PART 5-8: Composite column with partially encased steel sections

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1 TASK

The following example deals with a composite column made of partially encased steel sections. It is part of an office building and has a length of L = 4.0 m. In this example, the simple calculation model and the “tabulated data” method are used. The column is part of a braced frame and is connected bending resistant to the upper and lower column. Therefore the buckling length can be reduced as seen in Figure 1. The required standard fire resistance class for the column is R 60.

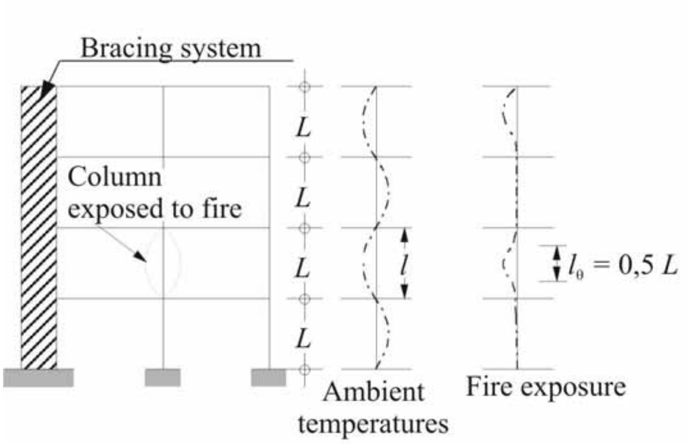

Figure 1. Buckling lengths of columns in braced frames

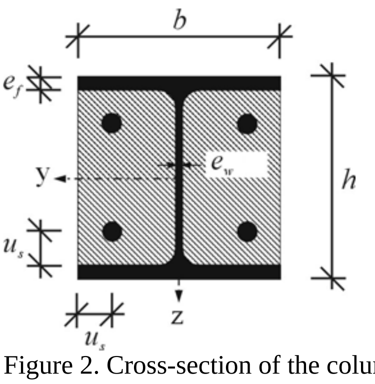

Figure 2. Cross-section of the column

Material properties:

Steel column:

Profile:	rolled section HE 300 B	
Steel grade:	S 235	
Height:	h =	300 mm
Width:	b =	300 mm
Thickness of web:	e_w =	11 mm
Thickness of flange:	e_f =	19 mm
Cross-sectional area:	A_a =	14900 mm²
Yield stress:	f_y =	235 N/mm²
Elastic modulus:	E_a =	210,000 N/mm²
Moment of inertia:	I_z =	8560 cm^4 (weak axis)

Reinforcement:

Steel grade:	S 500	
Diameter:	4 Ø 25	
Cross-sectional area:	A_s =	1960 mm²
Yield stress:	f_s =	500 N/mm²
Elastic modulus:	E_s =	210,000 N/mm²
Moment of inertia:	I_s =	$4 \cdot 4.9 \cdot (30.0 / 2 - 5.0)^2 = 1960$ cm^4
Axis distance:	u_s =	50 mm

Concrete:

Strength category:	C 25/30	
Cross-sectional area:	A_c =	$300 \cdot 300 - 14900 - 1960 = 73{,}140$ mm²
Compression strength:	f_c =	25 N/mm²
Elastic modulus:	E_{cm} =	30,500 N/mm²
Moment of inertia:	I_c =	$30 \cdot 30^3 / 12 - 8560 - 1960 = 56{,}980$ cm^4

Loads:

Permanent loads:	G_k =	960 kN
Variable loads:	P_k =	612.5 kN

2 FIRE RESISTANCE OF A COMPOSITE COLUMN WITH PARTIALLY ENCASED STEEL SECTIONS

2.1 *Mechanical actions during fire exposure*

EN 1991-1-2

For fire design the accidental situation for combining loads is used:

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

Section 4.3

With $\psi_{2,1} = 0.3$ the axial design load during fire exposure is:

$$N_{fi,d} = 1.0 \cdot 960 + 0.3 \cdot 612.5 = 1143.8 \text{ kN}$$

2.2 *Verification using simple calculation model*

EN 1994-1-2

2.2.1 *Scope of application*

The simple calculation model is a verification in the strength domain. It has to be verified, that the load at elevated temperatures is smaller than the design resistance.

$$N_{fi,d} / N_{fi,Rd} \leq 1$$

Section 4.3.5.1

The design resistance for axial loads and buckling around the z-axis (weak axis) is calculated to:

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

where:

χ_z	Reduction factor depending on buckling curve c and non-dimensional slenderness
$N_{fi,pl,Rd}$	Design value of the plastic resistance to axial compression in the fire situation

To use the simple calculation model, different constraints have to be fulfilled. Additionally, the column should be part of a braced frame.

Section 4.3.5.2
Section G.8

Table 1. Constraints for using simple calculation model

Constraint	Existing	
$\max l_\theta = 13.5 \cdot b = 13.5 \cdot 0.3 = 4.05$ m	$l_\theta = 0.5 \cdot 4.0 = 2.0$ m	✓
230 mm $\le h \le$ 1100 mm	h = 300 mm	✓
230 mm $\le b \le$ 500 mm	b = 300 mm	✓
$1\% \le A_s / (A_c + A_s) \le 6\%$	$19.6/(731.4+19.6) = 0.03 = 3\%$	✓
max R 120	R 60	✓
$l_\theta < 10 \cdot b$ if $\begin{cases} 230 \le b < 300 \text{ or} \\ h/b > 3 \end{cases}$	$b = 300$ mm $h/b = 300/300 = 1$	
$l_\theta < 10 \cdot b = 10 \cdot 300\ mm = 3\ m$	$l_\theta = 0.5 \cdot 4.0 = 2.0$ m	✓

2.2.2 *Calculation of the plastic design resistance and the effective flexural stiffness*

According to Annex G of EN 1994 Part 1-2, the cross-section of the composite column is reduced. Some parts of the cross-section are reduced by reducing the cross-sectional area and some by reducing the yield stress and modulus of elasticity.

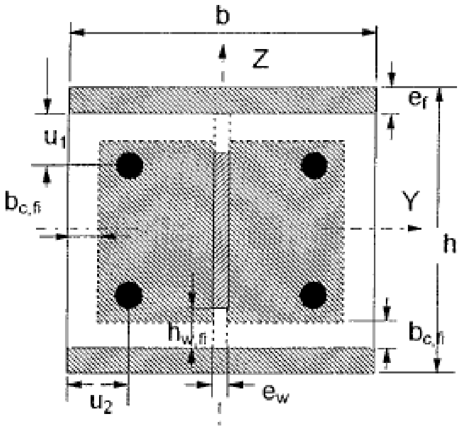

Figure 3. Reduced cross-section for structural fire design

The flanges of the steel profiles are reduced by determining reduction factors for the yield stress and the modulus of elasticity. For this, the average temperature of the flanges has to be calculated.

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

Section G.2

The temperature $\theta_{o,t}$ and the reduction factor k_t are given in Table 2. The section factor is calculated as below:

$$\frac{A_m}{V}=\frac{2\cdot(h+b)}{h\cdot b}=\frac{2\cdot(0.3+0.3)}{0.3\cdot 0.3}=13.3 \text{ m}^{-1}$$

Table 2. Parameters for calculating the average flange temperature (see EN 1994 Part 1-2, Annex G, Table G.1)

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

For R 60, the average temperature arises to:

$$\theta_{f,t}=680+9.55\cdot 13.3=807 \text{ °C}$$

With this temperature, the reduction factors $k_{y,\theta}$ and $k_{E,\theta}$ are given in Table 3.2 of EN 1994 Part 1-2, where intermediate values can be interpolated linearly.

$$k_{y,\theta}=0.06+\left((900-807)/(900-800)\right)\cdot(0.11-0.06)=0.107$$

$$k_{E,\theta}=0.0675+\left((900-807)/(900-800)\right)\cdot(0.09-0.0675)=0.088$$

The plastic axial design resistance for the flanges and its flexural stiffness are determined to:

$$N_{fi,pl,Rd,f}=2\cdot\left(b\cdot e_f\cdot k_{y,\theta}\cdot f_{ay,f}\right)/\gamma_{M,fi,a}=2\cdot(30\cdot 1.9\cdot 0.107\cdot 23.5)/1.0$$
$$=286.65 \text{ kN}$$

$$(EI)_{fi,f,z}=k_{E,\theta}\cdot E_{a,f}\cdot\left(e_f\cdot b^3\right)/6=0.088\cdot 21{,}000\cdot\left(1.9\cdot 30^3\right)/6$$
$$=1.58\cdot 10^7 \text{ kNcm}^2$$

The web is reduced by its cross-sectional area and by the yield stress. The reduction of the height is calculated as below, where this height is reduced at both edges of the flange.

$$h_{w,fi}=0.5\cdot\left(h-2\cdot e_f\right)\cdot\left(1-\sqrt{1-0.16\cdot(H_t/h)}\right)$$ Section G.3

The parameter H_t is given in Table 3.

Table 3. Parameter for reduction of the web (see EN 1994 Part 1-2, Annex G, Table G.2)

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

Therefore, $h_{w,fi}$ is calculated to:

$$h_{w,fi}=0.5\cdot(30-2\cdot 1.9)\cdot\left(1-\sqrt{1-0.16\cdot(77/30)}\right)=3.04 \text{ cm}$$

The yield stress is reduced to:

$$f_{ay,w,t}=f_{ay,w}\cdot\sqrt{1-0.16\cdot(H_t/h)}=23.5\cdot\sqrt{1-0.16\cdot(77/30)}=18.04 \text{ kN/cm}^2$$

The axial design resistance and flexural stiffness for the web during fire exposure are:

$$N_{fi,pl,Rd,w} = \left[e_w \cdot \left(h - 2 \cdot e_f - 2 \cdot h_{w,fi}\right) \cdot f_{ay,w,t}\right] / \gamma_{M,fi,a}$$
$$= \left[1.1 \cdot (30 - 2 \cdot 1.9 - 2 \cdot 3.04) \cdot 18.04\right] / 1.0$$
$$= 399.26 \text{ kN}$$

$$(EI)_{fi,w,z} = \left[E_{a,w} \cdot \left(h - 2 \cdot e_f - 2 \cdot h_{w,fi}\right) \cdot e_w^3\right] / 12$$
$$= \left[21{,}000 \cdot (30 - 2 \cdot 1.9 - 2 \cdot 3.04) \cdot 1.1^3\right] / 12$$
$$= 0.0047 \cdot 10^7 \text{ kNcm}$$

An exterior layer of concrete with a thickness $b_{c,fi}$ is neglected in the calculation. This thickness is given in Table 4.

$\Rightarrow \quad b_{c,fi} = 1.5$ cm

Section G.4

Table 4. Thickness reduction of concrete (see EN 1994 Part 1-2, Annex G, Table G.3)

Standard fire resistance	$b_{c,fi}$ [mm]
R30	4.0
R 60	15.0
R 90	$0.5 \cdot (A_m / V) + 22.5$
R 120	$2.0 \cdot (A_m / V) + 24.0$

The rest of the concrete is reduced by the reduction factor $k_{c,\theta}$ which depends on the temperature of the concrete. The average temperature of the concrete is given in Table 5. It depends on the section factor A_m/V.

Table 5. Average temperature of the concrete depending on the section factor (see EN 1994 Part 1-2, Annex G, Table G.4)

R 30		R 60		R 90		R 120	
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

$$\Rightarrow \quad \theta_{c,t} = 400 - \left((21 - 13.3)/(21 - 9)\right) \cdot (400 - 300) = 336 \text{ °C}$$

where:

$$A_m / V = 13.3 \text{ m}^{-1},$$

The reduction factor $k_{c,\theta}$ and the strain $\varepsilon_{cu,\theta}$ corresponding to $f_{c,\theta}$ are given in Table 3.3 of EN 1994 Part 1-2.

$$k_{c,\theta} = 0.75 + \left((400 - 336)/(400 - 300)\right) \cdot (0.85 - 0.75) = 0.814$$

$$\varepsilon_{cu,\theta} = \left[10 - \left((400 - 336)/(400 - 300)\right) \cdot (10 - 7)\right] \cdot 10^{-3} = 8.08 \cdot 10^{-3}$$

The secant modulus of concrete can therefore be calculated to:

$$E_{c,sec,\theta} = k_{c,\theta} \cdot f_c / \varepsilon_{cu,\theta} = 0.814 \cdot 2.5 / \left(8.08 \cdot 10^{-3}\right) = 251.9 \text{ kN/cm}^2$$

The axial design resistance and the flexural stiffness of the concrete can now be determined:

$$\begin{aligned} N_{fi,pl,Rd,c} &= 0{,}86 \cdot \left(\left(\left(h - 2 \cdot e_f - 2 \cdot b_{c,fi}\right) \cdot \left(b - e_w - 2 \cdot b_{c,fi}\right)\right) - A_s\right) \\ &\quad \cdot f_{c,\theta} / \gamma_{M,fi,c} \\ &= 0{,}86 \cdot \left(\left(\left(30 - 2 \cdot 1{,}9 - 2 \cdot 1{,}5\right) \cdot \left(30 - 1{,}1 - 2 \cdot 1{,}5\right)\right) - 19{,}6\right) \\ &\quad \cdot \left(0{,}814 \cdot 2{,}5\right) / 1{,}0 \\ &= 1017{,}3 \text{ kN} \end{aligned}$$

$$\begin{aligned} \left(EI\right)_{fi,c,z} &= E_{c,sec,\theta} \cdot \left(\left(\left(h - 2 \cdot e_f - 2 \cdot b_{c,fi}\right) \cdot \left(\left(b - 2 \cdot b_{c,fi}\right)^3 - e_w^3\right) / 12\right) - I_{s,z}\right) \\ &= 251{,}9 \cdot \left(\left(\left(30 - 2 \cdot 1{,}9 - 2 \cdot 1{,}5\right) \cdot \left(\left(30 - 2 \cdot 1{,}5\right)^3 - 1{,}1^3\right) / 12\right) - 1960\right) \\ &= 0{,}909 \cdot 10^7 \text{ kNcm}^2 \end{aligned}$$

The reinforcing bars are only reduced by its yield stress and modulus of elasticity. The reduction factor $k_{y,t}$ for the reduction of the yield stress is given in Table 6 and the reduction factor $k_{E,t}$ for the reduction of the modulus of elasticity is gained from Table 7. Both are depending on the fire resistance class and the geometrical average u of the axis distances of the reinforcing bars to the outer borders of the concrete.

$$u = \sqrt{u_1 \cdot u_2} = \sqrt{50 \cdot 50} = 50 \text{ mm}$$ Section G.5

where:

u_1 the axis distance from the outer reinforcing bar to the inner flange edge

u_2 the axis distance from the outer reinforcing bar to the concrete surface

Table 6. Reduction factor $k_{y,t}$ for the yield stress f_{sy} of the reinforcing bars (see EN 1994 Part 1-2, Annex G, Table G.5)

Standard fire resistance	u [mm] 40	45	50	55	60
R 30	1	1	1	1	1
R 60	0.789	0.883	0.976	1	1
R 90	0.314	0.434	0.572	0.696	0.822
R 120	0.170	0.223	0.288	0.367	0.436

Table 7. Reduction factor $k_{E,t}$ for the modulus of elasticity E_s of the reinforcing bars (see EN 1994 Part 1-2, Annex G, Table G.6)

Standard fire resistance	u [mm] 40	45	50	55	60
R 30	0.830	0.865	0.888	0.914	0.935
R 60	0.604	0.647	0.689	0.729	0.763
R 90	0.193	0.283	0.406	0.522	0.619
R 120	0.110	0.128	0.173	0.233	0.285

$\Rightarrow$ $k_{y,t}$ = 0,976

$k_{E,t}$ = 0,689

The plastic design resistance and the flexural stiffness of the reinforcing bars are calculated to:

$$N_{fi,pl,Rd,s} = A_s \cdot k_{y,t} \cdot f_{sy} / \gamma_{M,fi,s} = 19,6 \cdot 0,976 \cdot 50,0/1,0 = 956,5 \text{ kN}$$

$$(EI)_{fi,s,z} = k_{E,t} \cdot E_s \cdot I_{s,z} = 0,689 \cdot 21\,000 \cdot 1960 = 2,836 \cdot 10^7 \text{ kNcm}^2$$

The design resistance all-over the cross-section is determined to:

$$\begin{aligned} N_{fi,pl,Rd} &= N_{fi,pl,Rd,f} + N_{fi,pl,Rd,w} + N_{fi,pl,Rd,c} + N_{fi,pl,Rd,s} \\ &= 286,7 + 399,3 + 1017,3 + 956,5 \\ &= 2659,8 \text{ kN} \end{aligned}$$

To calculate the effective flexural stiffness of the cross-section, reduction coefficients $\varphi_{i,\theta}$ have to be determined. They are given in Table 8.

Table 8. Reduction coefficients for calculation of effective flexural stiffness (see EN 1994 Part 1-2, Annex G, Table G.7)

Standard fire resistance	$\varphi_{f,\theta}$	$\varphi_{w,\theta}$	$\varphi_{c,\theta}$	$\varphi_{s,\theta}$
R 30	1.0	1.0	0.8	1.0
R 60	0.9	1.0	0.8	0.9
R 90	0.8	1.0	0.8	0.8
R 120	1.0	1.0	0.8	1.0

$$\begin{aligned} (EI)_{fi,eff,z} &= \varphi_{f,\theta} \cdot (EI)_{fi,f,z} + \varphi_{w,\theta} \cdot (EI)_{fi,w,z} + \varphi_{c,\theta} \cdot (EI)_{fi,c,z} + \varphi_{s,\theta} \cdot (EI)_{fi,s,z} \\ &= 0,9 \cdot 1,58 \cdot 10^7 + 1,0 \cdot 0,0047 \cdot 10^7 + 0,8 \cdot 0,909 \cdot 10^7 + 0,9 \cdot 2,836 \cdot 10^7 \\ &= 4,70 \cdot 10^7 \text{ kNcm}^2 \end{aligned}$$

2.2.3 *Calculation of the axial buckling load at elevated temperatures*

Section G.6

The Euler buckling load or elastic critical load follows by:

$$N_{fi,cr,z} = \pi^2 \cdot (EI)_{fi,eff,z} / l_\theta^2 = \pi^2 \cdot 4,70 \cdot 10^7 / (0,5 \cdot 400)^2 = 11610,7 \text{ kN}$$

where:

l_θ buckling length of the column in the fire situation

The non-dimensional slenderness is obtained from:

$$\bar{\lambda}_\theta = \sqrt{N_{fi,pl,R} / N_{fi,cr,z}} = \sqrt{2659,8/11610} = 0,48$$

where:

$N_{fi,pl,R}$ the value $N_{fi,pl,Rd}$ with partial safety factors $\gamma_{M,fi,I}$ = 1.0

The reduction factor χ_z is determined by using buckling curve c of Table 6.1 of EN 1993 Part 1-1 and the non-dimensional slenderness in the fire situation.

EN 1993-1-1

Section 6.3.1.2

$$\chi_z = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}_\theta^2}} = \frac{1}{0,68 + \sqrt{0,68^2 - 0,48^2}} = 0,86$$

where:

$$\begin{aligned} \Phi &= 0,5 \cdot \left(1 + \alpha \cdot (\bar{\lambda}_\theta - 0,2) + \bar{\lambda}_\theta^2\right) = 0,5 \cdot \left(1 + 0,49 \cdot (0,48 - 0,2) + 0,48^2\right) \\ &= 0,68 \end{aligned}$$

The buckling design resistance is calculated to: EN 1994-1-2

$$N_{fi,Rd,z} = \chi_z \cdot N_{fi,pl,Rd} = 0,86 \cdot 2659,8 = 2287,4 \text{ kN}$$

Verification:

$$N_{fi,d} / N_{fi,Rd,z} = 1143,8/2287,4 = 0,50 < 1 \quad \checkmark$$

2.3 *Verification using tabulated data method*

The verification using tabulated data is to be done in the strength domain. Section 4.2.3.3

When determining the load level $\eta_{fi,t}$ the reinforcement ratio should be between 1% and 6%. Higher or lower ratios should not be taken into account.

$$\frac{A_s}{A_c + A_s} \begin{cases} \geq 1\% \\ \leq 6\% \end{cases}$$

$$\frac{19.6}{731.4 + 19.6} = 0.03 = 3\% \begin{cases} > 1\% \\ < 6\% \end{cases}$$

The load level is calculated to:

$$\eta_{fi,t} = E_{fi,d,t} / R_d = N_{fi,d} / N_{Rd} = 1143.8/4130.4 = 0.28$$

The parameters given in Table 4.6 of EN 1994-1-2 may be interpolated linearly. In this case it is not necessary to interpolate.

Table 9. Verification of composite column with partially encased steel sections

Minimum	Existing	
$\min e_w/e_f = 0.5$	$e_w/e_f = 1.1/1.9 = 0.58$	✓
$\min b = \min h = 200$ mm	$b = h = 300$ mm	✓
$\min u_s = 50$ mm	$u_s = 50$ mm	✓
$\min A_s/(A_c + A_s) = 4\%$	$A_s/(A_c + A_s) = 3\%$	$\cancel{\checkmark}$

The reinforcement ratio of the composite column is too low. To increase the ratio, reinforcement bars with bigger diameters or multiple reinforcement bars per corner can be applied.

However, the verification by using the simple calculation model could be accomplished successfully. This shows that the “tabulated data” method leads to conservative results.

REFERENCES

EN 1991, *Eurocode 1:Actions on structures – Part 1-2: General actions – Actions on structures exposed to fire*, Brussels: CEN, November 2002

EN 1993, *Eurocode 3: Design of steel structures – Part 1-1: General rules*, Brussels: CEN, May 2005

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		
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		
ORIGINAL DOCUMENT			
	Name	**Company**	**Date**
Created by	P. Schaumann	Univ.of Hannover	24/11/2005
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