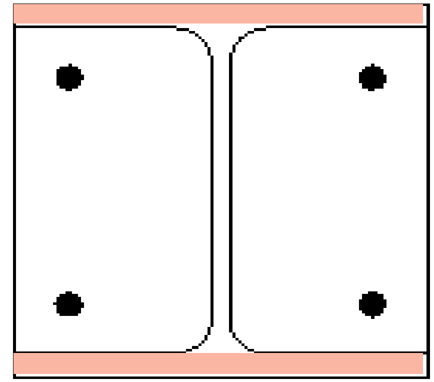
Allowed parameters R60	Existing parameters	Fulfilled conditions ?
$\ell_\theta \leq 13{,}5b = 13{,}5 \,.\, 0{,}26 = 3{,}51m$	$\ell_{\theta z} = \ell_{\theta y} = 0{,}7 \,.\, 3{,}4 = 2{,}38$ m	YES
$230mm \leq h \leq 1100mm$	h = 250 mm	YES
$230mm \leq b \leq 500$	b = 260 mm	YES
$1\% \leq A_s/(A_c + A_s) \leq 6\%$	24,63/(538,6+24,63)=4,4%	YES
max R120	R60	YES
ℓ_θ limited to 10b if $230 \leq b < 300$	Weak axis : $\ell_{\theta z} = 2{,}38 < 10b = 2{,}6$	YES
	Strong axis : $\ell_{\theta y} = 2{,}38 < 10b = 2{,}6$	YES



Method applicable

Verification of the composite column

Resistance to axial compression according to **weak axis**



Flanges of the profile

Mean temperature

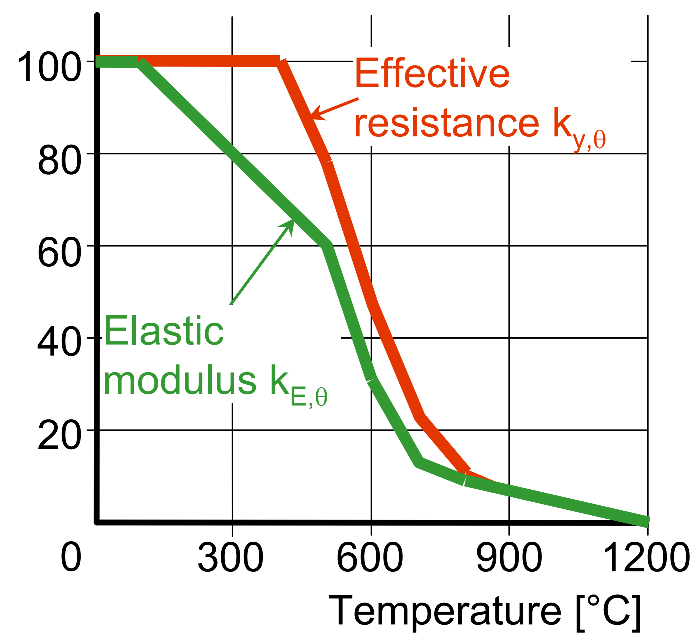
Standard fire resistance	$\theta_{o,t}$ [°C]	k_t [m°C]
R 30	550	9,65
R 60	680	9,55
R 90	805	6,15
R 120	900	4,65

$$\theta_{f,t} = \theta_{0,t} + k_t \cdot A_m / V$$

$$\text{with} \quad \frac{A_m}{V} = \frac{2(h+b)}{h.b} = 15{,}7\,m^{-1}$$

$$\theta_{f,t} = 680 + 9{,}55 \cdot 15{,}7 = 830°C \Rightarrow \begin{cases} k_{y,\theta} = 0{,}095 \\ k_{E,\theta} = 0{,}083 \end{cases}$$

% of the nominal value



Plastic resistance :

$$N_{fi,pl,Rd,f} = 2 \cdot (b \cdot e_f \cdot f_{ay,f} \cdot k_{y,\theta}) / \gamma_{M,fi,a}$$

$$N_{fi,pl,Rd,f} = 2 \cdot (260 \cdot 12{,}5 \cdot 460 \cdot 0{,}095)/1{,}0 = 284{,}3\,kN$$

Effective stiffness :

$$(EI)_{fi,f,z} = E_{a,f} \cdot k_{E,\theta} \cdot (e_f \cdot b^3)/6$$

$$(EI)_{fi,f,z} = 210000 \cdot 0{,}083 \cdot (12{,}5 \cdot 260^3)/6 = 640{,}4\,kN.m^2$$

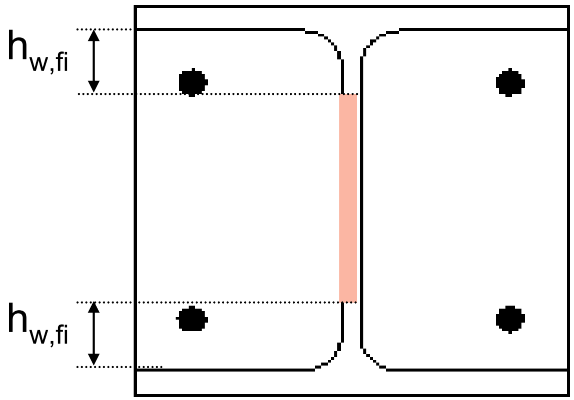
Verification of the composite column

Web of the profile

Reduced height of web :

$$h_{w,fi} = 0{,}5 \cdot \left(h - 2 \cdot e_f\right) \cdot \left(1 - \sqrt{1 - 0{,}16 \cdot (H_t / h)}\right)$$

$$h_{w,fi} = 0{,}5 \cdot (250 - 2 \cdot 12{,}5)\left(1 - \sqrt{1 - 0{,}16(770/250)}\right) = 32{,}4\,\text{mm}$$



Standard fire resistance	H_t [mm]
R 30	350
R 60	770
R 90	1100
R 120	1250

Level of maximum stress : $f_{ay,w,t} = f_{ay,w}\sqrt{1 - (0{,}16 H_t / h)}$

$$f_{ay,w,t} = 460\sqrt{1 - (0{,}16 \cdot 770\ /250)} = 327{,}6\,\text{MPa}$$

Plastic resistance : $N_{fi,pl,Rd,w} = \left[e_w\left(h - 2e_f - 2h_{w,fi}\right)f_{ay,w,t}\right]/\gamma_{M,fi,a}$

$$N_{fi,pl,Rd,w} = \left(7{,}5(250 - 2 \cdot 12{,}5 - 2 \cdot 32{,}4)327{,}6\right)/1{,}0 = 393{,}7\,\text{kN}$$

Effective stiffness : $(EI)_{fi,w,z} = \left[E_{a,w}\left(h - 2e_f - 2h_{w,fi}\right)e_w^3\right]/12$

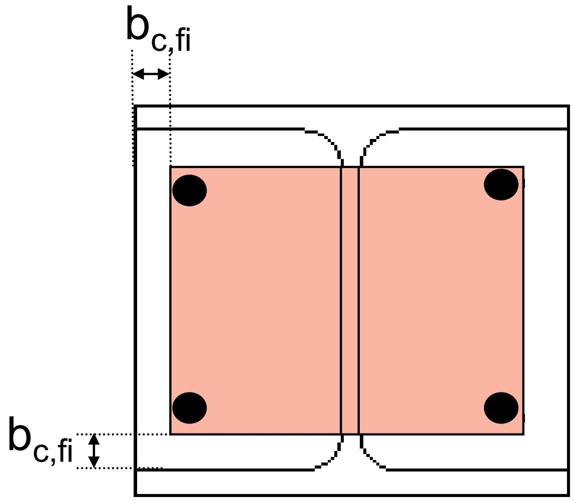
$$(EI)_{fi,w,z} = \left(210000\left(250 - 2 \cdot 12{,}5 - 2 \cdot 32{,}4\right)7{,}5^3\right)/12 = 1{,}18\,\text{kN.m}^2$$

Verification of the composite column

Concrete

Reduced thickness of concrete : $b_{c,fi}$

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

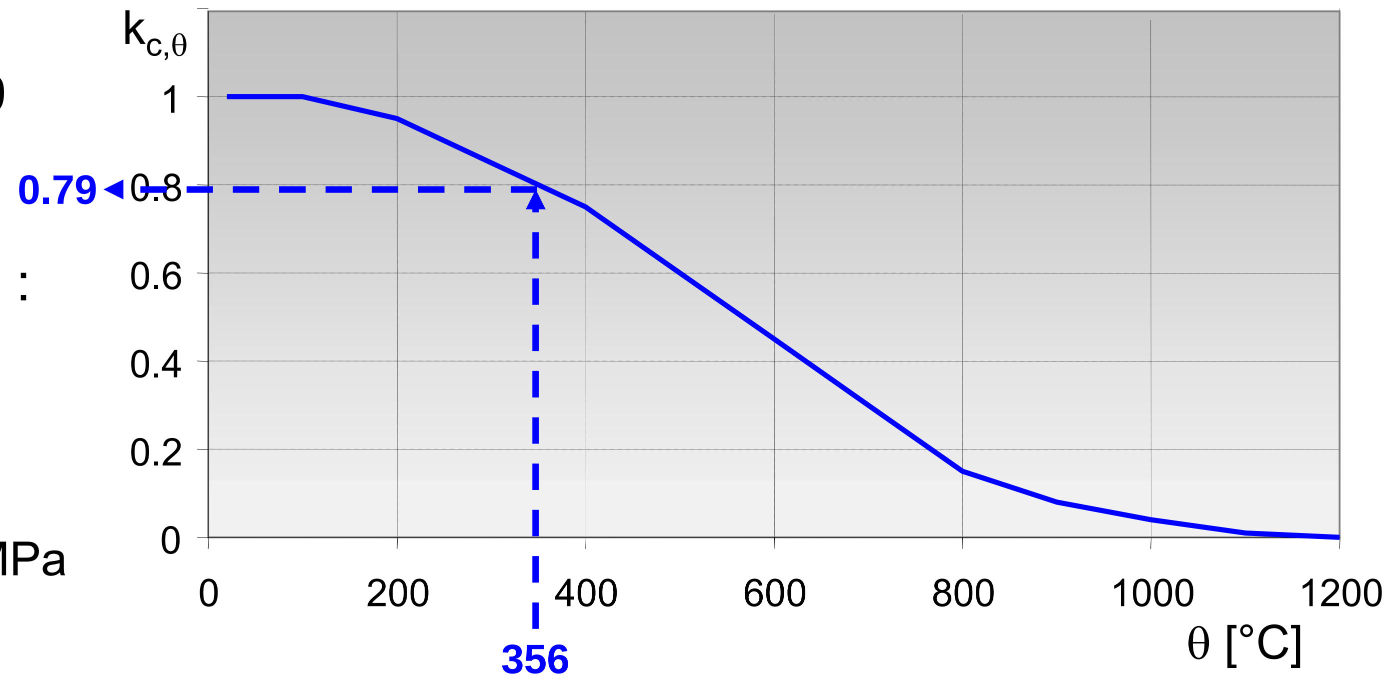


R30		R60		R90		R120	
A_m/V [m^{-1}]	$\theta_{c,t}$ [°C]	A_m/V [m^{-1}]	$\theta_{c,t}$ [°C]	A_m/V [m^{-1}]	$\theta_{c,t}$ [°C]	A_m/V [m^{-1}]	$\theta_{c,t}$ [°C]
4	136	4	214	4	256	4	265
23	300	9	300	6	300	5	300
46	400	21	400	13	400	9	400
---	---	50	600	33	600	23	600
---	---	---	---	54	800	38	800
---	---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

$A_m/V = 15,7 m^{-1}$ ⇨ Average temperature in the concrete $\theta_{c,t} = 356°C$

Verification of the composite column

$\theta_{c,t} = 356°C \Rightarrow k_{c,\theta}=0{,}79$



Secant modulus of concrete :

$$E_{c,sec,\theta} = \frac{f_{c,\theta}}{\varepsilon_{cu,\theta}} = \frac{f_c . k_{c,\theta}}{\varepsilon_{cu,\theta}}$$

$$E_{c,sec,\theta} = \frac{30 \cdot 0{,}79}{8{,}68 \cdot 10^{-3}} = 2746{,}4\,\text{MPa}$$

Plastic resistance :

$$N_{fi,pl,Rd,c} = 0{,}86\{(h - 2e_f - 2b_{c,fi})(b - e_w - 2b_{c,fi}) - A_s\} f_{c,\theta}/\gamma_{M,fi,c}$$

$$N_{fi,pl,Rd,c} = 0{,}86 \cdot \{(250 - 2 \cdot 12{,}5 - 2 \cdot 15)(260 - 7{,}5 - 2 \cdot 15) - 2463\} \cdot 25 \cdot 0{,}79/1{,}0 = 839\text{ kN}$$

Effective stiffness :

$$(EI)_{fi,c,z} = E_{c,sec,\theta}\left[\{(h - 2e_f - 2b_{c,fi})((b - 2b_{c,fi})^3 - e_w^3)/12\} - I_{s,z}\right]$$

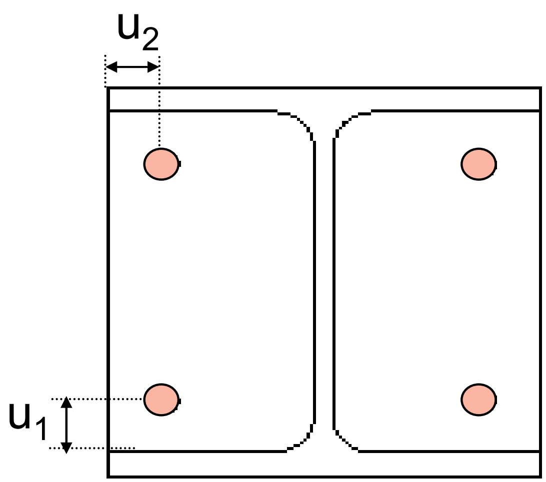
$$(EI)_{fi,c,z} = 2746{,}4\left[\{(250 - 2 \cdot 12{,}5 - 2 \cdot 15)((260 - 2 \cdot 15)^3 - 7{,}5^3)/12\} - 1324{,}6 \cdot 10^4\right] = 509{,}5\,\text{kN} \cdot \text{m}^2$$

Verification of the composite column

Reinforcing bars

Reduction factor for yield strength : $k_{y,t}$					
Standard fire resistance	***u* [mm]**				
	40	**45**	**50**	**55**	**60**
R30	1	1	1	1	1
R60	0,789	0,883	0,9763	1	1
R90	0,314	0,434	0,572	0,696	0,822
R120	0,170	0,223	0,288	0,367	0,436

Reduction factor for Young modulus : $k_{E,t}$					
Standard fire resistance	***u* [mm]**				
	40	**45**	**50**	**55**	**60**
R30	0,830	0,865	0,88	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



$$u = \sqrt{u_1 \cdot u_2} = \sqrt{52 \cdot 60} = 55{,}86\,\text{mm}$$

Plastic resistance : $N_{fi,pl,Rd,s} = A_s \cdot k_{y,t} \cdot f_{s,y} / \gamma_{M,fi,s}$

$$N_{fi,pl,Rd,s} = 2463 \cdot 1{,}0 \cdot 500 / 1{,}0 = 1231{,}5\,\text{kN}$$

Effective stiffness : $(EI)_{fi,s,z} = k_{E,t} \cdot E_s \cdot I_{s,z}$

$$(EI)_{fi,s,z} = 0{,}735 \cdot 210000 \cdot 1218{,}9 \cdot 10^4 = 1881{,}4\ \text{kN.m}^2$$

Verification of the composite column

Plastic resistance of the composite section

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

$$N_{fi,pl,Rd} = 284{,}3 + 393{,}7 + 839 + 1231{,}5 = 2748\,kN$$

Effective stiffness of the composite section

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

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

$$(EI)_{fi,eff,z} = 0{,}9 \cdot 640{,}4 + 1{,}0 \cdot 1{,}18 + 0{,}8 \cdot 509{,}5 + 0{,}9 \cdot 1881{,}4 = 2678{,}4\ kN.m^2$$

Verification of the composite column

Determination of the axial buckling load at elevated temperatures

Euler buckling load :

$$N_{fi,cr,z} = \frac{\pi^2 \cdot (EI)_{fi,eff,z}}{\ell_{\theta z}^2} \qquad \text{with } \ell_{\theta z} = 2{,}38\,\text{m}$$

$$N_{fi,cr,z} = \frac{\pi^2 \cdot 2678{,}4}{2{,}38^2} = 4667\,\text{kN}$$

Slenderness ratio :

$$\bar{\lambda}_\theta = \sqrt{\frac{N_{fi,pl,R}}{N_{fi,cr,z}}} = \sqrt{\frac{2748}{4667}} = 0{,}767 \quad \Rightarrow \quad \text{c curve} \rightarrow \chi_z = 0{,}683$$

Axial buckling resistance :

$$N_{fi,Rd,z} = \chi_z \cdot N_{fi,pl,Rd}$$

$$N_{fi,Rd,z} = 0{,}683 \cdot 2748 = 1876\,\text{kN} > N_{fi,Rd} = 1726\,\text{kN}$$

Verification of the composite column

Resistance to axial compression according to **strong axis**

Same method as for weak axis, excepted of the inertia !

Plastic resistance of the composite section

$$N_{fi,pl,Rd} = N_{fi,pl,Rd,f} + N_{fi,pl,Rd,w} + N_{fi,pl,Rd,c} + N_{fi,pl,Rd,s} = 2748\,kN$$

Effective stiffness of the composite section

$$(EI)_{fi,eff,y} = \varphi_{f,\theta}(EI)_{fi,f,y} + \varphi_{w,\theta}(EI)_{fi,w,y} + \varphi_{c,\theta}(EI)_{fi,c,y} + \varphi_{s,\theta}(EI)_{fi,s,y} = 4097{,}1 kN.m^2$$

Euler buckling load

$$N_{fi,cr,y} = \frac{\pi^2 \cdot (EI)_{fi,eff,y}}{\ell_{\theta y}^2} = 7138{,}8\,kN$$

Slenderness ratio

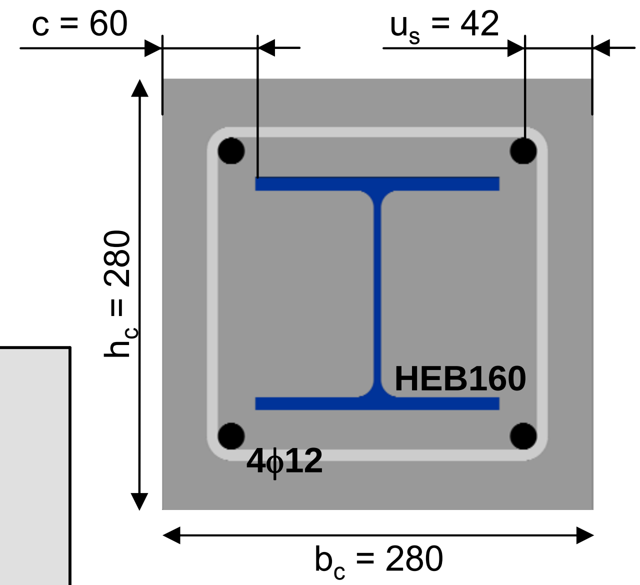
$$\bar{\lambda}_\theta = \sqrt{\frac{N_{fi,pl,R}}{N_{fi,cr,y}}} = 0{,}62$$

Axial buckling resistance

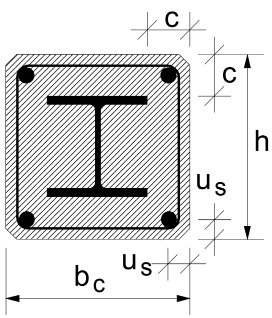
$$N_{fi,Rd,y} = \chi_y . N_{fi,pl,Rd} = 2124{,}8\,kN > N_{fi,Rd} = 1726 kN$$

Verification of the composite column

A3C calculation (Alternative solution : Fully encased HEB160)



Tabulated data



		Standard Fire Resistance					
		R30	R60	R90	R120	R180	R240
1.1	Minimum dimensions h_c and b_c [mm]	150	180	220	300	350	400
1.2	minimum concrete cover of steel section c [mm]	40	50	50	75	75	75
1.3	minimum axis distance of reinforcing bars u_s [mm]	20*	30	30	40	50	50
	or						
2.1	Minimum dimensions h_c and b_c [mm]	-	200	250	350	400	-
2.2	minimum concrete cover of steel section c [mm]	-	40	40	50	60	-
2.3	minimum axis distance of reinforcing bars u_s [mm]	-	20*	20*	30	40	-

Table 4.4: Minimum cross-sectional dimensions, minimum concrete cover of the steel section and minimum axis distance of the reinforcing bars, of composite columns made of totally encased steel sections.