Fire design of composite columns in bending and compression

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ABSTRACT: A new simple calculation model in fire situation is described for composite columns in compression and biaxial bending. This design procedure is very similar to the normal temperature design. Instead of member imperfections the moment area of imperfection is used for the transition from the member in bending and compression to the compression member. As an example the buckling curve and the interaction curve for combined compression and bending in the fire situation were examined and derived for the circular hollow steel sections with additional H-section. The comparison with all values of the design resistance of the composite column in compression and bending of the "Verbundstützenkatalog" shows sufficient correspondence on the safe side. In Minnert & Wagenknecht (2008) an example is given for a concrete filled circular hollow steel section with additional H-section. For a composite column being part of a braced multi-storey frame, the fire safety in bending and compression is determined. The range of applicability of the proposed simple calculation model should be extended for other composite cross sections.

1 BASIS OF THE SIMPLE CALCULATION MODEL

Apart from tabulated data, a simple calculation model for fire design is of special interest for the practice. Therefore, the range of applicability of simple calculation models should be extended.

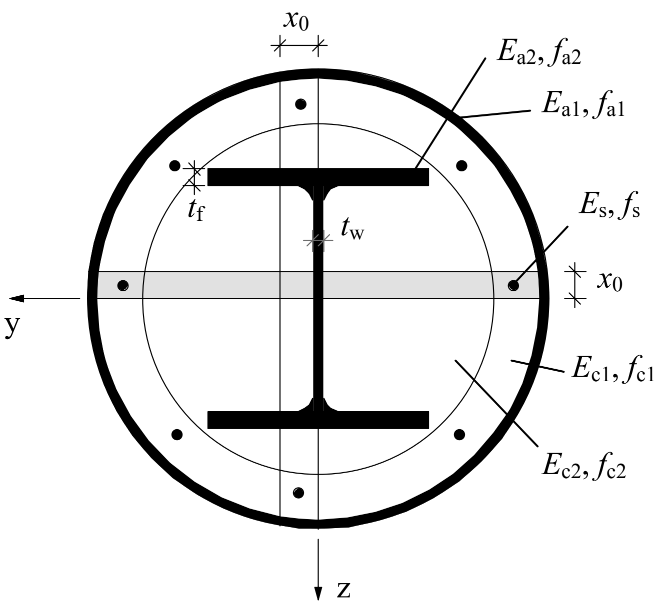

Figure 1 Cross section of a composite column in fire situation

The design procedure in the fire situation described hereafter is very similar to the normal temperature design of composite columns, Minnert & Wagenknecht (2008). In structural fire design, the composite cross section is divided into several parts with constant temperature θ. This is also a composite section but with a greater number of partial sections with reduced characteristic strength and stiffness depending on the temperature θ, for example the cross section of the composite column in Figure 1 is divided into 4 partial cross sections.

- cross-sectional area of the circular hollow section E_{a1}, f_{a1}
- cross-sectional area of the H-section E_{a2}, f_{a2}
- external cross-sectional area of the concrete E_{c1}, f_{c1}
- internal cross-sectional area of the concrete E_{c2}, f_{c2}

The design procedure of composite columns in fire situation does not differ from the design at normal temperature.

2 COLUMNS IN AXIAL COMPRESSION

1.) Characteristic value of the resistance of a member in axial compression in the fire situation

$$N_{\mathrm{fi,pl,R}} = \sum_{j} A_a \cdot f_{\mathrm{ay},\theta} + \sum_{k} A_s \cdot f_{\mathrm{sy},\theta} + \sum_{m} A_c \cdot f_{\mathrm{c},\theta} \tag{1}$$

2.) Design value of the resistance of a member in axial compression in the fire situation

$$N_{\mathrm{fi,pl,Rd}} = \sum_{j} A_a \cdot \frac{f_{\mathrm{ay},\theta}}{\gamma_{\mathrm{M,fi,a}}} + \sum_{k} A_s \cdot \frac{f_{\mathrm{sy},\theta}}{\gamma_{\mathrm{M,fi,s}}} + \sum_{m} A_c \cdot \frac{f_{\mathrm{c},\theta}}{\gamma_{\mathrm{M,fi,c}}} \tag{2}$$

The partial safety factors $\gamma_{\mathrm{M,fi}}$ are taken as 1,0.

3.) Elastic critical load in the fire situation
For a simple calculation of the elastic critical load assumptions about the flexural stiffness and the buckling length are necessary. The simplest way for the calculation of the elastic critical load is to assume a constant effective flexural stiffness over the length of the compression member.

$$N_{\mathrm{fi,cr}} = \frac{\pi^2 \cdot (EI)_{\mathrm{fi,eff}}}{l_\theta^2} \tag{3}$$

$$(EI)_{\mathrm{fi,eff}} = \sum_{j} \varphi_{\mathrm{a},\theta} \cdot E_{\mathrm{a},\theta} \cdot I_a + \sum_{k} \varphi_{\mathrm{s},\theta} \cdot E_{\mathrm{s},\theta} \cdot I_s + \sum_{m} \varphi_{\mathrm{c},\theta} \cdot E_{\mathrm{c},\theta} \cdot I_c \tag{4}$$

l_θ – buckling length in fire situation

I_i – second moment of area of the partial section *i*

$E_{i,\theta}$ – the mean value of the modulus of elasticity of the partial section *i* in fire situation

$\varphi_{i,\theta}$ – reduction factor of the variable modulus of elasticity of the partial section *i* in fire situation

In EN 1994-1-2 the modulus of elasticity $E_{a,\theta}$ of structural steel and $E_{s,\theta}$ of reinforcing steel depending on the temperature θ are specified. In this code there is no value of the tangent modulus of elasticity $E_{c,\theta}$ for concrete at elevated temperatures θ. But a mathematical formula of the stress-strain relationship for concrete in compression at elevated temperature θ is given, see figure 2 and also Schneider, Franssen, Lebeda (2008).

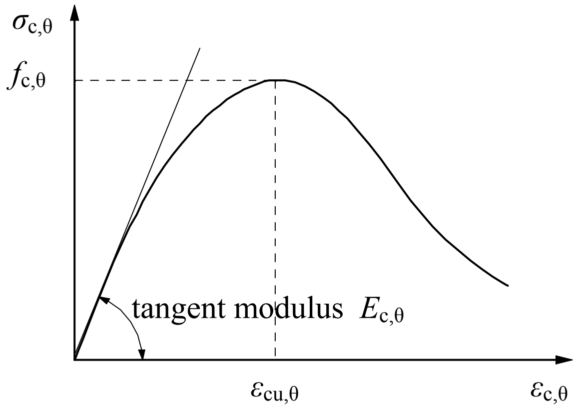

Figure 2 Stress-strain relationship for concrete in compression at elevated temperature θ

$$\sigma_{c,\theta} = \frac{3 \cdot \frac{f_{c,\theta}}{\varepsilon_{cu,\theta}} \cdot \varepsilon_{c,\theta}}{2 + \frac{1}{\varepsilon_{cu,\theta}^3} \cdot \varepsilon_{c,\theta}^3} \tag{5}$$

It is proposed to specify the modulus of elasticity $E_{c,\theta}$ for concrete at elevated temperatures θ as the tangent modulus for $\varepsilon_{c\theta}$=0.

$$E_{c,\theta} = 1,5 \cdot \frac{f_{c,\theta}}{\varepsilon_{cu,\theta}} \tag{6}$$

The values for $f_{c,\theta}$ and $\varepsilon_{cu,\theta}$ are given in table 3.3 of EN 1994-1-2. The equation (6) makes possible the transition to the design at normal temperature. Since at normal temperature θ= 20° it follows:

$$f_{c,20°C} = f_{ck} \qquad \varepsilon_{cu,20°C} = 2,5 \cdot 10^{-3} \qquad E_{c,20°C} = 600 \cdot f_{ck} \tag{7}$$

But in EN 1994-1-1 the modulus of elasticity $E_{c,20°C}$ was substituted by a reduced modulus of elasticity of concrete:

$$E_{c,20} = 600 \cdot f_{ck} \cong 0,6 \cdot E_{cm} \tag{8}$$

4.) Relative slenderness

$$\bar{\lambda}_\theta = \sqrt{\frac{N_{fi,pl,R}}{N_{fi,cr}}} \tag{9}$$

5.) Reduction factor for flexural buckling χ

For the design of columns in axial compression reduction factor χ of the buckling curves are used.

$$N_{fi,Rd} = \chi \cdot N_{fi,pl,Rd} \tag{10}$$

The most important reference value for the design of composite columns in compression is the elastic critical normal force $N_{fi,cr}$. In the Eurocode the buckling curves can be expressed by the following equations:

In the fire situation the relative slenderness is expressed as $\bar{\lambda} = \bar{\lambda}_\theta$. A reduction is not necessary if $\bar{\lambda} \leq \bar{\lambda}_0$.

$$\Phi = 0,5 \cdot \left[1 + \alpha \cdot \left(\bar{\lambda} - \bar{\lambda}_0\right) + \beta \cdot \bar{\lambda}^2\right]$$

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \beta \cdot \bar{\lambda}^2}} \qquad \text{but } \chi \leq 1,0 \text{ and } \chi \leq \frac{1}{\bar{\lambda}^2} \tag{11}$$

The free choice of the factors α and β allows it to approximate the buckling curves to the results of experiments and numerical analysis.

6.) Verification

$$\frac{N_{\mathrm{fi,Ed}}}{\chi \cdot N_{\mathrm{fi,pl,Rd}}} \leq 1 \qquad (12)$$

3 BENDING AND COMPRESSON IN FIRE SITUATION

For composite columns subjected to compression and bending the internal forces are bending moments, axial forces and vertical forces. Therefore it is necessary to know the plastic bending resistance $M_{\mathrm{fi,pl,N,Rd}}$ taking into account the normal force. Here a simplification for the interaction curve is used, see figure 3. The influence of transverse shear forces on the resistance to bending and normal force should be considered, if the shear force on the steel section exceeds 50% of the design shear resistance of the steel section.

A linear interaction is taken into account in the area I:

$$N_{\mathrm{fi,Ed}} \geq N_{\mathrm{fi,c,Rd}} \qquad M_{\mathrm{fi,pl,N,Rd}} = \left(1 - \frac{N_{\mathrm{fi,Ed}} - N_{\mathrm{fi,c,Rd}}}{N_{\mathrm{fi,pl,Rd}} - N_{\mathrm{fi,c,Rd}}}\right) \cdot M_{\mathrm{fi,pl,Rd}} \qquad (13)$$

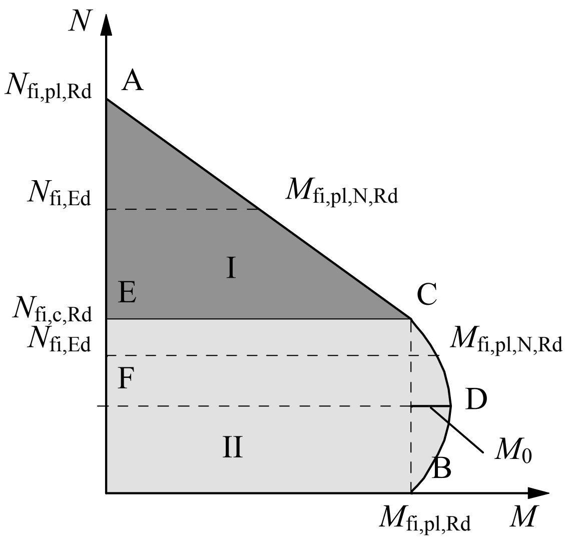

Figure 3 Simplified interaction curve for combined compression and uniaxial bending

In area II the interaction curve is approximately a parabola with the rise M_0.

$$N_{\mathrm{fi,Ed}} < N_{\mathrm{fi,c,Rd}} \qquad M_{\mathrm{fi,pl,N,Rd}} = M_{\mathrm{fi,pl,Rd}} + 4 \cdot M_0 \cdot \left(\frac{N_{\mathrm{fi,Ed}}}{N_{\mathrm{fi,c,Rd}}} - \left(\frac{N_{\mathrm{fi,Ed}}}{N_{\mathrm{fi,c,Rd}}}\right)^2\right) \qquad (14)$$

Calculating the design value of the plastic resistance of the composite section the reduction of the variable strength in the partial cross section i should be considered.

$$M_{\mathrm{fi,pl,Rd}} = \sum_{\mathrm{j}} \rho_{\mathrm{a},\theta} \cdot W_{\mathrm{pl,a}} \cdot \frac{f_{\mathrm{ay},\theta}}{\gamma_{\mathrm{M,fi,a}}} + \sum_{\mathrm{k}} \rho_{\mathrm{s},\theta} \cdot W_{\mathrm{pl,s}} \cdot \frac{f_{\mathrm{s},\theta}}{\gamma_{\mathrm{M,fi,s}}} + \sum_{\mathrm{m}} \rho_{\mathrm{c},\theta} \cdot W_{\mathrm{pl,c}} \cdot \frac{f_{\mathrm{c},\theta}}{\gamma_{\mathrm{M,fi,c}}} \qquad (15)$$

$\rho_{\mathrm{i},\theta}$ – reduction factor of the variable strength of the partial section i in fire situation

4 EFFECT OF ACTIONS IN FIRE SITUATION

The second order analysis should be used calculating the effects of actions of the structure. The transition from the member in bending and compression to the compression member is possible by different ways:

- consideration of member imperfections
- consideration of loads of imperfection
- consideration of the moment area of imperfection, see figure 4

Basis of the 3 methods of verification is the elastic critical normal force of the structure and the resistance of members in axial compression.

A simple method for the fire design of columns in bending and compression is proposed using the "moment area of imperfection", see figure 4. If the normal force $N_{fi,Ed}$ reaches the design compression resistance $N_{fi,Rd}$, a column cannot carry any additional bending moment. If $N_{fi,Ed}$ is less than $N_{fi,Rd}$, a reduced "bending moment of imperfection" $M_{0,Ed}$ can be taken into account. The reduced "bending moment of imperfection" $M_{0,Ed}$ can be expressed as:

$$M_{0,Ed} = \frac{N_{fi,Ed}}{N_{fi,Rd}} \cdot \alpha_M \cdot M_{0,Rd} \qquad (16)$$

$\alpha_M = 0{,}9$ steel grades S235, S275 and S355

$\alpha_M = 0{,}8$ steel grades S420 and S460

The factor α_M takes into account that the full plastic bending resistance cannot be achieved because of the limit strains of the concrete.
The linear reduction of this moment $M_{0,Ed}$ is on the safe side taking into account the partial plasticity depending on the additional bending moment. For the proposed design method, the resistance to normal force alone and the interaction curve for compression and bending must be known (for any given composite cross section).

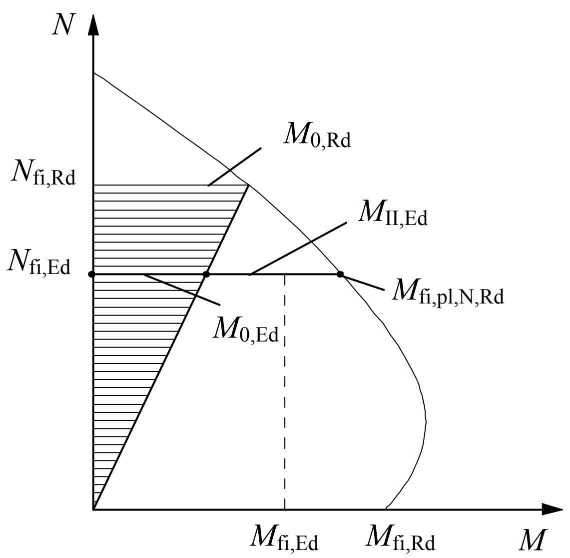

Figure 4 Definition of the moment area of imperfection

The "moment area of imperfection" may be used either for the design of one-story columns or for columns in braced and unbraced frames. The effects of member imperfections are taken into account by the reduced "bending moment of imperfection" $M_{0,Ed}$.

The load effects $N_{fi,Rd}$ and $M_{fi,Ed}$ due to design actions are calculated taking into account second order effects but without member imperfections, see $M_{II,Ed}$ in figure 4. The effective flexural stiffness due to equation (4) should be used. In this way, the load bearing capacity of the structural system, which depends on hot and cold elements in the case of fire, is taken into account.

$$M_{fi,Ed} = M_{0,Ed} + M_{II,Ed} \quad \text{but} \quad M_{II,Ed} \geq M_{I,Ed} \tag{17}$$

$M_{I,ed}$ – bending moment without second order effects

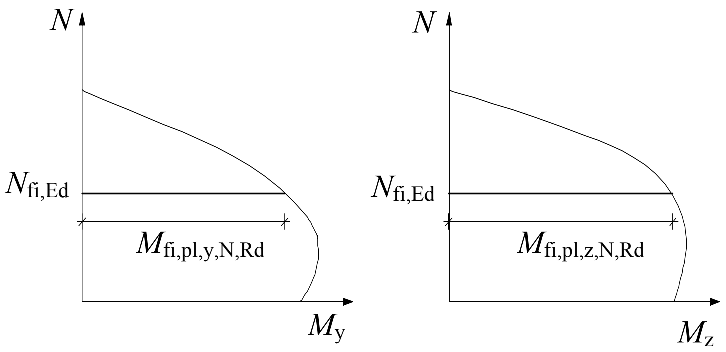

Figure 5 Design of compression and bending

For composite columns and compression members with uniaxial and biaxial bending the plastic bending resistance $M_{fi,pl,N,Rd}$ may be calculated separately for each axis, see figure 5.
The reduced “bending moment of imperfection” $M_{0,Ed}$ should be considered only in the plane in which failure is expected to occur. If it is not evident which plane is the more critical, checks should be made for both planes.
For combined compression and bending the following conditions should be satisfied for the stability check.

$$\frac{M_{fi,y,Ed}}{M_{fi,pl,y,N,Rd}} \leq \alpha_{M,y} \quad \frac{M_{fi,z,Ed}}{M_{fi,pl,z,N,Rd}} \leq \alpha_{M,z} \quad \frac{M_{y,Ed}}{M_{fi,pl,y,N,Ed}} + \frac{M_{z,Ed}}{M_{fi,pl,z,N,Rd}} \leq 1{,}0 \tag{18}$$

The simple calculation model according to EN 1994-1-2 should be used only for braced frames. But unbraced frames are used often in steel structures. Unbraced frames with composite members may be calculated also with the proposed design model. In this case the elastic critical load, the flexural stiffness of the members and the flexibility of the connections must be known in the fire situation. The effective flexural stiffness $(EI)_{fi,eff}$ due to equation (4) should be taken into account for the composite columns. The cracking of the concrete should be considered determining the flexural stiffness of the other member of the frame.

5 SIMPLE CALCULATION MODEL FOR CIRCULAR HOLLOW SECTIONS WITH ADDIONTINAL H-SECTIONS

5.1 *Scope of application*

As example for the design procedure in the fire situation the circular hollow steel sections with additional H-section shall be examined, see Viehl (2008). The basis of the following simple calculation is the “Verbundstützenkatalog”, Prüfbericht Nr. 4117.20-007/04 (2005). In this paper the design resistance in compression and bending of different composite columns with this section has been determined due to the general method of design in EN 1994-1-2. According to this paper the following scope of application should be considered:

1. The calculation model should be used only for columns in braced structures and the critical plane for compression only should be the *z-z* axis.
2. The standard fire resistance should be R 60, R 90 and R 120.

3. The composite section is the circular hollow steel sections with additional H-section and a steel grade up to S355.
4. The calculation model applies to columns with concrete grades C30/37 to C50/60.
5. The P/A[m^{-1}]-values (perimeter/area) are 5 to 15.
6. The concrete cover of the additional H-section should be 4 cm.
7. The relationship between the diameter and the thickness of the circular hollow section is greater than 25.
8. The maximum values (*d*/*t*) of EN 1994-1-1 are not exceeded.
9. The relative slenderness $\bar{\lambda} = \bar{\lambda}_\theta$ does not exceed 1.3.
10. The simplified interaction curve for bending and compression should be applied.

5.2 *Reduction factors*

The temperature distribution over the cross section was determined with the NISA-program for the standard fire resistance R 60, R 90 and R 120 and the P/A[m^{-1}]-values 5-7,5-10-12,5-15, see Viehl (2008). The investigation shows, that it is sufficient to take into account a linear interaction and distinguish between two areas of P/A[m^{-1}]-values greater or smaller 10. The reduction factors are shown in table 1.

$$f_{a1,\theta} = k_{a1f} \cdot f_{yk} \quad f_{a2,\theta} = k_{a2f} \cdot f_{yk} \quad f_{c1,\theta} = k_{c1f} \cdot f_{ck} \quad f_{c2,\theta} = k_{c2f} \cdot f_{ck} \tag{19}$$

$$E_{a1,\theta} = k_{a1E} \cdot E_a \quad E_{a2,\theta} = k_{a2E} \cdot E_a \quad E_{c1,\theta} = k_{c1E} \cdot 600 \cdot f_{ck} \quad E_{c2,\theta} = k_{c2E} \cdot 600 \cdot f_{ck} \tag{20}$$

In the fire situation the modulus of elasticity and the strength of the chosen partial section *i* are not constant and increase in the direction of the elastic neutral axis. Therefore the assumption of a mean value for the modulus of elasticity and the strength is an approximation on the unsafe side. The analysis of the temperature distribution shows, that a reduction is not necessary for the circular hollow section, the external cross-sectional area of 4 cm of the concrete and the additional H-section. But for the internal cross-sectional area of the concrete a reduction factor $\varphi_{i,\theta}$ and $\rho_{i,\theta}$ should be

Table 1: Reduction factor of the mean modulus of elasticity and the mean strength, see Viehl (2008)

	P/A [m^{-1}]	R 60	R 90	R 120
k_{a1f}	≤10	0,0903-0,00036 * (P/A-5)	0,0563-0,0001 * (P/A-5)	0,0383-0,00008 * (P/A-5)
	>10	0,0885-0,0003 * (P/A-10)	0,0531-0,0001 * (P/A-10)	0,0379-0,0001 * (P/A-10)
k_{a2f}	≤10	1,00	1,00	1,00
	>10	1,00	1,00	1,00-0,039 * (P/A-10)
k_{c1f}	≤10	0,549-0,0066 * (P/A-5)	0,393-0,008 * (P/A-5)	0,286-0,0094 * (P/A-5)
	>10	0,516-0,0098 * (P/A-10)	0,353-0,0144 * (P/A-10)	0,239-0,00178 * (P/A-10)
k_{c2f}	≤10	1,00-0,006 * (P/A-5)	0,988-0,0162 * (P/A-5)	0,971-0,0288 * (P/A-5)
	>10	0,970-0,014 * (P/A-10)	0,907-0,0304 * (P/A-10)	0,827-0,0476 * (P/A-10)
k_{a1E}	≤10	0,0811-0,00016 * (P/A-5)	0,0603-0,00012 * (P/A-5)	0,0431-0,0001 * (P/A-5)
	>10	0,0803-0,00014 * (P/A-10)	0,0597-0,00012 * (P/A-10)	0,0426-0,00012 * (P/A-10)
k_{a2E}	≤10	0,978-0,0046 * (P/A-5)	0,949-0,0168 * (P/A-5)	0,897-0,0248 * (P/A-5)
	>10	0,956-0,0204 * (P/A-10)	0,864-0,025 * (P/A-10)	0,773-0,0324 * (P/A-10)
k_{c1E}	≤10	0,0813-0,0024 * (P/A-5)	0,0393-0,0008 * (P/A-5)	0,0286-0,00094 * (P/A-5)
	>10	0,0692-0,00362 * (P/A-10)	0,0353-0,00144 * (P/A-10)	0,0239-0,00178 * (P/A-10)
k_{c2E}	≤10	0,660-0,0304 * (P/A-5)	0,577-0,040 * (P/A-5)	0,513-0,0472 * (P/A-5)
	>10	0,508-0,028 * (P/A-10)	0,377-0,0366 * (P/A-10)	0,277-0,0362 * (P/A-10)

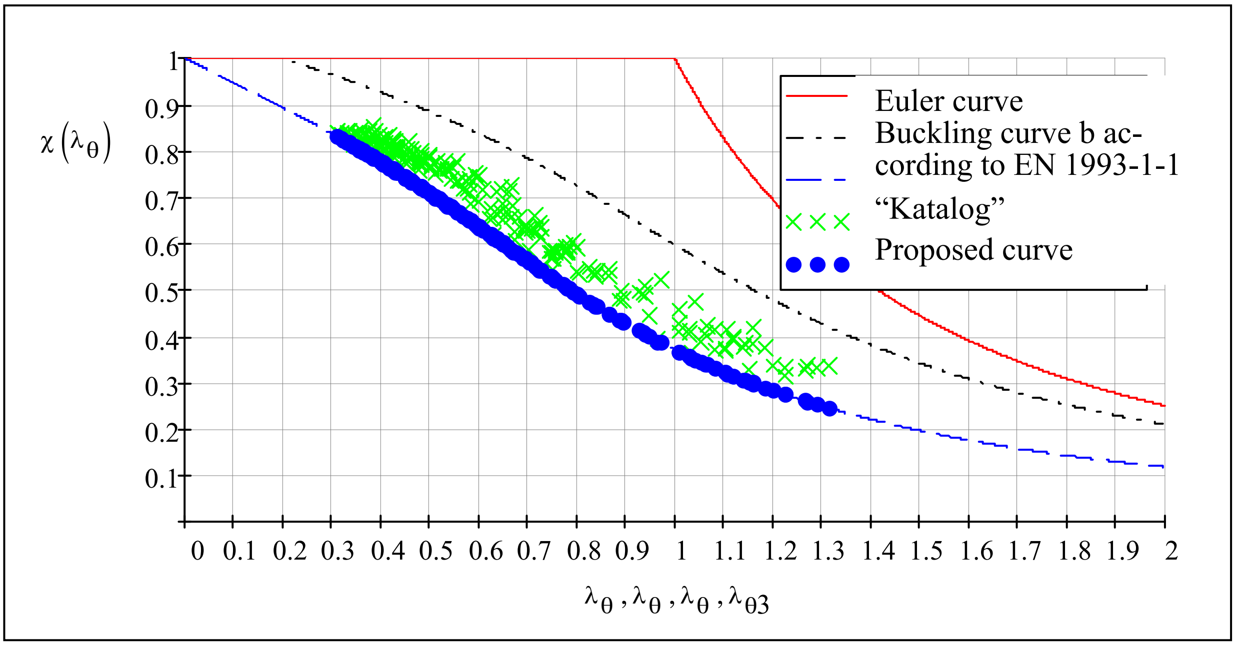

Figure 6 Comparison of "Verbundstützenkatalog" and the proposed buckling curve, see Viehl (2008)

taken into account. These values also depend on the standard fire resistance and the P/A[m^{-1}]-values. For simplification a constant reduction factor of 2/3 for the internal cross-sectional area of the concrete is used for the modulus of elasticity and the strength. The comparative calculation of all values of the "Verbundstützenkatalog" was carried out with this value. The following buckling curve according to equation (11) has been developed only for buckling about *z-z* axis. Basis of the values of α and β were the comparative calculations of all composite columns of the "Verbundstützenkatalog".

$$\Phi = 0,5 \cdot \left[1 + 0,56 \cdot \bar{\lambda}_\theta + 1,8 \cdot \bar{\lambda}_\theta^2\right]$$
$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - 1,8 \cdot \bar{\lambda}_\theta^2}} \quad \text{but } \chi \le 1,0 \text{ and } \chi \le \frac{1}{\bar{\lambda}_\theta^2} \qquad (21)$$

5.3 *Comparison with "Verbundstützenkatalog"*

The proposed buckling curve and all values of the design resistance of the composite column in compression of the "Verbundstützenkatalog" are shown in figure 6. For all values the proposed buckling curve is on the safe side. All examples for eccentrically loaded columns were calculated also. The comparison shows good results between 0.7 and 1.0, see Viehl (2008).

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