PART 5-6: Composite beam

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1 TASK

A fire safety verification has to be done for a composite beam of an office building. It is a simply supported beam and is loaded uniformly. The concrete slab of the composite beam protects the beam from the top in the fire situation, so the steel beam is exposed to fire on three sides. For fire protection of the steel beam a contour encasement of plaster is chosen. The required standard fire resistance class for the beam is R 60.

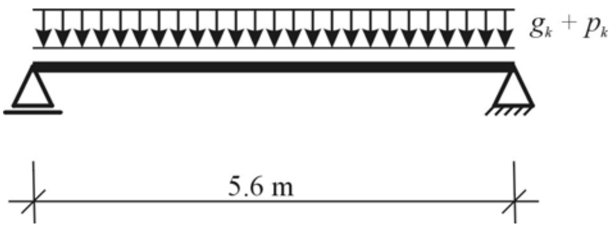


Figure 1. Static system

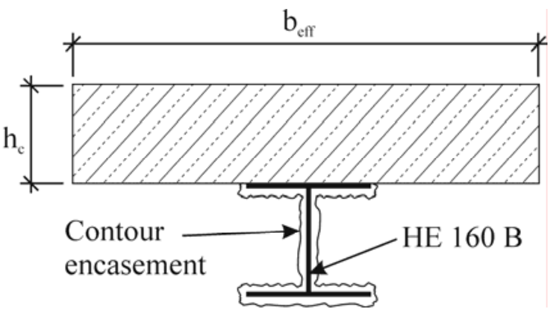


Figure 2. Cross-section

Material properties:

Beam:

Profile:	rolled section HE 160 B
Steel grade:	S 355
Height:	h = 160 mm
Height of web:	h_w = 134 mm
Width:	$b = b_1 = b_2 = 160$ mm

Thickness of web: e_w = 8 mm
Thickness of flange: $e_f = e_1 = e_2 = 13$ mm
Cross-sectional area: A_a = 5430 mm²
Yield stress: $f_{y,a}$ = 355 N/mm²

Slab:
Strength category: C 25/30
Height: h_c = 160 mm
Effective width: b_{eff} = 1400 mm
Compression strength: f_c = 25 N/mm²
Elastic modulus: E_{cm} = 29,000 N/mm²

Shear connectors:
Quantity: n = 34 (equidistant)
Diameter: d = 22 mm
Tensile strength: f_u = 500 N/mm²

Encasement:
Material: plaster
Thickness: d_p = 15 mm (contour encasement)
Thermal conductivity: λ_p = 0.12 W/(m·K)
Specific heat: c_p = 1100 J/(kg·K)
Density: ρ_p = 550 kg/m³

Loads:
Permanent loads:
Self weight: g_k = 20.5 kN/m
Finishing load: g_k = 7.5 kN/m
Variable loads:
Live load: p_k = 15.0 kN/m

2 FIRE RESISTANCE OF A COMPOSITE BEAM

2.1 *Mechanical actions during fire exposure*

EN 1991-1-2

Actions on structures from fire exposure are classified as accidental situation:

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

Section 4.3

The partial safety factor γ_{GA} for the accidental situation is γ_{GA} = 1.0. The combination factor for the leading variable action for office buildings is set to $\psi_{2,1}$ = 0.3.

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

$$M_{fi,d} = (20.5 + 7.5) + 0.3 \cdot (15.0) \cdot \frac{5.6^2}{8} = 127.4 \text{ kNm}$$

2.2 *Calculation of the temperatures in the cross-section*

EN 1994-1-2

For the calculation of the temperatures, the cross-section is split into different sections. These are the concrete slab, the upper flange, the web and the lower flange. It is done according to Section 4.3.4.2 of EN 1994-1-2.

The temperatures of the upper flange, the web and the lower flange are determined by using the Euro-Nomogram (“Euro-Nomogram”, ECCS No.89, 1996). Therefore, the section factors of these parts are required.

Lower flange:

$$\left(\frac{A_p}{V}\right)_l = \frac{2\cdot\left(b_1+e_1\right)}{b_1\cdot e_1} = \frac{2\cdot\left(0.16+0.013\right)}{0.16\cdot 0.013} = 166.3\ \text{m}^{-1}$$

Section 4.3.4.2

Web:

$$\left(\frac{A_p}{V}\right)_w = \frac{2\cdot\left(h_w\right)}{h_w\cdot e_w} = \frac{2\cdot\left(0.134\right)}{0.134\cdot 0.008} = 250.0\ \text{m}^{-1}$$

Upper flange (more than 85% of the upper flange is in contact with the concrete slab):

$$\left(\frac{A_p}{V}\right)_u = \frac{\left(b_2+2\cdot e_2\right)}{b_2\cdot e_2} = \frac{\left(0.16+2\cdot 0.013\right)}{0.16\cdot 0.013} = 89.4\ \text{m}^{-1}$$

The temperatures are determined to:

ECCS No.89

Table 1. Temperatures of upper flange, web and lower flange

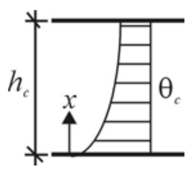
	$\left(\frac{A_p}{V}\right)_i\cdot\frac{\lambda_p}{d_p}$ $\left[\frac{\text{W}}{\text{m}^3\text{K}}\right]$	$\theta_{a,max,60}$ [°C]
Upper flange	715	390
Web	2000	650
Lower flange	1330	550

The temperature of the concrete slab is not constant over its thickness. Therefore the compression strength varies over the thickness. For temperatures lower than 250 °C, the compression strength is not reduced. Above 250 °C it is reduced by the reduction factor $k_{c,\theta}$. Assessment of the temperatures may be done in layers of 10 mm thickness on basis of Table 2.

EN 1994-1-2
Section D.3

Table 2. Temperature distribution in a solid slab of 100 mm thickness composed of normal weight concrete and not insulated (see EN 1994-1-2, Annex D.3, Table D.5)

Depth x [mm]	Temperature q_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



2.3 *Verification using simple calculation model*

The composite beam is verified by the simple calculation model. It is done in the strength domain. The calculation of the moment resistance is accomplished according to Annex E.

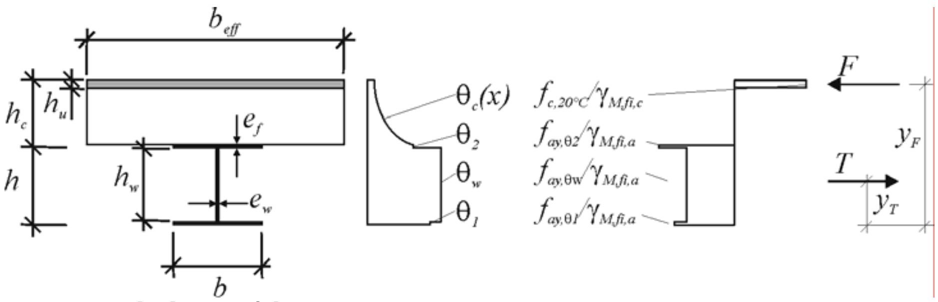


Figure 3. Calculation of the moment resistance

The temperatures of the parts of the steel beam were determined in Section 3.2. The reduction factors $k_{y,\theta,i}$ for the calculation of the yield stresses at elevated temperatures, are given in Table 3.2 of EN 1994-1-2, Section 3.2.1.

Table 3. Calculation of the reduced yield stresses

	$\theta_{a,max,60}$ [°C]	$k_{y,\theta}$ [-]	$f_{ay,\theta}$ [kN/cm²]
Upper flange	390	1.00	35.5
Web	650	$(0.47+0.23)/2=0.35$	12.4
Lower flange	550	$(0.78+0.47)/2=0.625$	22.2

Section E.1

The next step is the calculation of the tensile force T of the steel beam according to Figure 3.

$$T=\frac{f_{ay,\theta 1}\cdot\left(b\cdot e_f\right)+f_{ay,\theta w}\cdot\left(h_w\cdot e_w\right)+f_{ay,\theta 2}\cdot\left(b\cdot e_f\right)}{\gamma_{M,fi,a}}$$

$$=\frac{22.2\cdot(16\cdot 1.3)+12.4\cdot(13.4\cdot 0.8)+35.5\cdot(16\cdot 1.3)}{1.0}$$

$$=1333.1\text{ kN}$$

The location of the tensile force is determined to:

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

$$=\frac{22.2\cdot\left(16\cdot\frac{1.3^2}{2}\right)+12.4\cdot(13.4\cdot 0.8)\cdot\left(1.3+\frac{13.4}{2}\right)+35.5\cdot(16\cdot 1.3)\cdot\left(16-\frac{1.3}{2}\right)}{1333.1\cdot 1.0}$$

$$=9.53\text{ cm}$$

In a simply supported beam, the value of the tensile force T is limited by:

$$T \le N \cdot P_{fi,Rd}$$

where:

N Number of shear connectors in one of the critical lengths of the beam

$P_{fi,Rd}$ Design resistance in the fire situation of a shear connector

To get $P_{fi,Rd}$,, the reduction factors $k_{u,\theta}$ and $k_{c,\theta}$ (Table 5) as well as the design resistances of a shear connector at ambient temperatures $P_{Rd,1}$ and $P_{Rd,2}$ are needed.

The temperatures for getting the reduction factors are determined as 80 % (stud connector) and 40 % (concrete) of the steel flange (see EN 1994 Part 1-2, Section 4.3.4.2.5 (2)). The reduction factor for the tensile strength of the stud connector is given it Table 3.2 of EN 1994-1-2, Section 3.2.1. The effect of strain hardening ($k_{u,\theta}$ >1) should only be accounted, if it is proven that local failures (i.e. local buckling, shear failure, concrete spalling, etc) do not occur. So in this case the strain hardening is not considered. The reduction factor for the compression strength of the concrete is given in Table 3.3 of EN 1994-1-2, Section 3.2.1.

$$\theta_v = 0.8 \cdot 390 = 312 \text{ °C}$$

$$\Rightarrow \quad k_{u,\theta} = 1.0$$

$$\theta_c = 0.4 \cdot 390 = 156 \text{ °C}$$

$$\Rightarrow \quad k_{c,\theta} = 0.98$$

The design resistances of the shear connector are calculated according to EN 1994-1-1, with the partial safety factor $\gamma_{M,fi,v}$ replacing γ_v. $\boxed{\text{EN 1994-1-1}}$

$$P_{Rd,1} = 0.8 \cdot \frac{f_u}{\gamma_{M,fi,v}} \cdot \frac{\pi \cdot d^2}{4} = 0.8 \cdot \frac{50.0}{1.0} \cdot \frac{\pi \cdot 2.2^2}{4} = 152 \text{ kN}$$ Section 6.6.3.1

$$P_{Rd,2} = 0.29 \cdot \alpha \cdot d^2 \cdot \frac{\sqrt{f_c \cdot E_{cm}}}{\gamma_{M,fi,v}} = 0.29 \cdot 1.0 \cdot 2.2^2 \cdot \frac{\sqrt{2.5 \cdot 2900}}{1.0} = 120 \text{ kN}$$

The design resistance in the fire situation of a shear connector is: $\boxed{\text{EN 1994-1-2}}$

$$P_{fi,Rd} = \min \begin{cases} P_{fi,Rd,1} = 0.8 \cdot k_{u,\theta} \cdot P_{Rd,1} = 0.8 \cdot 1.0 \cdot 152 = 121.6 \text{ kN} \\ P_{fi,Rd,2} = k_{c,\theta} \cdot P_{Rd,2} = 0.98 \cdot 120 = 117.6 \text{ kN} \quad \leftarrow \text{ relevant} \end{cases}$$ Section 4.3.4.2

So, the limitation can be verified:

$$1333.1 \text{ kN} < 34/2 \cdot 117.6 = 1999.2 \text{ kN}$$ Section E.1

For equilibrium of forces, the compression force has to be equal to the tension force. Therefore the thickness of the compressive zone h_u is determined to:

$$h_u = \frac{T}{b_{eff} \cdot f_c / \gamma_{M,fi,c}} = \frac{1333.1}{140.0 \cdot 2.5/1.0} = 3.8 \text{ cm}$$

Now, two situations may occur. The first one is that the temperature in every layer of the concrete in the compression zone is lower than 250 °C. In the second situation the temperature of some layers of the concrete is above 250 °C. To check which situation occurs, following calculation has to be done:

$$\left(h_c - h_u\right) = 16 - 3.8 = 12.2 \text{ cm}$$

If the result of this equation is greater than the depth x according to Table 2, corresponding to a concrete temperature below 250 °C, the concrete in the compression zone may not be reduced.

$$h_{cr} = x = 5.0 \text{ cm} < 12.2 \text{ cm}$$

The point of application of the compression force y_F is determined to:

$$y_F = h + h_c - (h_u/2) = 16 + 16 - (3.8/2) = 30.1 \text{ cm}$$

The moment resistance is calculated to:

$$M_{fi,Rd} = T \cdot (y_F - y_T) = 1333.1 \cdot (30.1 - 9.53) \cdot 10^{-2} = 274.2 \text{ kNm}$$

Verification:

$$127.4/274.2 = 0.46 < 1 \qquad \checkmark$$

REFERENCES

ECCS No.89, *Euro-Nomogram*, Brussels: ECCS – Technical Committee 3 – Fire Safety of Steel Structures, 1995

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 1-1: General Rules and rules for buildings*, Brussels: CEN, December 2004

EN 1994, Eurocode *4: Design of composite steel and concrete structures – Part 1-2: General Rules – Structural Fire Design*, Brussels: CEN, November 2006

QUALITY RECORD	WP5 DIFISEK		
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