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INVESTIGATIONS ON THE LOAD BEARING BEHAVIOR OF COMPOSITE COLUMNS UNDER FIRE CONDITIONS

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ABSTRACT

A suitable alternative to fire tests has been provided with the development of simple and advanced calculation methods. They offer the possibility to do extensive parametrical studies and global structural analysis. Several codes have implemented such procedures for the design of structural elements, as for instance the code for structural fire design of composite members, Eurocode 4-1-2.

Within the EN-Version of Eurocode 4-1-1 a new design procedure for normal temperature design of composite columns was implemented, which allows a more general analysis of compression members. However, the existing design procedures for fire design of composite columns given in the Eurocode have been developed for single member verification and are not applicable for global structural analysis. Furthermore, they are only valid for compression members in braced frames.

It is intended to adapt this verification method of Eurocode 4-1-1 for normal temperature design to the fire situation to reach a wider field of application.

This contribution deals with this adaptation process. The temperature distribution over the cross section is needed for the investigation. Therefore the numerical simulation program BoFire will be presented, which is an incremental computer code based on the finite element method. The nonlinear and temperature dependent thermal and mechanical material properties of steel and concrete are used for the thermal and the load bearing analysis.

A special focus is given on the development of the interaction between the normal forces and the corresponding bending moment at high temperatures. The stress-strain-relationships of steel and concrete need special consideration resulting in a lower axial load bearing capacity compared to perfect plastic analysis, as it is achieved with the methods from normal temperature design.

UNTERSUCHUNGEN ZUM TRAGVERHALTEN VON VERBUNDSTÜTZEN IM BRANDFALL

KURZFASSUNG

Mit der Entwicklung von vereinfachten und allgemeinen Rechenverfahren steht eine geeignete Alternative zu Brandversuchen zur Verfügung. Diese Verfahren bieten die Möglichkeit, umfassende Parameterstudien sowie Berechnungen des Gesamttragwerks durchzuführen. Verschiedene Normen enthalten solche Verfahren für die Bemessung von Bauteilen, z. B. der Eurocode 4-1-2- für die brandschutztechnische Bemessung von Verbundbauteilen.

In die EN-Fassung von Eurocode 4-1-1 wurde ein neues Bemessungsverfahren für Verbundstützen bei Normaltemperatur aufgenommen, das eine allgemeingültigere Berechnung von Druckgliedern erlaubt. Allerdings wurden die derzeitigen brandschutztechnischen Bemessungsverfahren für Verbundstützen im Eurocode für Einzelbauteile entwickelt und sind nicht für Gesamttragwerke verwendbar. Außerdem gelten sie nur für Druckglieder in ausgesteiften Rahmen. Es ist beabsichtigt, das Nachweisverfahren aus dem Eurocode 4-1-1 für Normaltemperatur auf den Brandfall zu übertragen und den Anwendungsbereich zu erweitern.

Der vorliegende Beitrag befasst sich mit diesem Anpassungsprozess. Für die Untersuchung wird die Temperaturverteilung im Querschnitt benötigt. Dafür wird das Simulationsprogramm BoFire herangezogen, ein Computerprogramm auf Basis der Finite-Element-Methode. Für die thermische Analyse und die Berechnung des Tragverhaltens werden nichtlineare, temperaturabhängige Materialeigenschaften für Stahl und Beton vorgegeben.

Ein besonderes Augenmerk wird auf die Interaktion zwischen Normalkraft und Biegemoment bei hohen Temperaturen gerichtet. Dazu ist eine spezielle Betrachtung der Spannungs-Dehnungslinien von Stahl und Beton notwendig, aus der geringere axiale Traglasten resultieren als bei ideal-plastischer Berechnung wie im Rahmen der Bemessung bei Normaltemperatur.

INTRODUCTION

The combination of the two materials steel and concrete make slender structural elements for composite columns possible. Furthermore, this offers a higher fire resistance caused by the insulating effect of concrete. Over the years many different types of cross sections have been developed, each one adapted to the particular scope application.

Parallel to this evolution different design methods have been developed for the normal temperature design and for the design in the fire situation. A complete and consistent verification process is given in the Eurocode 4, part 1-1 and 1-2 for the design of composite members. However, the simplified calculation method for composite columns in the fire situation has a narrow range of application, since it is limited to concrete encased steel sections or concrete filled tubular steel sections built in braced frames. The normal temperature design method of Eurocode 4 offers a wider scope of application for most of the common cross sections, both for braced and unbraced structures. Thus, it is the intention to adapt the normal temperature design method to the fire situation.
This paper deals with aspects of this adaptation process concerning the interaction relationship between the axial normal force and the corresponding bending moment in the fire situation.

DIFFERENT CROSS SECTIONS FOR COMPOSITE COLUMNS

Practical examples

Through the years many different types of cross sections for composite columns have been developed. Ten different types are listed below as examples:

type 1:	I-shape steel profile with concrete encasement and reinforcement
type 2:	Concrete encased I-shape steel profile with reinforcement
type 3:	Concrete filled tubular hollow steel section with an embedded I-shape steel profile
type 4:	Concrete filled tubular hollow steel section with an embedded massive steel core
type 5:	Concrete filled double skin triangular hollow steel section
type 6:	Concrete filled tubular hollow steel section with reinforcement
type 7:	Concrete filled rectangular hollow steel section and reinforcement
type 8:	Concrete filled tubular hollow steel section with an embedded X-shape steel profile
type 9:	Crossed I-shape steel profiles with concrete filling and reinforcement
type 10:	Concrete filled double skin tubular hollow steel section [1]

A graphical overview over the different types of cross sections is given in Figure 1.

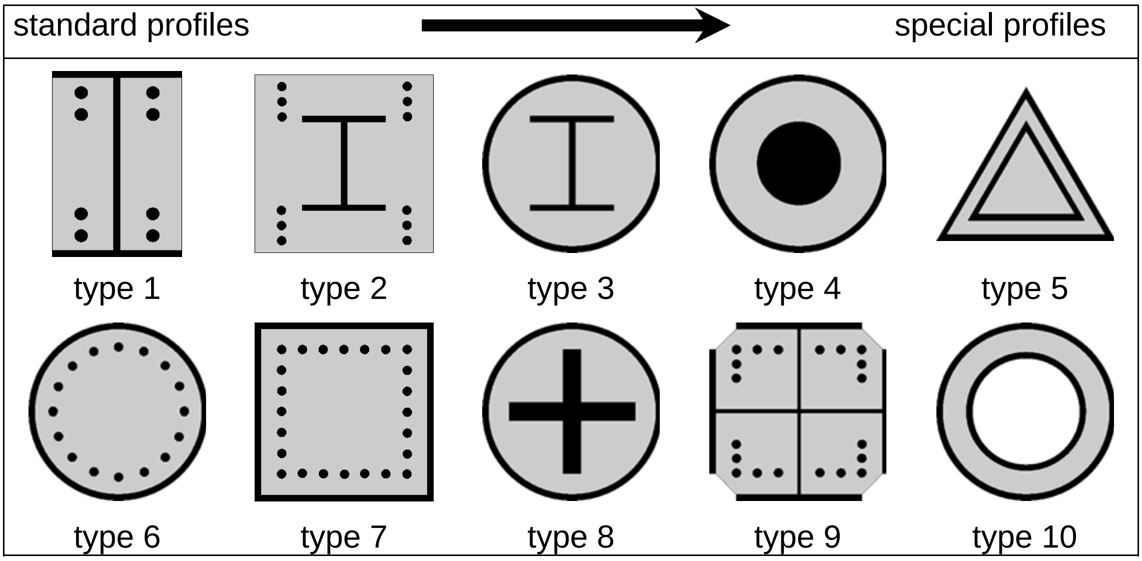


Figure 1 Types of cross sections for composite columns

Widely-used cross sections are illustrated on the left side of Figure 1 while on the right side the cross sections have a rather specialized field of application. The I-shape steel profile with concrete encasement (type 1) and the concrete filled hollow sections (type 6 and type 7) are very common in Europe for the application in office buildings and shopping centres.

The photos in Figure 2 show three large buildings in Germany where special types of cross sections have been used for the performance of composite columns. The German Technical Museum in Berlin has been expanded (inauguration 2002) using composite slabs with concrete encased steel beams and composite columns consisting of two crossed I-shape profiles. The new office building of the Nord LB in Hannover was opened in 2002 and concrete filled hollow sections with embedded massive steel core were used for the vertical elements. A rare type of composite cross section was installed in the office building of the Commerzbank in Frankfurt, which was inaugurated in 1997. The triangle shape of the concrete filled double skin steel section evolves inevitably from the triangular ground view of the building.

a) b) c)

Figure 2 Application of special types of cross sections:
a) Crossed I-shape profiles in the German Technical Museum in Berlin (*Deutsches Technik Museum Berlin*); photo: www.dtmb.de
b) concrete filled tubular hollow sections with embedded massive core in the office building of the Nord LB in Hannover;
photo: www.hamburgfotos.de
c) Concrete filled double skin triangular hollow section in the office building of the Commerzbank in Frankfurt;
photo: http//:skyline.flugzeugposter.de

Normative regulations in Germany

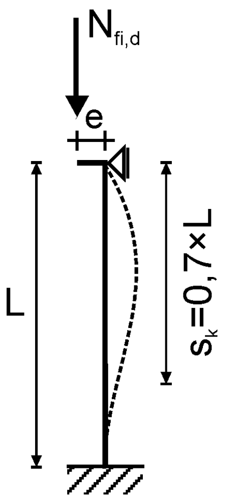
The fire design of composite columns with common cross sections like type 1, type 2, type 6 or type 7 is regulated in Germany according to DIN 4102 or Eurocode. For an application of other types of cross sections not ruled in the codes an expert certificate or an approval of the authorities is required. These circumstances lead to additional risks for both time schedule and costs.

It is essential to reduce the organizational complexity and to simplify the process to make composite columns more attractive. Thus the author made investigations on concrete filled tubular hollow steel section either with an embedded I- or X-section (type 3, type 8), which are very simple to fabricate.

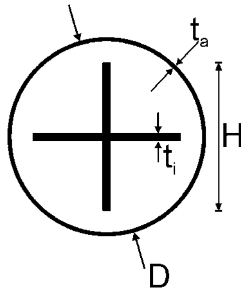
Table 1 Field of application of the Catalogue for the Design of Composite Columns ([2] and [3])

1	2	3	4	5	6
steeltube	**embedded steel profile**	**concrete**	**buckling length s_k**	**excen-tricity e**	**fire resistance class**
D x t_a	**(Cross H x t_i)**		**[m]**		
273x6,3	HE 100 B	C 30/37	2.35	0	normal temperature design
	HE 100 M				
	Cross 170x40				
	Cross 170x60				
355,6x8,0	HE 160 B				
	HE 160 M				
	Cross 240x40		2.52		R60
	Cross 240x60				
406,5x8,8	HE 200 B			D/10	
	HE 200 M				
	Cross 300x40				
	Cross 300x60				
508x8,8	HE 260 B	C 50/60	2.80		R90
	HE 260 M				
	Cross 400x40				
	Cross 400x80				
610x8,8	HE 360 B			D/5	
	HE 360 M				
	Cross 500x40		(3.36)		R120
	Cross 500x80				
711x11	HE 500 B		(3.60)		
	HE 500 M				
	Cross 600x40		(4.00)		
	Cross 600x80				

System:



Cross section:



In cooperation with the Institute for Steel Construction and Material Mechanics of the Technical University of Darmstadt a catalogue of load bearing capacities for composite columns with the cross sections mentioned ([2] and [3]) on the basis of Eurocode 4-1-2 [7] has been developed. The research project was funded by Bauen mit Stahl e.V.. The intention of this project was to provide a basis for the fire design of composite columns in standard multi storey buildings. Table 1 gives an overview of the covered field of application of the catalogue. Each of six different steel tubes can be combined with 4 different embedded profiles: One of the HEA and one of the HEM series and two X-sections with different sheet thickness t_i. Every given steel section can be filled with concrete of two different concrete strength classes. The load bearing values have been calculated for three storey heights L = 3.35 m, 3.60 m and 4.00 m. The values for the normal temperature design have been calculated with a buckling length equal to the storey height.

In addition to the normal temperature design the load bearing values are given for the fire resistance classes of 60, 90, and 120 minutes. According to

Eurocode 4-1-2 it is assumed in the fire situation, that each storey is a separated fire compartment. The buckling length can be reduced due to the higher stiffness in the upper and lower storey, which is not exposed to fire. As a result the values for the fire situation in the catalogue have been calculated for a buckling length of $s_k = 0.7 \cdot L = 2.35$ m, 2.52 m and 2.80 m. Furthermore load eccentricities D/10 and D/5 can be considered, where D is the diameter of the steel tube. Figure 3 shows an example page of the presented catalogue.

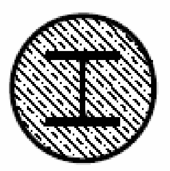
NORMAL TEMPERATURE DESIGN OF COMPOSITE COLUMNS

With the revision of Eurocode 4-1-1 for the normal temperature design of composite structures the simple calculation method for composite columns has changed. While the ENV-version [4] of the code offers a method based on the European buckling curves, the EN-version [5] contains a calculation procedure developed by Bergmann and Lindner [6]. This method allows a more general calculation, as the imperfections and the deformation of the column are considered for the internal forces with an elastic second order theory calculation. The ENV-method was limited to columns in braced frames whereas the calculation by a second order theory allows the verification of non-swayed frames and thus opens a wider field of application.

Table 2 shows the differences between the two calculation models of the ENV- and the EN-version of Eurocode 4. The level of imperfection is almost the same, since the values for the initial bow of imperfection have been evolved directly from the European buckling curves.

Besides a different consideration of the imperfections the method of Lindner and Bergmann offers an equation for the determination of the bending stiffness for the second order calculation. Following the different consideration of the imperfections the analysis of the interaction curve has changed as shown on the bottom part of Table 2. It is not necessary to consider the imperfections in the analysis of the interaction curve, since they are considered in the second order calculation.
The equation for the final verification is printed at the very low part of Table 2. With respect to the cracking of the concrete and the influence on the bending stiffness of the member the acting bending moment M_{Sd} is limited to 90% of the plastic moment resistance given from the interaction curve.

Tube 273 x 6.3 - S235
Buckling length = 2.80 m



Inner profile	**Concrete**	**Excen-tricity**	**Fire Resistance class**	**Load bearing capacity [kN]**
HE 100 B (S355)	C 30/37	0	Normal temperature	2800
			R60	799
			R90	580
			R120	458
		d/10	Normal temperature	2096
			R60	586
			R90	410
			R120	320
		d/5	Normal temperature	1704
			R60	485
			R90	346
			R120	266
	C 50/60	0	Normal temperature	3384
			R60	1085
			R90	754
			R120	572
		d/10	Normal temperature	2513
			R60	799
			R90	551
			R120	413
		d/5	Normal temperature	2018
			R60	621
			R90	427
			R120	324
			R120	593
		d/5	Normal temperature	2275
			R60	860
			R90	620
			R120	475

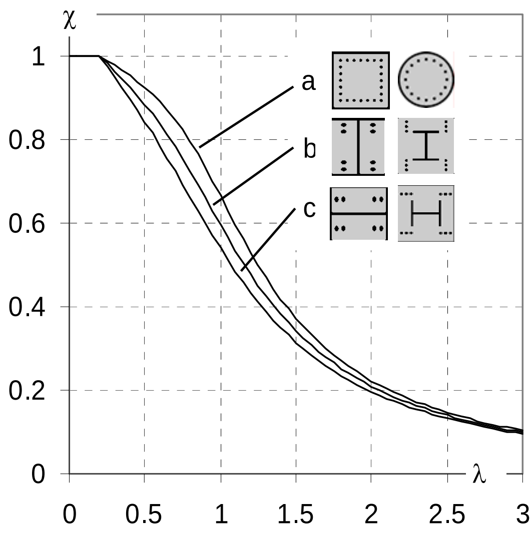
Figure 3 Extract of the Catalogue for the Design of Composite Columns [2]

Table 2 Comparison of the design procedures of the ENV- and the EN-version of Eurocode 4 for the normal temperature design of composite columns

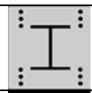
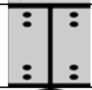
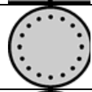
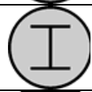
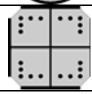
ENV 1994-1-1 (1992)	EN 1994-1-1 (2004)

Consideration of imperfections:

ENV 1994-1-1 (1992): European buckling curves



EN 1994-1-1 (2004):

cross section	Axis of buckling	w_0	
	y-y	L/200	
	z-z	L/150	
	y-y	L/200	
	z-z	L/150	
	y-y/z-z	$\rho_s \leq 3\%$	L/300
		$\rho_s \geq 3\%$	L/200
	y-y	L/200	
	z-z	L/200	
	y-y/z-z	L/200	

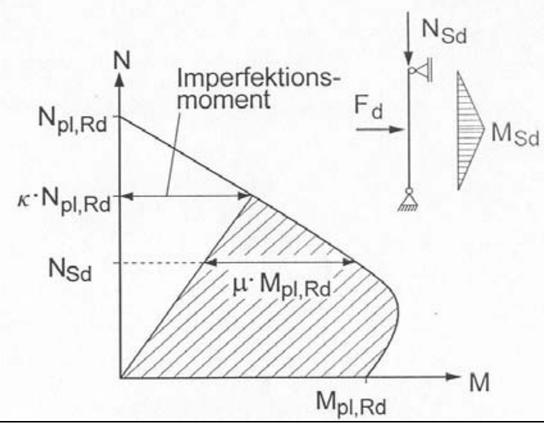
Bending stiffness:

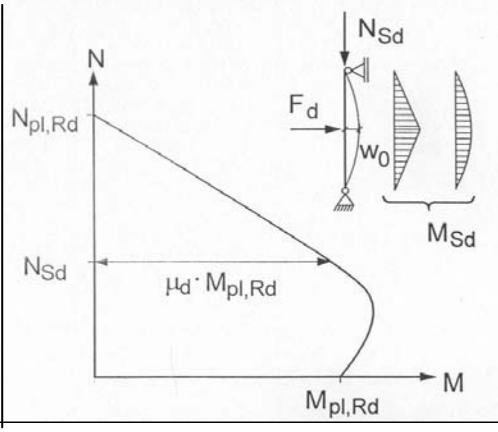
ENV 1994-1-1 (1992)	EN 1994-1-1 (2004)
For the calculation of $\bar{\lambda}$: $(EI)_e = E_a I_a + 0.59 \cdot E_{cm} I_c + E_s I_s$	For the calculation of $\bar{\lambda}$: $(EI)_e = E_a I_a + 0.6 \cdot E_{cm} I_c + E_s I_s$ For the second order theory calculation: $(EI)_e = 0,9 \cdot \left(E_a I_a + 0.5 \cdot E_{cm} I_c + E_s I_s \right)$

Calculation of the inner forces:

ENV 1994-1-1 (1992)	EN 1994-1-1 (2004)
First order theory (a second order effect can be considered with a factor k)	Second order theory

Analysis of the full plastic interaction relation:





Verification:

ENV 1994-1-1 (1992)	EN 1994-1-1 (2004)
$M_{sd}(F_d) \leq 0.9 \cdot \mu \cdot M_{pl,Rd}$	$M_{sd}(F_d, w_0) \leq 0.9 \cdot \mu \cdot M_{pl,Rd}$

FIRE DESIGN METHODS FOR COMPOSITE COLUMNS

Existing design procedures in Eurocode 4

The Eurocode 4 offers three possibilities to verify the fire resistance of composite columns:

Level 1: Tabulated data
Level 2: Simplified calculation methods
Level 3: Advanced calculation methods

One of the main principles of the Eurocodes is that the verification method is the more conservative the lower the level is. The following investigations concern simplified calculation methods (Level 2) of Eurocode 4-1-2. An analysis and comparison of other calculation methods is given in [8], but the presented procedures only differ in the way, the reduced cross section is determined.

EN 1994-1-2 [9] contains two simple calculation methods for the fire design of composite columns:

Annex G: Partially encased steel sections for buckling around the weak axis
Annex H: Concrete filled hollow sections

The design procedure for steel sections with concrete encasement will be explained in detail below.

- Temperature distribution over the cross section and its analysis

The temperature distribution after a required fire resistance time (R30 ÷ R120) is considered in a reduced cross section as illustrated in Figure 4. Every part of the cross section will be reduced by its strength and stiffness properties and/or its area. The required values can be calculated with simple equations and tabulated data. The partial forces of the flanges for example have to be calculated with a temperature reduced value for the yield strength whereas the concrete area between the flanges is reduced with a width $b_{c,fi}$ and furthermore the concrete strength of the remained area is reduced dependent on the temperature.

With the reduced cross section the design values of the plastic resistance to axial compression (1) and the flexural stiffness (2) can be determined by summation over the different parts of the cross section:

$$N_{fi,pl,Rd} = N_{fi,pl,Rd,f} + N_{fi,pl,Rd,w} + N_{fi,pl,Rd,c} + N_{fi,pl,Rd,s} \tag{1}$$

$$(EI)_{fi,eff,z} = \varphi_{f,\theta}\ (EI)_{fi,f,z} + \varphi_{w,\theta}\ (EI)_{fi,w,z} + \varphi_{c,\theta}\ (EI)_{fi,c,z} + \varphi_{s,\theta}\ (EI)_{fi,s,z} \tag{2}$$

The factors $\varphi_{i,\theta}$ consider the effect of the thermal stresses on the flexural stiffness for each part i of the cross section.

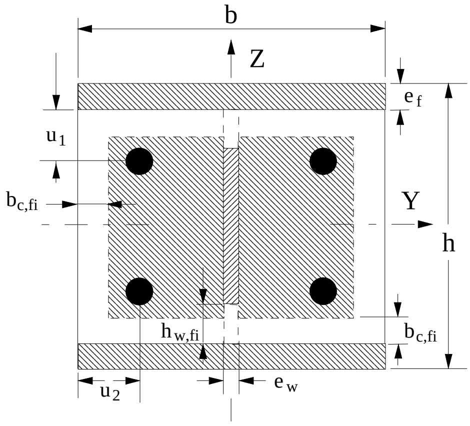


Figure 4 Reduced cross section for the simple calculation model for partially encased steel sections in EN 1994-1-2 [9]

- Calculation of the axial buckling load at elevated temperatures

The following procedure for the determination of the load bearing capacity $N_{fi,Rd,z}$ is according to the normal temperature design method of the ENV-version of Eurocode 4, as explained above, using an analysis of the European buckling curves. The required equations are listed below:

$$N_{fi,cr,z} = \pi^2 \left(EI\right)_{fi,eff,z} \Big/ \ell_\theta^2 \tag{3}$$

$$\bar{\lambda}_\theta = \sqrt{N_{fi,pl,R} / N_{fi,cr,z}} \tag{4}$$

$$N_{fi,Rd,z} = \chi_z \; N_{fi,pl,Rd} \tag{5}$$

- The relation between the non-dimensional slenderness $\bar{\lambda}_\theta$ and the reduction coefficient χ_z is determined by the European buckling curve c as given in Table 2Provision of load eccentricities

Load eccentricities can be considered by equation (6). It represents an analysis of the ratio between the eccentric and the axial load bearing capacity $N_{Rd,\delta}/N_{Rd}$ for normal temperature design.

$$N_{fi,Rd,\delta} = N_{fi,Rd} \left(N_{Rd,\delta}/N_{Rd}\right) \tag{6}$$

Software implementation of the simple calculation methods of the Eurocodes

Within an AIF-research project the Institute for Steel Construction of the University of Hannover developed a software package called H-FIRE. It contains the simple calculation models for different steel and composite members given in the Eurocodes. One of them is H-FIRE COL for the calculation of the axial buckling load for composite columns with partially encased steel sections according to ENV 1994-1-2, Annex G.

The normative basis of all programs is the ENV-version of the Eurocodes . The programs are implemented user-friendly in MICROSOFT® EXCEL and a background code in VBA (Visual Basic for Application).

The software is free available and can be requested at:

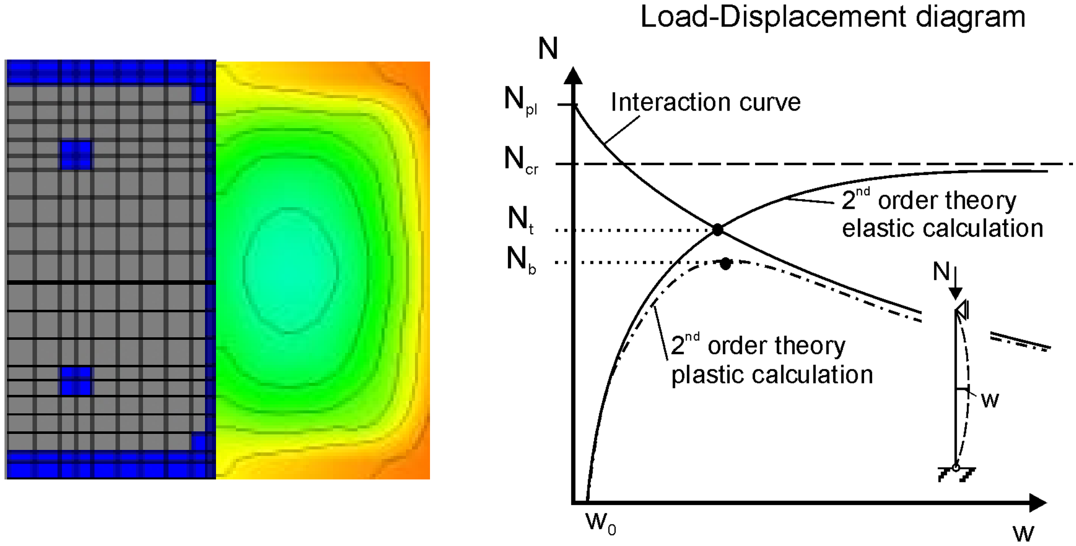
www.stahlbau.uni-hannover.de → Publications → Software

ADAPTATION OF THE NORMAL TEMPERATURE DESIGN METHOD FOR COMPOSITE COLUMNS TO THE FIRE SITUATION

General

The determination of the ultimate load in the fire situation requires a thermal and a mechanical analysis as illustrated in Figure 5. These two problems can be solved separated from each other, assuming that the deformation of the structure has no influences on the heating of the structural member. In the mechanical analysis must not be neglected that the non-uniform temperature distribution over the cross section causes thermal stresses,

In the first step the heating of the cross section for the required fire resistance duration has to be determined independently from the mechanical boundary conditions of the structural member. With this fixed temperature distribution the mechanical problem can be solved as illustrated on the right side of Figure 5. The internal forces of the column can be calculated with an elastic second order theory. Due to the nonlinear material properties of steel and concrete the internal forces are limited by the interaction relationship between the axial normal force and the corresponding bending moment, which is normally the perfect plastic interaction curve.



1. Thermal analysis 2. Mechanical analysis

Figure 5 General procedure for the determination of the ultimate load

The following investigations will show that in the fire situation the perfect plastic interaction has to be reduced because of inconsistent strains for the maximum stress level.

Furthermore the left side of Figure 5 shows that the real load bearing capacity N_b is lower than the theoretical value N_t at the calculated intersection point, because of the influence of plasticizing parts of the cross section on the bending stiffness of the member. Hence, the choice of the equivalent imperfection w_0 and the bending stiffness has to result in a conservative load bearing capacity. These phenomena will not be investigated in this paper. The special focus is on the heating of the cross section and on the interaction relationship.

THERMAL ANALYSIS OF THE CROSS SECTION

The numerical simulation program BoFire

Through the last years the numerical simulation tool called BoFire has been developed. The basic implementation was done by Schaumann [10]. It is a transient, non-linear, incremental computer code based on the finite element method. The program includes two main calculation modules: one to calculate the development and the distribution of the temperature in the structural member (thermal response model) and another to consider the mechanical behaviour of the structure, taking into account the change of material properties at elevated temperatures (mechanical response model). BoFire includes the required thermal and mechanical material properties of EN 1994-1-2 for steel and concrete.

During the preparation of the catalogue for load bearing capacities of composite columns ([2] and [3]), which is mentioned above, BoFire has been extended to calculate the temperature distribution of circular cross sections. For this a Finite Element Routine was implemented.

Thermal analysis with a Finite Element Method

The basic mathematical formulation is given through the differential equation of the transient heat conductivity by Fourier (7).

$$-\mathrm{div}\left(\lambda \cdot \mathrm{grad}\,\theta\right) + \rho \cdot c \cdot \dot{\theta} - f = 0 \qquad (7)$$

where:

λ	heat conductivity
θ	temperature
$\dot{\theta}$	derivation of the temperature with respect to the time
ρ	density
c	thermal capacity
f	internal heat source

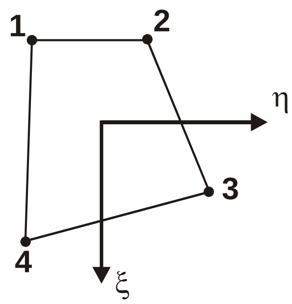
A mathematical transformation of equation (7) results in the weak formulation of the differential equation (8):

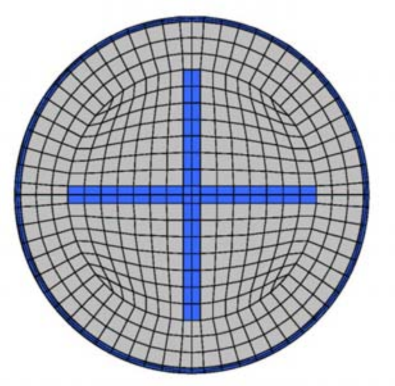
$$\int_{\Omega} \lambda \cdot \mathrm{grad}\,\theta : \mathrm{grad}\,\delta\theta \; dA + \int_{\Gamma} q \cdot \delta\theta \cdot n \; dS + \int_{\Omega} \rho \cdot c \cdot \dot{\theta} \cdot \delta\theta \; dA = 0 \qquad (8)$$

where:

Ω	area
Γ	boundary of the considered area
q	heat flux
n	normal vector on the boundary

The approach for the solution of the weak form of the differential equation is done with bi-linear shape functions (9) on a four node isoparametric element as shown on the left side of Figure 6. On the right side of the figure the final mesh of a cross section is plotted.





$$N_i = \frac{1}{4}(1 \pm \eta)(1 \pm \xi) \quad (9)$$

Figure 6 Four node isoparametric element (left)
final mesh generation for a concrete filled tubular hollow section with embedded X-shape profile (right)

MECHANICAL ANALYSIS FOR THE INTERACTION CURVE

The stress-strain relationships of steel and concrete are required for the calculation of the interaction curves between the internal normal force and bending moment. Figure 7 shows the temperature dependent values given in EN 1994-1-2 [9]. The values for steel are symmetric for tension and compression, whereas the stresses of concrete in tension are neglected.

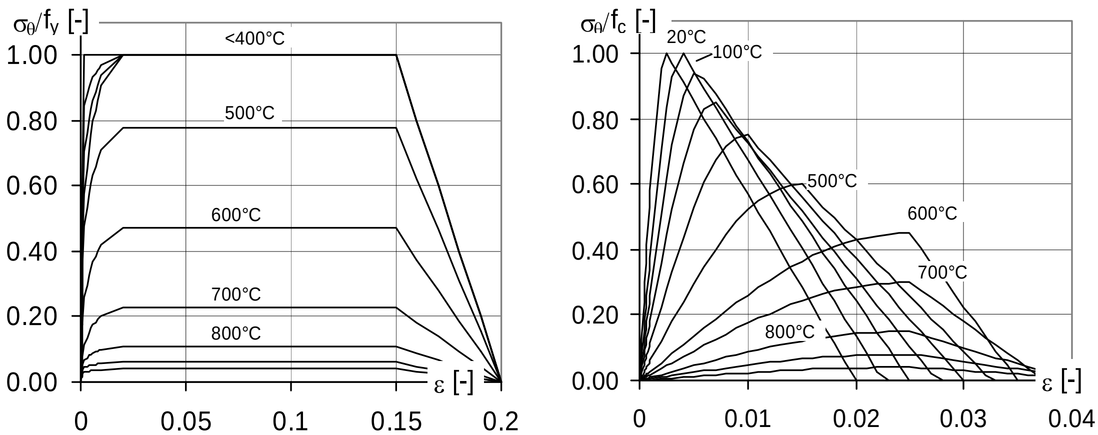


Figure 7 Stress-strain relationship of steel (left) and concrete (right) at elevated temperatures given in EN 1994-1-2 [9]

The stresses have to be incorporated to obtain the values of the interaction curve. They are distributed over the cross section on the basis of the Bernoulli hypothesis for plain state of strains. For normal temperature design a full plastic stress distribution can be supposed. With this assumption the internal forces can be calculated as a numerical integration over the cross section as follows:

$$N = \sum_i f_{y,i} \cdot A_{a,i} + \sum_i f_{c,i} \cdot A_{c,i} \tag{10}$$

$$M = \sum_i f_{y,i} \cdot A_{a,i} \cdot z_i + \sum_i f_{c,i} \cdot A_{c,i} \cdot z_i \tag{11}$$

where

$f_{y,i}$, $f_{c,i}$	maximum stress level of steel and concrete
A_i	elemental area
z_i	lever arm of the elemental area

Each assumption for a plastic neutral axis in the cross section results in one pair of values N and M of the interaction curve.

The adaptation of (10) and (11) to the fire situation results in the following expressions:

$$N_\theta = \sum_i k_y(\theta) \cdot f_{y,i} \cdot A_{a,i} + \sum_i k_c(\theta) \cdot f_{c,i} \cdot A_{c,i} \tag{12}$$

$$M_\theta = \sum_i k_y(\theta) \cdot f_{y,i} \cdot A_{a,i} \cdot z_i + \sum_i k_c(\theta) \cdot f_{c,i} \cdot A_{c,i} \cdot z_i \tag{13}$$

where

k_y, k_c	temperature dependent reduction factors for the maximum stress level of steel and concrete

A more general formulation for the interaction relationship is the strain dependent analysis. On the assumption of the Bernoulli hypothesis the stresses are integrated for a determined plain strain ε_0 and a curvature κ (see Figure 11). With

$$\varepsilon_i = \varepsilon_0 + \kappa \cdot z_i \tag{14}$$

the inner forces can be calculated as follows:

$$N_\theta(\varepsilon_0, \kappa) = \sum_i \sigma_{y,i}(\varepsilon, \theta) \cdot A_{a,i} + \sum_i \sigma_{c,i}(\varepsilon, \theta) \cdot A_{c,i} \tag{15}$$

$$M_\theta(\varepsilon_0, \kappa) = \sum_i \sigma_{y,i}(\varepsilon, \theta) \cdot A_{a,i} \cdot z_i + \sum_i \sigma_{c,i}(\varepsilon, \theta) \cdot A_{c,i} \cdot z_i \tag{16}$$

where:

σ_y, σ_c	temperature dependent stresses of steel and concrete as illustrated in Figure 7

INVESTIGATED CROSS SECTIONS

Following the numerical background presented above, two different cross sections will be analysed, a concrete filled hollow section with embedded massive X-shape profile (CHM) and a concrete encased HEA-section as shown in Figure 8.

cross section A	cross section B

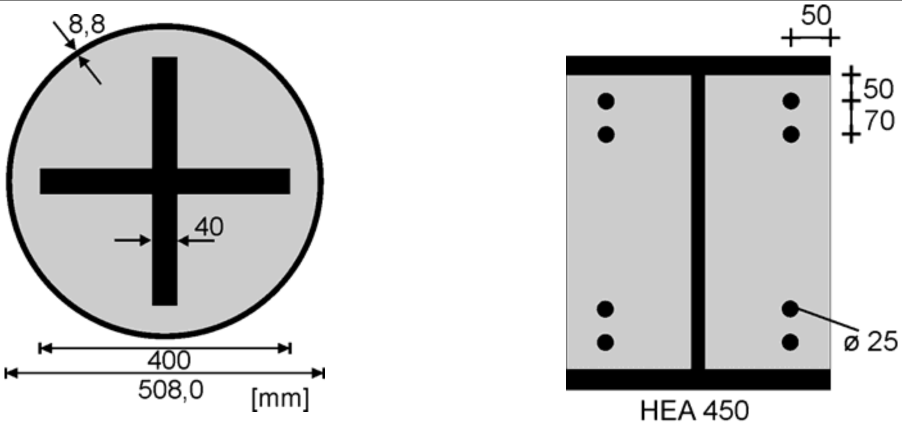


Figure 8 Investigated cross sections

HEATING OF THE CROSS SECTION

Basis of further mechanical analysis is the temperature distribution over the cross section calculated for defined period of fire exposure. Figure 9 shows the results of a 30, 60, 90 and 120 minutes temperature calculation under ISO-fire for cross section A (left) and B (right).

STRAIN DEPENDENT INTERACTION CURVES

Mechanical material properties under fire conditions

Figure 10 shows a comparison of the stress-strain relationship for steel and concrete as given in EN 1994-1-2 (see also Figure 7). The stress values are related to the maximum stress level at room temperature. It is conspicuous, that the maximum stress level for the different materials and for different temperatures is not reached at the same strain value. As the considered cross section is a heated composite section with different materials and different temperatures it is not sufficient to consider only the maximum stress level, when calculating the limiting internal forces (interaction curve) as given in equation (12) and (13). It is necessary to take into account the actual state of strains.

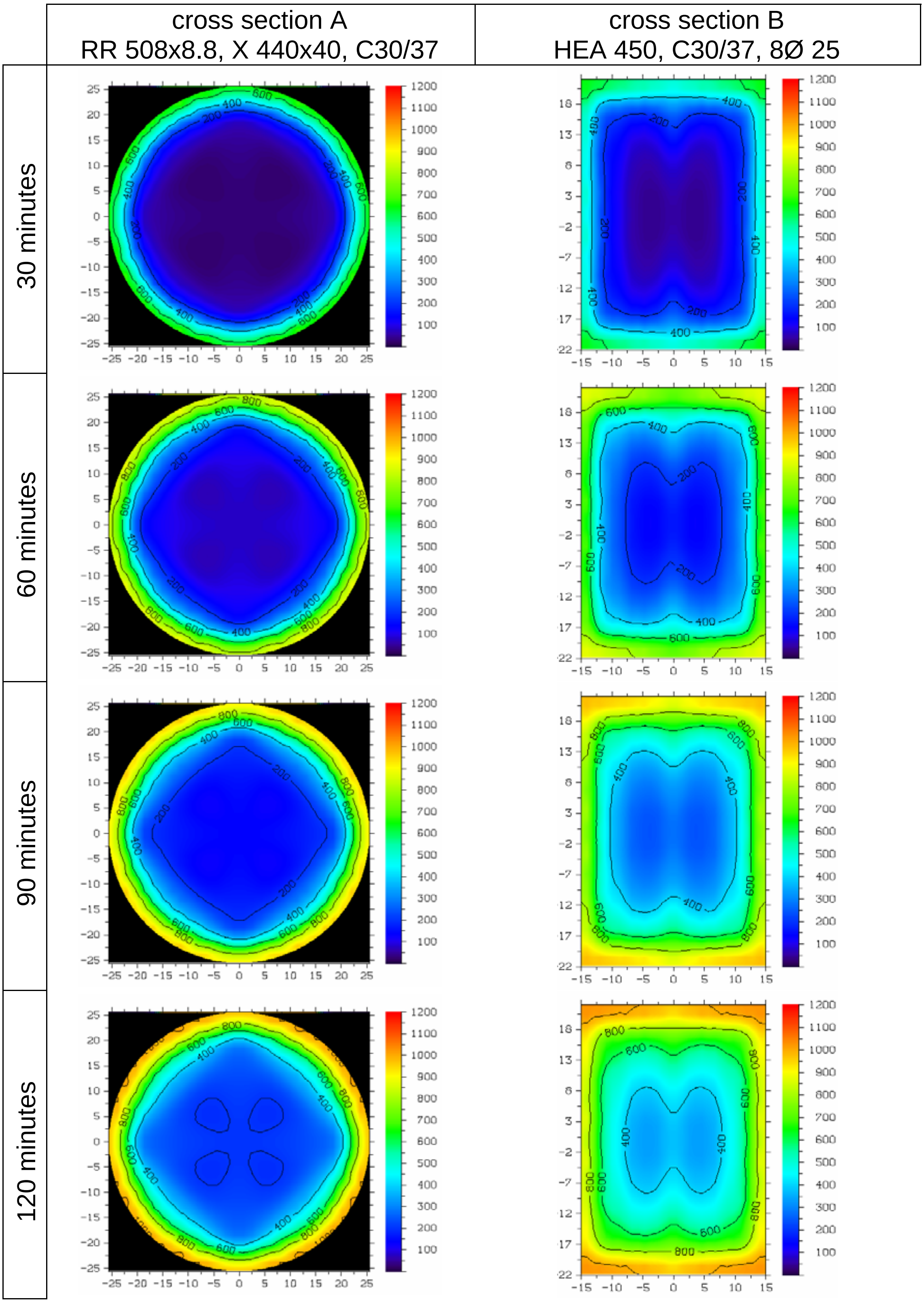


Figure 9 Heating of the cross sections

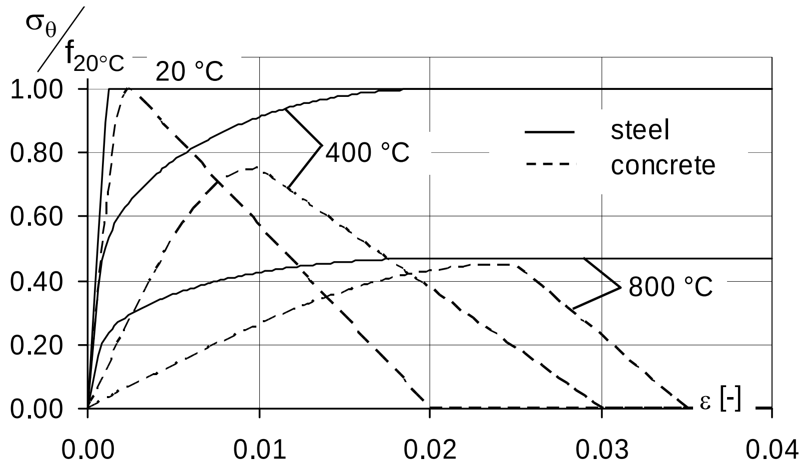


Figure 10 Comparison of the stress-strain relationship of steel and concrete at elevated temperatures

Strain dependent interaction curves

For a calculated temperature distribution at a given fire duration period as given in Figure 9 the inner normal force can be integrated over the cross section for certain plain strains ε_0 from equation (15). The result for compression strains is shown on the left side of Figure 11 without any curvature at the cross section (κ = 0). The maximum normal force that is reached is smaller than the perfect plastic normal force calculated with the maximum stress levels. This is because of the descending stress-strain-relationship of the concrete. While parts of the cross section have not reached the maximum stress level, others are already on the descending part of the stress-strain relationship. With increasing strains larger areas of concrete exceed the maximum stress level $f_{c,\theta}$ and the curve converges to the inner normal force of the pure steel section given by the maximum stress level of steel.

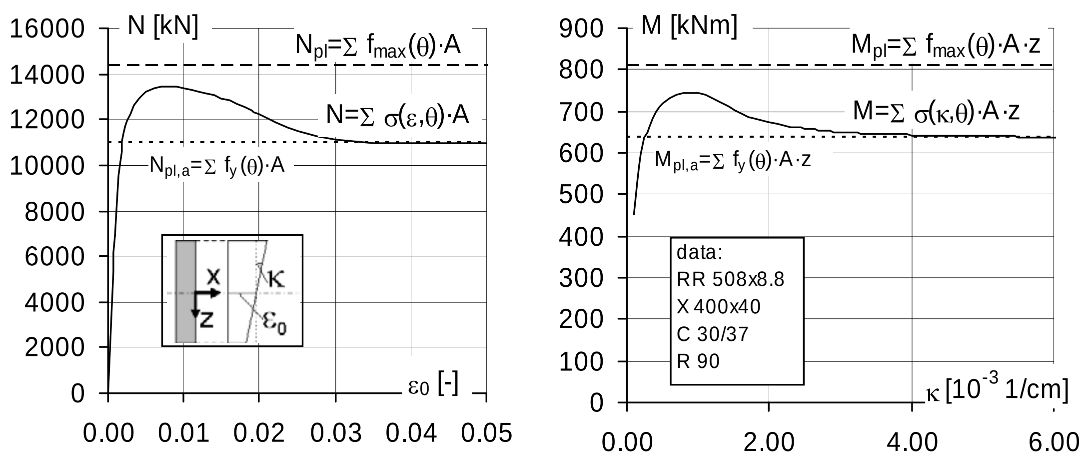


Figure 11 Integrated inner forces: axial normal force in relation to a plain strain ε_0 (left), bending moment in relation to the curvature κ (right)

The same phenomenon is observed for a given curvature κ and the plastic moment resistance as shown on the right side of Figure 11. Again after exceeding the maximum stress level of concrete the values converge to the inner bending moment of the pure steel section.

The integration of different strain distributions over the cross section leads to an array of curves for the normal force N_θ and the bending moment M_θ. Figure 12 shows the results for the hollow profile (cross section A) on the left side and for the HEA (cross section) profile on the right side. In this diagram each dotted line is assigned to a constant value of the plain strain ε_0 and varying values of the curvature κ. The thick line is the envelope of this array of curves and describes a strain dependent interaction curve for the presented CHM cross section. The perfect plastic interaction curve as given through (12) and (13) with the maximum stress level of steel and concrete is plotted with the dashed line.

Further investigation showed that the consideration of thermal stresses has no influence on the development of the interaction curve.

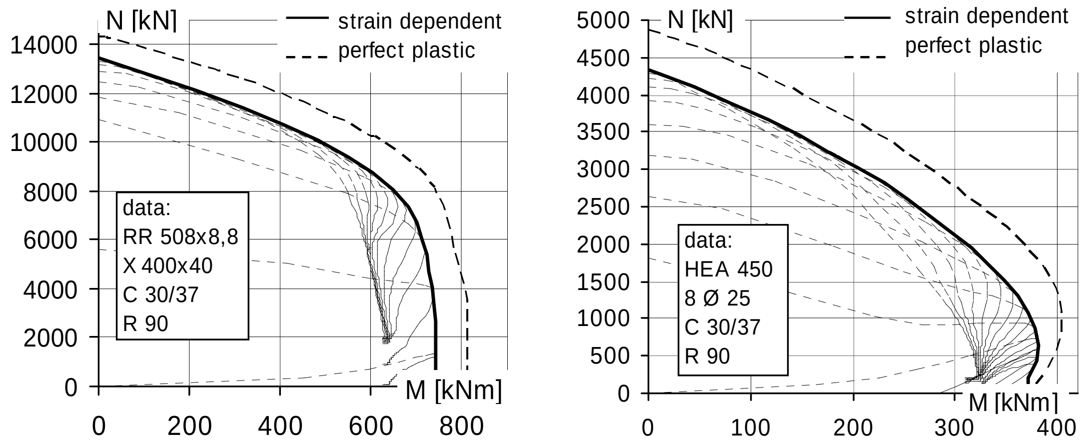


Figure 12 Strain dependent and perfect plastic interaction curve at elevated temperature for a: Concrete filled tubular hollow section with embedded X-section (left)
I-section with concrete encasement (right)

Significance of strain dependent interaction curves for member verification

Compared to the perfect plastic interaction curve defined by the maximum stress level (Figure 12, dashed line) the limiting inner forces given by the strain dependent relationship are obviously smaller. Particularly for members with a high degree of utilization for the normal force the bending moment capacity reacts sensitively to the reduction of the interaction curve. This is important for the verification of more compact members without stability phenomena.

CONCLUSION

Within the EN-Version of Eurocode 4-1-1 a new design procedure for normal temperature design of composite columns was implemented, which allows a more general analysis of compression members. The method was presented in this contribution and compared to the older procedure of the ENV-version. The existing design procedure for the fire design of composite columns given in the Eurocode is only valid for single member verification and is not applicable for global structural analysis. Furthermore it is valid for compression members only in braced frames.

It is the intention to adapt this verification method of Eurocode 4-1-1 for normal temperature design to the fire situation to reach a wider field of application. Special focus was on the interaction relationship between the internal axial normal force and the corresponding bending moment at elevated temperatures. The comparison between the material properties of steel and concrete showed an inconsistency of the strains. An analysis of strain dependent interaction curves results in smaller values than the perfect plastic interaction curve given by the maximum stress level of steel and concrete. Hence, this phenomenon has to be considered for verification with interaction curves in the fire situation.

Future investigations concentrate on the effect of strain dependent interaction curves on different cross sections. Furthermore analysis is necessary concerning the bending stiffness of compression members under fire conditions. The influence of thermal stresses on the bending stiffness has to be determined and values for imperfection and bending stiffness for the elastic second order theory calculation have to be evaluated to reach conservative load bearing values. Finally the calculated values have to be compared to results from fire tests and numerical simulations.

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