

# PART 5-7: Composite beam comprising steel beam with partial concrete encasement

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## 1 TASK

A fire safety verification has to be done for a composite beam of a storehouse. It is a simply supported beam with a uniform load and has a span of 12.0 m. The steel beam is partially encased and the slab is a composite slab with a re-entrant steel sheet. The required standard fire resistance class for the beam is R 90.

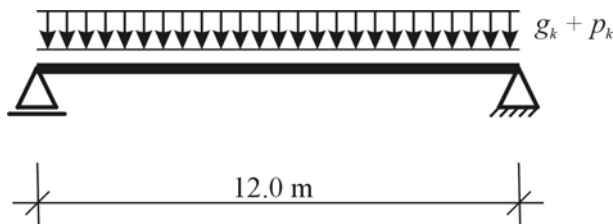


Figure 1. Static system

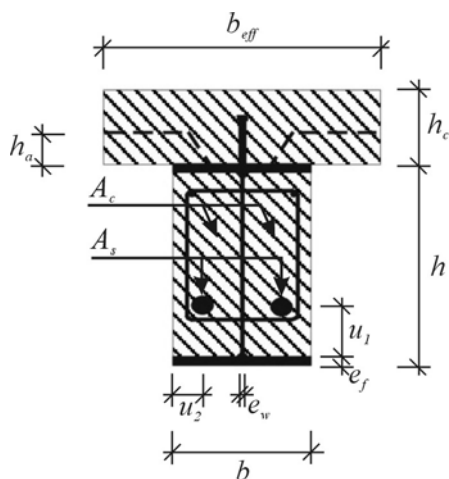


Figure 2. Cross-section

Material properties:

Beam:

Profile:	rolled section IPE 500
Steel grade:	S 355
Height:	$h = 500$ mm
Width:	$b = 200$ mm
Thickness of web:	$e_w = 10.2$ mm
Thickness of flange:	$e_f = 16$ mm
Cross-sectional area:	$A_a = 11,600$ mm <sup>2</sup>
Yield stress:	$f_{y,a} = 355$ N/mm <sup>2</sup>

Slab:

Strength category:	C 25/30
Height:	$h_c = 160$ mm
Effective width:	$b_{eff} = 3000$ mm
Compression strength:	$f_c = 25$ N/mm <sup>2</sup>

Profiled steel sheet:

Type:	re-entrant
Height:	$h_a = 51$ mm

Reinforcement in partial concrete encasement:

Steel grade:	S 500
Diameter:	2 Ø 30
Cross-sectional area:	$A_s = 1410$ mm <sup>2</sup>
Axis distances:	$u_l = 110$ mm
	$u_{s,l} = 60$ mm
Yield stress:	$f_{y,s} = 500$ N/mm <sup>2</sup>

Concrete between flanges:

Strength category:	C 25/30
Width:	$b_c = 200$ mm
Compression strength:	$f_c = 25$ N/mm <sup>2</sup>

Loads:

Permanent loads:

Self-weight:	$g_{s,k} = 15.0$ kN/m
Finishing load:	$g_{f,k} = 6.0$ kN/m

Variable loads:

Live load:	$p_k = 30.0$ kN/m
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## 2 FIRE RESISTANCE OF A COMPOSITE BEAM COMPRISING STEEL BEAM WITH PARTIAL CONCRETE ENCASEMENT

### 2.1 Mechanical actions during fire exposure

Actions on structures in fire situation are classified as an accidental situation:

$$E_{dA} = E \left( \sum G_k + A_d + \sum \psi_{2,i} \cdot Q_{k,i} \right)$$

The combination factor for the leading variable action and for a storehouse is  $\psi_{2,1} = 0.8$ .

With these parameters, the design bending moment during fire exposure can be calculated:

$$M_{f,d} = \left( (15.0 + 6.0) + 0.8 \cdot (30.0) \right) \cdot \frac{12.0^2}{8} = 810.0 \text{ kNm}$$

EN 1991-1-2

Section 4.3

2.2 Verification using simple calculation model

The composite beam is verified by the simple calculation model. It is accomplished according to EN 1994 Part 1-2, Section 4.3.4.3 and Annex F.

To use this model, the slab should have a minimum thickness  $h_c$ . Additionally the steel beam should have a minimum height  $h$ , a minimum width  $b_c$  (where  $b_c$  is the minimum width of steel beam or concrete encasement) and a minimum area  $h \cdot b_c$  (see Table 1).

Table 1. Minimum dimensions for the use of the simple calculation model for composite beams comprising steel beams with partial concrete encasement (see EN 1994 Part 1-2, Section 4.3.4.3, Table 4.8)

Standard fire resistance	Minimum slab thickness $h_c$ [mm]
R 30	60
R 60	80
R 90	100
R 120	120
R 180	150

$h_c = 160 \text{ mm} > \min h_c = 100 \text{ mm} \quad \checkmark$

In the calculation model of Annex F, the cross section is divided into different parts. At some parts the yield stress at other parts the cross-sectional area is reduced.

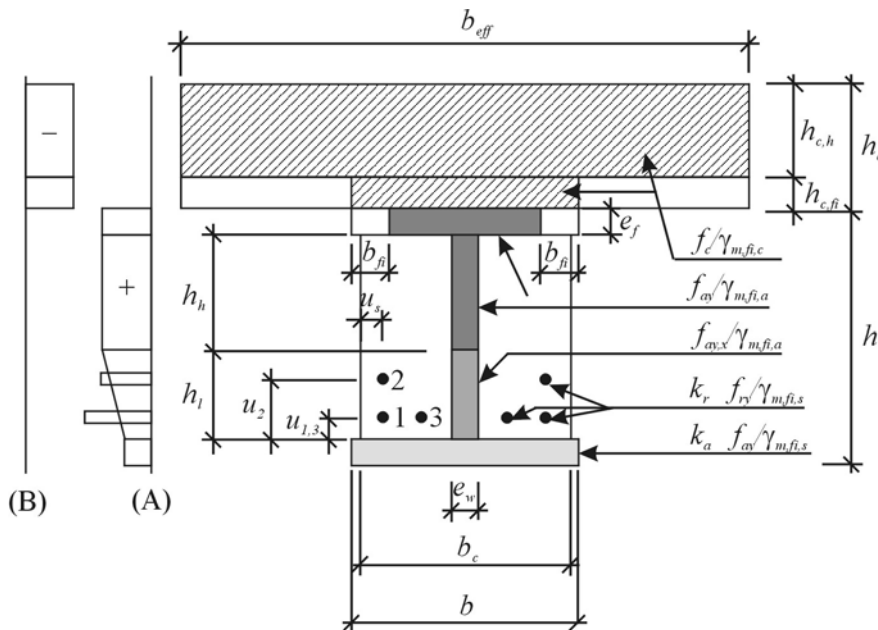


Figure 3. Reduced cross-section for the calculation of the plastic moment resistance and stress distributions in steel (A) and concrete (B)

The heating of the concrete slab is considered by reducing the cross-sectional area. For the different fire resistance classes, the thickness reduction  $h_{c,fi}$  is given in Table 2. For composite slabs made with re-entrant steel sheets, a minimum thickness reduction  $h_{c,fi,min}$  has to be considered. This minimum thickness reduction is equal to the height of the steel sheet (see EN 1994 Part 1-2, Annex F, Figure F.2).

$h_{c,fi} = 30 \text{ mm}$

$$h_{c,fi,min} = 51 \text{ mm}$$

For this, the height of the concrete during fire exposure  $h_{c,h}$  is:

$$h_{c,h} = 160 - 51 = 109 \text{ mm}$$

Table 2. Thickness reduction  $h_{c,fi}$  of the concrete slab (see EN 1994 Part 1-2, Annex F, Table F.1)

Standard fire resistance	Slab reduction $h_{c,fi}$ [mm]
R 30	10
R 60	20
R 90	30
R 120	40
R 180	55

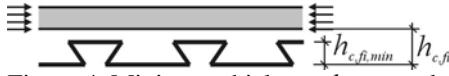


Figure 4. Minimum thickness  $h_{c,fi,min}$  reduction for re-entrant profiles

The heating of the upper flange of the steel beam is considered by reducing its cross-sectional area. The calculation of the width reduction  $b_{fi}$  is shown in Table 3.

$$b_{fi} = (16.0/2) + 30 + (200 - 200)/2 = 38.0 \text{ mm}$$

The effective width is calculated to:

$$b_{fi,u} = 200 - 2 \cdot 38 = 124.0 \text{ mm}$$

Table 3. Width reduction  $b_{fi}$  of the upper flange (see prEN 1994 Part 1-2, Annex F, Table F.2)

Standard fire resistance	Width reduction $b_{fi}$ [mm]
R 30	$(e_f/2) + (b - b_c)/2$
R 60	$(e_f/2) + 10 + (b - b_c)/2$
R 90	$(e_f/2) + 30 + (b - b_c)/2$
R 120	$(e_f/2) + 40 + (b - b_c)/2$
R 180	$(e_f/2) + 60 + (b - b_c)/2$

The web of the steel beam is divided into two parts. The upper part of the web possesses the full yield stress, where the yield stress of the lower part has a linear gradient, from the yield stress of the upper part to the reduced yield stress of the lower flange. The height of the lower part of the web  $h_l$  is calculated to:

$$h_l = \frac{a_1}{b_c} + \frac{a_2 \cdot e_w}{b_c \cdot h} > h_{l,min}$$

The parameters  $a_1$  and  $a_2$ , as well as the minimum height  $h_{l,min}$ , are given in Table 4 for  $h/b_c > 2$ .

$$h/b_c = 500 \text{ mm} / 200 \text{ mm} = 2.5$$

$$h_l = \frac{14,000}{200} + \frac{75,000 \cdot 10.2}{200 \cdot 500} = 77.7 \text{ mm} > 40 \text{ mm}$$

Table 4. Parameters  $a_1$ ,  $a_2$  and minimum height  $h_{l,min}$  for  $h/b_c > 2$  (see EN 1994 Part 1-2, Annex F, Table F.3)

Standard fire resistance	$a_1$ [mm <sup>2</sup> ]	$a_2$ [mm <sup>2</sup> ]	$h_{l,min}$ [mm]
R 30	3600	0	20
R 60	9500	0	30
R 90	14,000	75,000	40
R 120	23,000	110,000	45
R 180	35,000	250,000	55

The lower flange is not reduced by its cross-sectional area. Here, the yield stress is reduced by the factor  $k_a$ . This factor is limited by a minimum and maximum value. These limits, as well as the calculation of  $k_a$ , are given in Table 5.

$$a_0 = 0.018 \cdot e_f + 0.7 = 0.018 \cdot 16.0 + 0.7 = 0.988$$

$$k_a = \left( 0.12 - \frac{17}{200} + \frac{500}{38 \cdot 200} \right) \cdot 0.988 = 0.100 \begin{cases} > 0.06 \\ < 0.12 \end{cases}$$

Table 5. Reduction factor  $k_a$  of the yield stress of the lower flange (see EN 1994 Part 1-2, Annex F, Table F.4)

Standard fire resistance	Reduction factor $k_a$	$k_{a,min}$	$k_{a,max}$
R 30	$\left( 1.12 - \frac{84}{b_c} + \frac{h}{22 \cdot b_c} \right) \cdot a_0$	0.5	0.8
R 60	$\left( 0.21 - \frac{26}{b_c} + \frac{h}{24 \cdot b_c} \right) \cdot a_0$	0.12	0.4
R 90	$\left( 0.12 - \frac{17}{b_c} + \frac{h}{38 \cdot b_c} \right) \cdot a_0$	0.06	0.12
R 120	$\left( 0.1 - \frac{15}{b_c} + \frac{h}{40 \cdot b_c} \right) \cdot a_0$	0.05	0.1
R 180	$\left( 0.03 - \frac{3}{b_c} + \frac{h}{50 \cdot b_c} \right) \cdot a_0$	0.03	0.06

The heating of the reinforcing bars in the partial concrete encasement is considered by reducing the yield stress. The reduction factor depends on the fire resistance class and the position of the reinforcing bars. Like the reduction factor  $k_a$ , the reduction factor  $k_r$  has an upper and lower limit.

$$A_m = 2 \cdot h + b_c = 2 \cdot 500 + 200 = 1200 \text{ mm}$$

$$V = h \cdot b_c = 500 \cdot 200 = 100,000 \text{ mm}^2$$

$$u = \frac{1}{\left(\frac{1}{u_i}\right) + \left(\frac{1}{u_{si}\right) + \frac{1}{(b_c - e_w - u_{si})}}$$

$$= \frac{1}{\left(\frac{1}{110}\right) + \left(\frac{1}{60}\right) + \frac{1}{(200 - 10.2 - 60)}}$$

$$= 29.88 \text{ mm}$$

$$k_r = \frac{(u \cdot a_3 + a_4) \cdot a_5}{\sqrt{A_m/V}} = \frac{(29.88 \cdot 0.026 - 0.154) \cdot 0.09}{\sqrt{1200/100,000}} = 0.51 \begin{cases} > 0.1 \\ < 1.0 \end{cases}$$

Table 6. Parameters for calculation of  $k_r$  (see EN 1994 Part 1-2, Annex F, Table F.5)

Standard fire resistance	$a_3$	$a_4$	$a_5$	$k_{r,min}$	$k_{r,max}$
R 30	0.062	0.16	0.126	0.1	1.0
R 60	0.034	-0.04	0.101	0.1	1.0
R 90	0.026	-0.154	0.090	0.1	1.0
R 120	0.026	-0.284	0.082	0.1	1.0
R 180	0.024	-0.562	0.076	0.1	1.0

To acquire the plastic moment resistance, the axial forces of the different parts should be determined.

Concrete:

$$C_c = b_{eff} \cdot h_{c,h} \cdot \alpha_c \cdot f_c = 300.0 \cdot 10.9 \cdot 0.85 \cdot 2.5 = 6948.8 \text{ kN}$$

Upper flange:

$$T_{f,u} = b_{fi,u} \cdot e_f \cdot f_y = 12.4 \cdot 1.60 \cdot 35.5 = 704.3 \text{ kN}$$

Upper web:

$$T_{w,u} = e_w \cdot h_h \cdot f_y = 1.02 \cdot 39.03 \cdot 35.5 = 1413.3 \text{ kN}$$

where:

$$h_h = h - 2 \cdot e_f - h_l = 50.0 - 2 \cdot 1.6 - 7.77 = 39.03 \text{ cm}$$

Lower web:

$$T_{w,l} = e_w \cdot h_l \cdot \left(\frac{1+k_a}{2}\right) \cdot f_y = 1.02 \cdot 7.77 \cdot \left(\frac{1+0.1}{2}\right) \cdot 35.5 = 154.7 \text{ kN}$$

$$z_{w,l} = h_l \cdot \frac{2 \cdot k_a + 1}{3 \cdot (k_a + 1)} = 7.77 \cdot \frac{2 \cdot 0.1 + 1}{3 \cdot (0.1 + 1)} = 2.8 \text{ cm}$$

Lower flange:

$$T_{f,l} = b \cdot e_f \cdot k_a \cdot f_{y,a} = 20.0 \cdot 1.6 \cdot 0.1 \cdot 35.5 = 113.6 \text{ kN}$$

Reinforcement bars:

$$T_r = A_s \cdot k_r \cdot f_{y,s} = 14.1 \cdot 0.51 \cdot 50.0 = 359.6 \text{ kN}$$

Due to the fact that the compression force  $C_c$  is larger than the sum of the tension forces  $T_i$ , the plastic neutral axis is situated in the concrete slab. So the plastic neutral axis is calculated to:

$$z_{pl} = \frac{\sum T_i}{\alpha_c \cdot f_c \cdot b_{eff}} = \frac{704.3 + 1413.3 + 154.7 + 113.6 + 359.6}{0.85 \cdot 2.5 \cdot 300} = 4.31 \text{ cm}$$

To get the moment resistance, the lever arms of the forces are needed:  
Concrete slab (referring to upper edge of slab):

$$z_c = z_{pl} / 2 = 4.31 / 2 = 2.16 \text{ cm}$$

Upper flange (referring to centre of gravity of concrete slab):

$$z_{f,u} = h_c + e_f / 2 - z_c = 16.0 + 1.6 / 2 - 2.16 = 14.64 \text{ cm}$$

Upper web:

$$z_{w,u} = h_c + e_f + h_h / 2 - z_c = 16.0 + 1.6 + 39.03 / 2 - 2.16 = 34.96 \text{ cm}$$

Lower web:

$$z_{w,l} = h_c + e_f + h_h + z_{w,l} - z_c = 16 + 1.6 + 39.03 + 2.8 - 2.16 = 57.27 \text{ cm}$$

Lower flange:

$$z_{f,l} = h_c + h - e_f / 2 - z_c = 16.0 + 50.0 - 1.6 / 2 - 2.16 = 63.04 \text{ cm}$$

Reinforcement:

$$z_r = h_c + h - e_f - u_1 - z_c = 16.0 + 50.0 - 1.6 - 11.0 - 2.16 = 51.24 \text{ cm}$$

The plastic moment resistance is determined to:


$$\begin{aligned} M_{f_i,Rd} &= T_{f,u} \cdot z_{f,u} + T_{w,u} \cdot z_{w,u} + T_{w,l} \cdot z_{w,l} + T_{f,l} \cdot z_{f,l} + T_r \cdot z_r \\ &= 704.3 \cdot 14.64 + 1413.3 \cdot 34.96 + 154.7 \cdot 57.27 + 113.6 \cdot 63.04 \\ &\quad + 359.6 \cdot 51.24 \\ &= 10,311 + 49,409 + 8860 + 7161 + 18,426 \\ &= 94,167 \text{ kNcm} = 942.7 \text{ kNm} \end{aligned}$$

Verification:

$$\frac{810.0}{942.7} = 0.86 < 1 \quad \checkmark$$

## REFERENCES

- EN 1991, *Eurocode 1: Actions on structures – Part 1-2: General actions – Actions on structures exposed to fire*, Brussels: CEN, November 2002  
EN 1994, *Eurocode 4: Design of composite steel and concrete structures – Part 1-2: General Rules – Structural Fire Design*, Brussels: CEN, November 2006

<b>QUALITY RECORD</b>	WP5 		
Title	Example to EN 1994 Part 1-2: Composite beam		
Eurocode reference(s)	EN 1991-1-2:2005; EN 1993-1-2:2006; EN 1994-1-1:2004; EN 1994-1-2:2006		
<b>ORIGINAL DOCUMENT</b>			
	<b>Name</b>	<b>Company</b>	<b>Date</b>
Created by	P. Schaumann	Univ.of Hannover	24/11/2005
	T. Trautmann	Univ.of Hannover	24/11/2005
Technical content checked by	M. Haller	ArcelorMittal	24/11/2005
<b>TRANSLATED DOCUMENT</b>			
Translation made and checked by:	J. Chlouba	CTU in Prague	10/01/2008
Translated resource approved by:	Z. Sokol	CTU in Prague	25/01/2008
National technical contact:	F. Wald	CTU in Prague	