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Composite Columns

Colonnes mixtes

Verbundstützen

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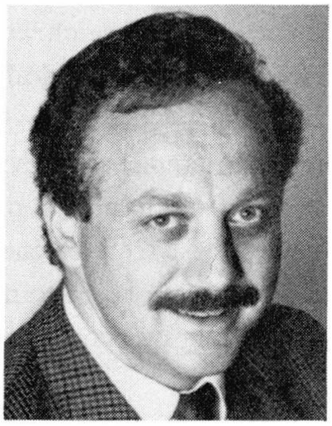


Reinhard Bergmann, born 1947, received his civil engineering degree at the University of Bochum, Germany. Since 1974 he is involved in research work on composite constructions especially composite columns. He is working at the University of Bochum in the field of steel and composite structures.

SUMMARY

This paper deals with the design of composite columns given in Eurocode 4. The main topic is the simplified method which should allow a design without the aid of a computer. Some background information is given. At least the proposed method for composite columns with monosymmetrical sections is shortly presented.

RÉSUMÉ

Ce rapport décrit la méthode du dimensionnement des colonnes mixtes selon l'Eurocode 4. En particulier les détails de la méthode simplifiée du dimensionnement sont montrés. Cette méthode doit rendre possible le dimensionnement des colonnes mixtes sans aide d'un ordinateur. Quelques détails sont donnés afin d'expliquer la méthode. La méthode simplifiée du dimensionnement des colonnes avec sections uni-symétriques est présentée.

ZUSAMMENFASSUNG

Der Aufsatz beschreibt die Bemessungsmethode für Verbundstützen nach Eurocode 4. Insbesondere wird auf die vereinfachte Methode eingegangen, die eine Bemessung von Verbundstützen ohne die Hilfe eines Computers ermöglichen soll. Einige Hintergrundinformationen werden zur Erläuterung angegeben. Schliesslich wird die vereinfachte Bemessungsmethode für Verbundstützen mit einfach symmetrischen Querschnitten kurz vorgestellt.

1. INTRODUCTION

In the sixties intensive research work on the problem of the load bearing capacity of columns in which a concrete cross section and a steel profile act together started off. These columns could neither be designed with the rules for steel construction nor with those for concrete structures. The results of this research work had been described in various publications.

In 1977 the recommendations for composite columns were published /1/, which were introduced together with the results of further research work into the draft of Eurocode 4 /2/ for composite constructions, in which the analysis and design of composite columns are ruled.

In Eurocode 4 first of all the general requirements on the design of composite columns are defined. All geometrical and physical non-linearities of the different materials shall be observed. Yet it is only possible to meet these requirements in verification by large numerical methods of analysis, which can generally only be performed with a vast EDP-program. For practical use a simplified method is given in Eurocode 4 which is presented in the following.

2. TYPES OF CROSS SECTIONS OF COMPOSITE COLUMNS

Fig. 1 shows typical cross sections of composite columns with the notations of Eurocode 4. They are only examples for the great variety of possible cross sections. The sections can be classified in two groups:
- concrete filled sections for which the concrete cannot be seen at the surface and
- totally and partly encased sections.

All cross sections are symmetrical about both axes and can additionally be reinforced.

The use of composite columns provides various advantages. With small dimensions of the cross section a high load bearing capacity can be achieved. On the other hand sections with different load bearing capacities but identical dimensions can be produced because of the great variability. Thus the outer dimension of a column can be held constant over a lot of floors in a building, what reduces the planning work. The economic efficiency also results from the combination with the low-priced material concrete. Moreover the highly developped connecting technique of steel construction can be applied.

With concrete filled profiles (fig. 1d-f) the steel section serves at the same time as casing for the concrete. Concrete filled sections provide the opportunity to set up a building as a mere steel construction and to fill the cross sections later with concrete, for example by pressing in the concrete. In doing so the time of erection can be reduced.

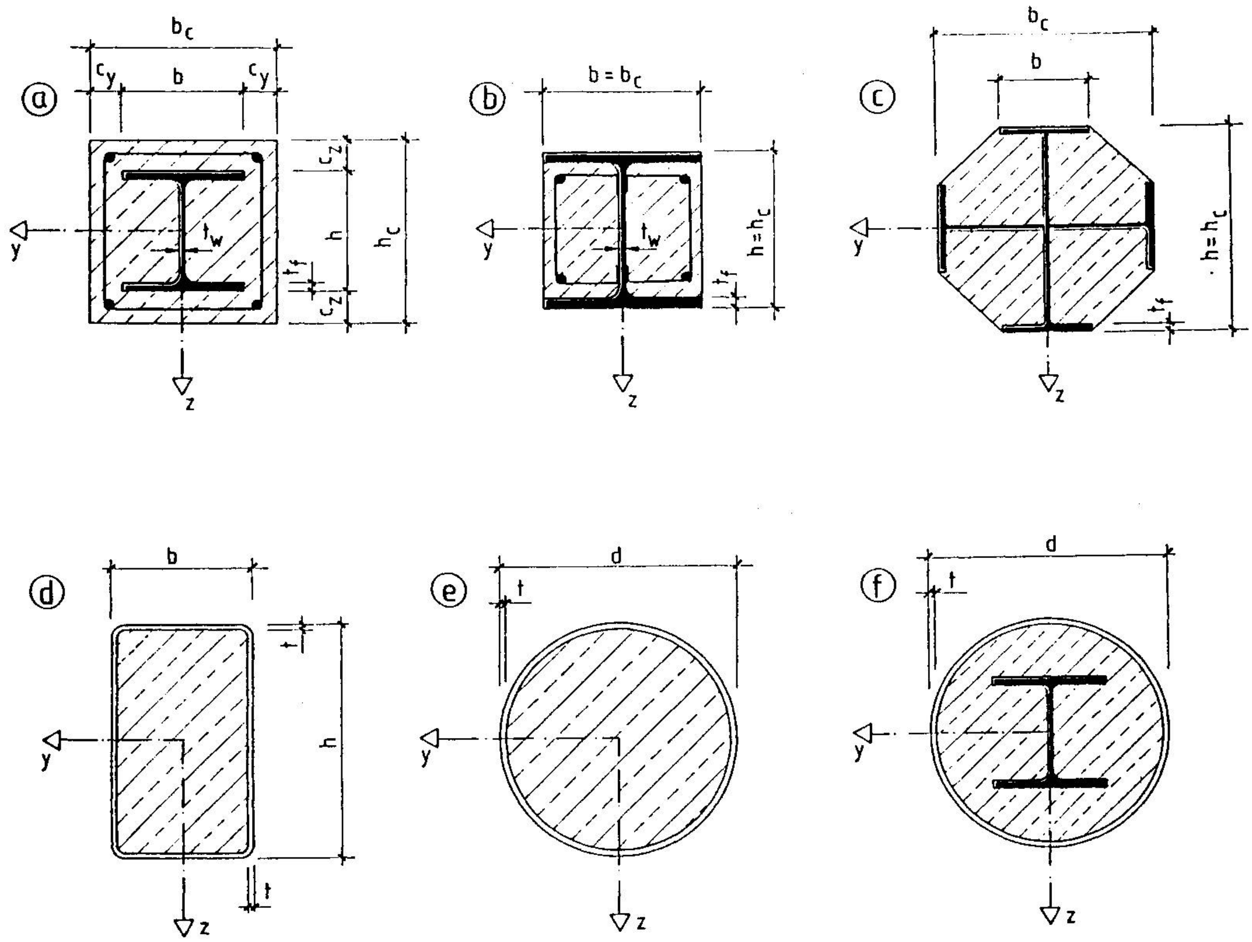


Fig. 1: Typical cross sections of composite columns with notations

The protective steel mantle further more allows the assumption of a higher design strength for the concrete. For concrete filled circular hollow profiles the effect of confinement additionally leads to an increase in the load bearing capacity. The influence of creep and shrinkage of the concrete can usually be neglected. This influence must be considered for concrete encased profiles (fig. 1a-c).

Due to the complete encasement by concrete the sections (fig. 1a) generally fulfill the technical requirements for high classes of fire protection without any additional measures. For partly encased sections as well as for concrete filled sections (fig. 1b+c) this can be achieved by additional reinforcement. Partly encased sections have further advantages. First they can be produced quite simply. The openings of the column can be filled with concrete in a horizontal position of the steel section. 24 hours later the column may be turned around and further more concrete filled. Casing is provided by the steel profile. For sections according to fig. 1b dropping out of the concrete while turning the column must be avoided by constructive means (headed stud connectors or similar). Secondly partial steel regions of the surface allow to weld on connective devices after concreting.

3. MATERIAL GRADES AND MATERIAL SAFETIES

For composite columns structural steel according to Eurocode 3 /3/ and concrete and reinforcement according to Eurocode 2 /4/ may be used.

For the common steel grades the nominal values of strength are given in table 1. They are valid for material thicknesses not greater than 40 mm. For higher material thicknesses the strengths must be reduced according to Eurocode 3 /3/.

steel grades (old notation)	Fe E 275 (St37)	Fe E 275 (St44)	Fe E 355 (St52)
f_y [N/mm^2]	235	275	355
E_a [kN/mm^2]	210	210	210

Table 1: Nominal values of strength f_y and moduli of stiffness for common types of structural steel according to Eurocode 4 for material thicknesses not greater than 40 mm; the specifications in brackets refer to the former classifications.

For the concrete the strengths of the different concrete grades according to Eurocode 2 /4/ are given in table 2. The classification C25/30 gives the cylinder strength (25) and the cube strength (30).

concrete grades	C20/25	C25/30	C30/37	C35/45	C40/50	C45/55	C50/60
f_{ck} [N/mm^2]	20	25	30	35	40	45	50
E_{cm} [kN/mm^2]	29	30.5	32	33.5	35	36	37

Table 2: Characteristic cylinder strength f_{ck} and mean values of the secant modulus E_{cm} for the different concrete grades according to Eurocode 2

Eurocode 2 mentiones 3 different classes for reinforcing steel which are given in table 3.

reinforcing steel grades	S 220	S 420	S 500
f_{sk} [N/mm^2]	220	420	500
E_s [kN/mm^2]	200	200	200

Table 3: Characteristic strengths f_{sk} and moduli of stiffness E_s for reinforcing steel according to Eurocode 2

The design has to show that the design values of the action effects Sd is not greater than the design resistance R_d according to equation 1.

$$S_d \leq R_d = \left[\frac{f_y}{\gamma_{Ma}}, \frac{f_{ck}}{\gamma_c}, \frac{f_{sk}}{\gamma_s} \right] \tag{1}$$

with the partial safety factors

- for the concrete $\gamma_c = 1.5$ (2)

- for the reinforcing steel $\gamma_s = 1.15$ (3)

- for the structural steel $\gamma_{Ma} = \gamma_{Rd} = 1.1$ for columns influenced by buckling (4)

  $\gamma_{Ma} = \gamma_a = 1.0$ for all other columns (5)

The design strengths for structural steel, concrete and reinforcement result from:

$$f_{yd} = f_y / \gamma_{Ma} \; ; \quad f_{cd} = f_{ck} / \gamma_c \; ; \quad f_{sd} = f_{sk} / \gamma_s \tag{6}$$

For considering the influence of long term load effects on the compressive strength, the concrete strength must be reduced by the factor $\alpha = 0.85$.

This reduction can be neglected for concrete filled composite sections ($\alpha = 1.0$) since the concrete experiences a better development of strength due to the complete shelter against the air, and moreover splitting is prevented.

4. LOCAL BUCKLING FAILURE

In the ultimate limit state the attainment of the material strength is assumed for all parts of the section. It must be ensured that this is possible without previous failure in stability of thin cross section parts.

This can be verified for steel parts at the surface by regarding the limit ratio of wall dimension to wall thickness. With the notations of fig.1 result the following limit ratios:

- for concrete filled circular tubes (fig. 1e+f) $d/t \leq 90\ \varepsilon^2$ (7)
- for concrete filled rectangular hollow profiles (fig. 1d) $h/t \leq 52\ \varepsilon$ (8)
  with h = larger side of the profile
- for partly encased I-sections (fig. 1b+c) $b/t_f \leq 44\varepsilon$ (9)

The factor ε accounts for different yield limits.

$$\varepsilon = \sqrt{\frac{235}{f_y}} \qquad (10)$$

with f_y in N/mm^2 units.

For the structural steel grades given in table 1 thus result the limit values of table 4:

steel grade		Fe E 235	Fe E 275	Fe E 355
concrete filled circular tubes	lim d/t	90	77	60
concrete filled rectangular hollow profiles	lim h/t	52	42	42
partially encased I-profiles	lim b/t_f	44	41	36

Table 4: Limit ratios of wall dimension to wall thickness for which local buckling is prevented

For completely encased steel parts verification for local buckling is not necessary. For larger steel parts (for ex. flanges in fig.1a) a sufficient concrete cover must be provided in order to avoid splitting of the concrete cover.

Therefore the minimum concrete cover of such steel parts may not be less than 40 mm or 1/6 of the dimension of the steel part. For cross sections according to fig. 1a follows:

$$40\ mm \leq c_z \geq b/6 \qquad (11)$$

If steel parts at the outer side should exceed the values given in table 4, special methods of analysis would have to be applied. The design according to the simplified method of Eurocode 4 presented herein is no more possible.

5. DESIGN FOR AXIAL COMPRESSION

5.1 Resistance of cross sections

The plastic resistance of the cross section of a composite column is given by the sum of the components:

$$N_{pl.Rd} = A_a f_{yd} + A_c \alpha f_{cd} + A_s f_{sd} \tag{12}$$

where

A_a, A_c and A_s	are the areas of the structural steel, the concrete and the reinforcement,
f_{yd}, f_{cd} and f_{sd}	are the above described design strengths of the materials and
α	is 1.0 for concrete filled cross sections and 0.85 in other cases

Fig. 2 shows the stress distribution on which eqn. 12 is based:

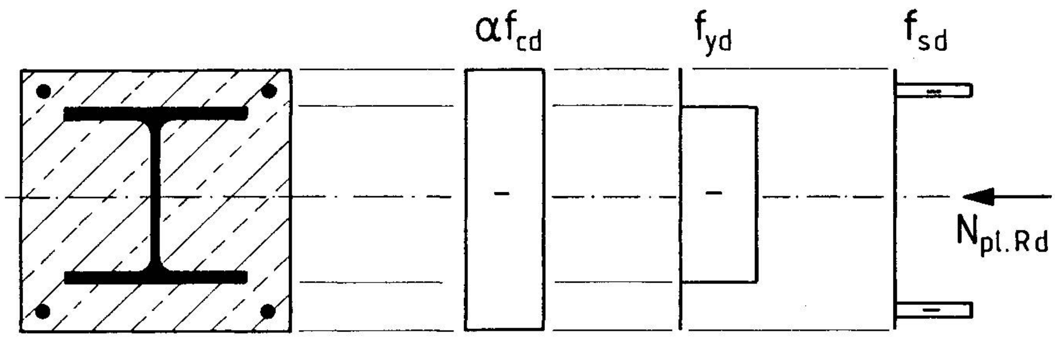


Figure 2: Stress distribution for the plastic resistance of the section for the example of an encased I-profile

For concrete filled circular hollow profiles the increased load bearing capacity of the concrete due to the effect of confinement of the steel tube may be taken into account.

Due to the impeded transverse strain of the concrete results a three-dimensional stress distribution in the concrete. By this means the bearing capacity of normal stress is increased. At the same time result circular tensile stresses in the tube which reduce its normal stress capacity in the tube.

This effect may only be considered up to a relative slenderness of $\bar{\lambda} \leq 0.5$. In addition the eccentricity of the normal force e may not exceed the value d/10, d being the outer dimension of the tube.

The eccentricity e is defined by:

$$e = \frac{M_{max.Sd}}{N_{Sd}} \tag{13}$$

with

$M_{max.Sd}$	maximum design moment from the loads according to 1^{st} order theory
N_{Sd}	design normal force

The plastic normal force for these cross sections may be determined from:

with

t	wall thickness of the circular hollow profile

$$N_{pl.Rd} = A_a f_{yd} \eta_2 + A_c f_{cd} \left(1 + \eta_1 \frac{t}{d} \frac{f_y}{f_{ck}}\right) + A_s f_{sd} \tag{14}$$

By means of the values

$$\eta_1 = \eta_{10} \left(1 - \frac{10\ e}{d}\right) \tag{15}$$

$$\eta_2 = \eta_{20} + (1 - \eta_{20}) \frac{10\ e}{d} \tag{16}$$

a linear interpolation is carried out for load eccentricities $e \leq d/10$ with the basic values μ_{10} and η_{20} which depend on the relative slenderness $\bar{\lambda}$:

$$\eta_{10} = 4.9 - 18.5\ \bar{\lambda} + 17\ \bar{\lambda}^2 \quad \text{but} \quad \eta_{10} \geq 0.0 \tag{17}$$

$$\eta_{20} = 0.25\ (3 + 2\ \bar{\lambda}) \quad \text{but} \quad \eta_{20} \leq 1.0 \tag{18}$$

Table 5 gives the basic values η_{10} and η_{20} for different values of $\bar{\lambda}$.

$\bar{\lambda}$	0.0	0.1	0.2	0.3	0.4	0.5
η_{10}	4.90	3.22	1.88	0.88	0.22	0.00
η_{20}	0.75	0.80	0.85	0.90	0.95	1.00

Table 5: Basic values η_{10} and η_{20} for considering the effect of confinement for concrete filled circular tubes

If the eccentricity e exceeds the value d/10 or the relative slenderness $\bar{\lambda}$ the value 0.5 it must be applied $\eta_1 = 0.0$ and $\eta_2 = 1.0$.

5.2 Resistance of members

For each of the main bending axes of the column it has to be verified that

$$N_{Sd} \le \chi\, N_{pl.Rd} \tag{19}$$

where:

$N_{pl.Rd}$ is the cross section resistance for axial load (cf. chap. 5)
χ is the reduction factor of the respective buckling curve

Fig. 3 shows the European buckling curves for composite columns:

curve a for concrete filled hollow profiles

curve b for partially and completely concrete encased I-profiles with bending about the strong axis of the steel profile

curve c for partially and completely concrete encased I-profiles with bending about the weak axis of the steel profile

These curves can also be described in form of equation:

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \tag{20}$$

with

$$\Phi = 0.5 \cdot [\, 1 + \alpha\, (\, \bar{\lambda} - 0.2\,) + \bar{\lambda}^2\,] \tag{21}$$

The factor α in eqn. 21 accounts for the different assumptions of imperfections for the cross sections, for the different buckling curves respectively (Table 6). The reduction factors χ can also be read immediately off table 7 and intermediate factors can be determined by interpolation.

European buckling curve	a	b	c
faktor of imperfection α	0.21	0.34	0.49

Table 6: Factor of imperfection α for the buckling curves according to Eurocode 3

$\bar{\lambda}$	χ for buckling curve		
	a	b	c
0.0	1.0000	1.0000	1.0000
0.2	1.0000	1.0000	1.0000
0.3	0.9775	0.9641	0.9491
0.4	0.9528	0.9261	0.8973
0.5	0.9243	0.8842	0.8430
0.6	0.8900	0.8371	0.7854
0.7	0.8477	0.7837	0.7247
0.8	0.7957	0.7245	0.6622
0.9	0.7339	0.6612	0.5998
1.0	0.6656	0.5970	0.5399
1.1	0.5960	0.5352	0.4842
1.2	0.5300	0.4781	0.4338
1.3	0.4703	0.4269	0.3888
1.4	0.4179	0.3817	0.3492
1.5	0.3724	0.3422	0.3145
1.6	0.3332	0.3079	0.2842
1.7	0.2994	0.2781	0.2577
1.8	0.2702	0.2521	0.2345
1.9	0.2449	0.2294	0.2141
2.0	0.2229	0.2095	0.1962

Table 7: Reduction factor χ of the European buckling curves according to /3/

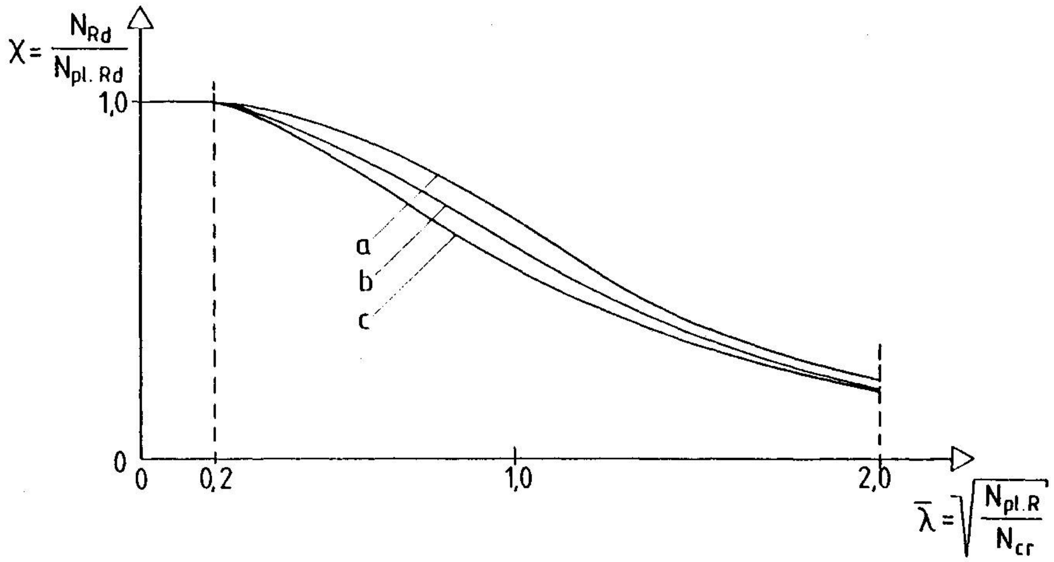


Fig. 3: European buckling curves according to Eurocode 3

5.3 Relative slenderness and stiffnesses

The relaive slenderness for the determination of the load bearing capacity of a column under axial load results from:

$$\bar{\lambda} = \sqrt{\frac{N_{pl.R}}{N_{cr}}} \qquad (22)$$

where

$N_{pl.R}$ is the cross section resistance for normal force according to eqn. 12, eqn. 14 respectively with $\gamma_a = \gamma_c = \gamma_s = 1.0$ and

N_{cr} is the elastic buckling load of a column.

$$N_{cr} = \frac{(EI)_e \, \pi^2}{l^2} \qquad (23)$$

with

$(EI)_e$ effective bending stiffness and

l buckling length of the column.

The buckling length of a column can be determined according to Eurocode 3. For isolated columns of non-sway systems the column length may be applied for the buckling length.

The effective bending stiffness is determined in a similar way as the plastic resistance to normal force of the cross section by adding up the single components:

$$(EI)_e = E_a I_a + 0.8\, E_{cd} I_c + E_s I_s \tag{24}$$

where

I_a, I_c and I_s are the moments of inertia for the areas of structural steel, concrete (here assumed as uncracked) and the reinforcement for the regarded bending axis,
E_a and E_s are the moduli of stiffness of structural steel and reinforcement, and
$0.8\, E_{cd} I_c$ is the effective bending stiffness of the concrete part.

$$E_{cd} = E_{cm} / \gamma_c \tag{25}$$

with

E_{cm} secant modulus of the concrete according to Eurocode 2, table 2 respectively.

The material safety γ_c can be reduced for the determination of the effective bending stiffness to $\gamma_c = 1.35$ according to Eurocode 2.

For slender columns the influence of long-term behaviour of the concrete (creep and shrinkage) on the load bearing capacity has to be considered.

If the normal force eccentricity according to eqn. 13 is more than the double value of the respective cross section dimension, creep and shrinkage need not be considered. This is also valid, if the relative slenderness $\bar{\lambda}$ is lesser than the limit value of table 8.

The influence of creep and shrinkage can be taken into account by a simple modification of the modulus of stiffness of the concrete from E_{cd} to E_c:

$$E_c = E_{cd} \left(1 - 0.5 \frac{N_{G.Sd}}{N_{Sd}} \right) \tag{26}$$

where

N_{Sd} is the design normal force and
$N_{G.Sd}$ is the permanently acting part of it.

	braced and non-sway systems	unbraced and sway systems
concrete encased cross sections	0.8	0.5
concrete filled cross sections	$\frac{0.8}{1-\delta}$	$\frac{0.5}{1-\delta}$

Table 8: Limit values $\bar{\lambda}$ for considering creep and shrinkage

The factor δ represents the contribution of the structural steel in the plastic normal force bearing capacity. For concrete filled cross sections the limit values are only applied for the concrete part $(1 - \delta)$.

$$\delta = \frac{A_a f_{yd}}{N_{pl.Rd}} \tag{27}$$

6. DESIGN FOR COMPRESSION AND BENDING

6.1. General

The design for compression and bending is done in the following steps:
The composite column is examined isolated from the system. In doing so the end moments which may result from the analysis of the system as a whole are taken up. These end moments may also have been determined by second order theory in the analysis of the whole system according to the respective requirements. With the end moments and possible horizontal forces within the column length, as well as with the normal force, action effects are determined. For slender columns this must be done considering second order effects. In the simplified method of Eurocode 4 imperfections need not be considered in the analysis of action effects for the column. They are taken into account in the determination of the resistance.

The resistance of the column to compression and bending is determined by help of the cross section interaction curve. The influence of shear forces may be considered in the interaction curve.

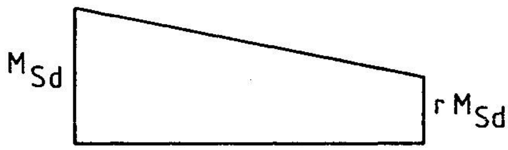
6.2 Analysis for bending moments

Analysis for bending moments according to second order theory may be neglected for braced and non-sway systems, if for the slenderness $\bar{\lambda}$ of the column counts:

$$\bar{\lambda} \leq 0.2\,(2 - r) \tag{28}$$

where

r is the ratio of smaller to larger end moment (fig. 4)



$-1 \leq r \leq +1$

Fig. 4: Ratio r of the end moments

For transverse loading within the column length r = 1.

The flexural stiffness, which is necessary for the analysis following second order theory, can be determined by eqn. 24.

In a simplified way, the moment according to second order theory can be calculated by multiplying the maximum first order bending moment with a factor k:

$$k = \frac{\beta}{1 - \dfrac{N_{Sd}}{N_{cr}}} \geq 1.0 \tag{29}$$

where

N_{Sd} is the design normal force
N_{cr} is the critical load according to eqn. 23 with l as the length of the column and
β is the moment factor according to eqn. 30.

For columns with transverse loading within the column length the value for β must be taken as 1.0. For pure end moments, β can be determined from:

$$\beta = 0.66 + 0.44\,r \qquad \text{but} \qquad \beta \geq 0.44 \tag{30}$$

with r according to fig. 4.

6.3 Design principle

Fig. 59 shows the principle of verification. First of all the bearing capacity of the composite column under axial compression has to be determined according to chapter 5. This bearing capacity is represented by the value χ. At the level of χ a value for the moment μ_k can be read off of the interaction curve, which is meant to be the moment of imperfection of the column.

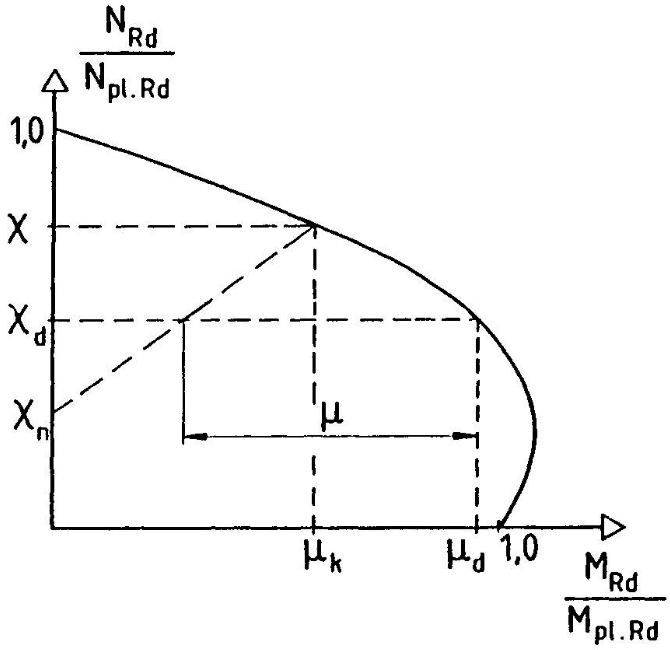


Fig. 5: Design procedure for compression and uniaxial bending

The influence of this imperfection is assumed to decrease linearily to the value χ_n. For an acting normal force $\chi_d = N_{Sd}/N_{pl.Rd}$ remains the moment factor μ for taking up additional bending moments. For the verification eqn. 31 has to be fullfilled.

$$M_{max.Sd} \leq 0.9\mu\ M_{pl.Rd} \tag{31}$$

In certain regions of the interaction curve the normal force increases the bending resistance ($\mu > 1.0$). The value of μ must be limited to 1.0, if bending moment and normal force are independent of each other.

The value χ_n accounts for the fact that the imperfection and the bending moment do not always act unfavourably together. For end moments eqn. 32 may be applied.

$$\chi_n = \chi \frac{1 - r}{4} \tag{32}$$

with r ratio of the end moments according to fig. 4.

If transverse loads occur within the column length, it must be applied $\chi_n = 0.0$.

The reduction of the moment resistance in eqn. 31 by 10% considers some simplifications. The interaction curve of the section has been determined once without considering strain limitations in the concrete, and hence the moment according to second order theory (eqn. 29) had been determined with the effective flexural stiffness $(EI)_e$ with the complete area of the concrete section.

6.4 Interaction curve for combined compression and bending

For the above mentioned verification the cross section interaction curve is needed. It can be found by moving the stress neutral axis over the whole cross section and determining the internal action effects from the resulting stress blocks. This can only be done adequately by a computer program due to the amount of equations to be solved. Yet it is possible to calculate certain points of the interaction curve quite simply without help of a computer. These points (A-E) are marked in the interaction diagram in fig. 6 and connected by a polygonal course. This polygonal interaction curve is sufficiently exact for the design according to fig. 5.

Fig. 7 shows the stress distributions belonging to the polygonal course for the example of a concrete filled rectangular hollow section.

Point A marks the resistance to normal force.

$$N_A = N_{pl.Rd} \tag{33}$$

$$M_A = 0.0 \tag{34}$$

Point B shows the stress distribution for moment resistance.

$$N_B = 0.0 \tag{35}$$

$$M_B = M_{pl.Rd} \tag{36}$$

Here it can be seen, that in the determination of the cross section resistance concrete regions under tension are taken as cracked.

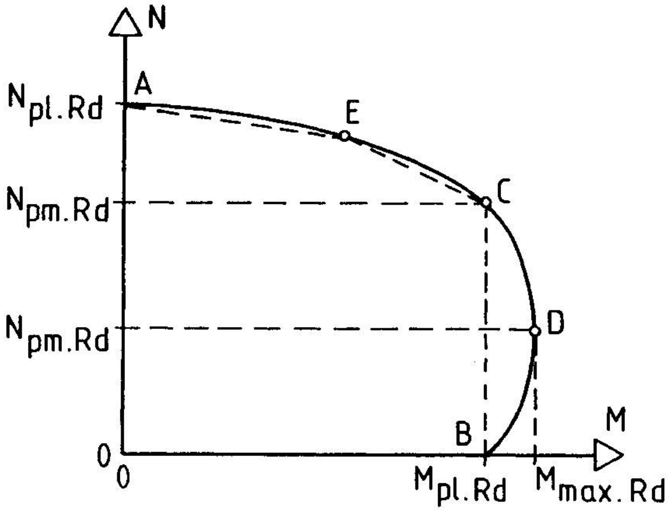


Fig. 6: *Interaction curve with polygonal approximation*

Point C again shows the moment resistance only, since the stress resultants from the additonally pressed parts nullify each other in region of 2 h_n in summarizing the internal moments.

But these additionally pressed regions create an internal normal force. It is the full plastic resistance of the concrete member. This can be comprehended adding up the stress distributions of point B and C in mind, which does not change anything in the normal force balance, since there is no resulting normal force for point B. All steel parts compensate each other and the compression area of the concrete in point B is identical with the concrete tension area of point C.

$$N_C = N_{pm.Rd} = A_c \alpha f_{cd} \tag{37}$$

with

α 1.0 for concrete filled profiles and

f_{cd} design strength of the concrete (eqn. 5)

$$M_C = M_{pl.Rd} \tag{38}$$

In point D the stress neutral axis lies within the central axis of the cross section. Here results the half force of point C. The stress distribution allows a rapid and simple analysis of the respective moment.

$$N_D = N_{pm.Rd} / 2 \tag{39}$$

$$M_D = M_{max.Rd} \tag{40}$$

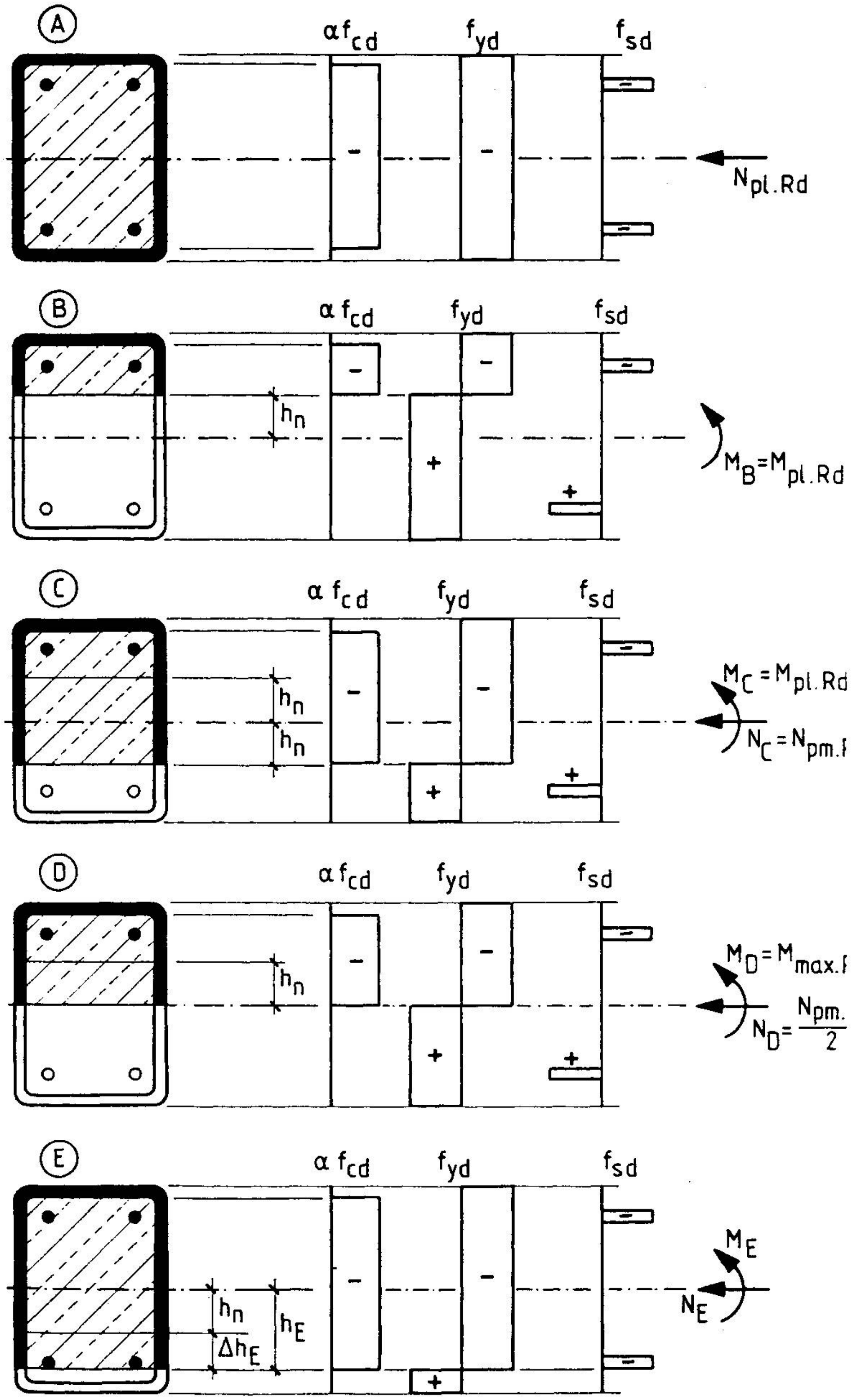


Fig. 7: *Stress distributions for the points of the interaction curve for the example of an concrete filled hollow section*

$$M_{max.Rd} = W_{pa}f_{yd} + \frac{1}{2} W_{pc}\alpha f_{cd} + W_{ps}f_{sd} \tag{41}$$

where

W_{pa}, W_{pc} and W_{ps} are the plastic moments of resistance of the structrual steel, concrete and reinforcement and

f_{yd}, f_{cd} and f_{sd} are the design strengths of the materials.

For point E the stress neutral axis should be placed between that of point C and the edge of the cross section. This should rather be done in such way that the stress resultant can easily be calculated. Point E need not be determined in all cases.

The position of the neutral axis for point B ($M_{pl.Rd}$), that of point C respectively, i.e. the distance h_n, can be determined from the difference of stresses of point C and point B. The stress blocks in region of 2 h_n remain. Since in region of 2 h_n the cross section parts are generally of a rectangular shape, the resulting forces in dependence of h_n can easily be described from that. The sum of these forces is equal to $N_{pm.Rd}$, as shown above. From that follows the equation for h_n. This equation is different for various types of sections.

For the example of a concrete filled ractangular hollow profile chosen herein with fig. 8 results eqn. 42.

$$2\,h_n = \frac{N_{pm.Rd}}{bf_{cd} - 2\,t\,(2\,f_{yd} - f_{cd})} \tag{42}$$

The moment resistance $M_{pl.Rd}$ can simply be calculated from the difference of stresses in point D and point B (fig. 9).

As difference moment results:

$$M_{n.Rd} = W_{pan}f_{yd} + \frac{1}{2} W_{pcn}\alpha f_{cd} + W_{psn}f_{sd} \tag{43}$$

where

W_{pan}, W_{pcn} and W_{psn} are the plastic moments of resistance of the areas in region of 2 h_n

For the moment resistance $M_{pl.Rd}$ results:

$$M_{pl.Rd} = M_{max.Rd} - M_{n.Rd} \tag{44}$$

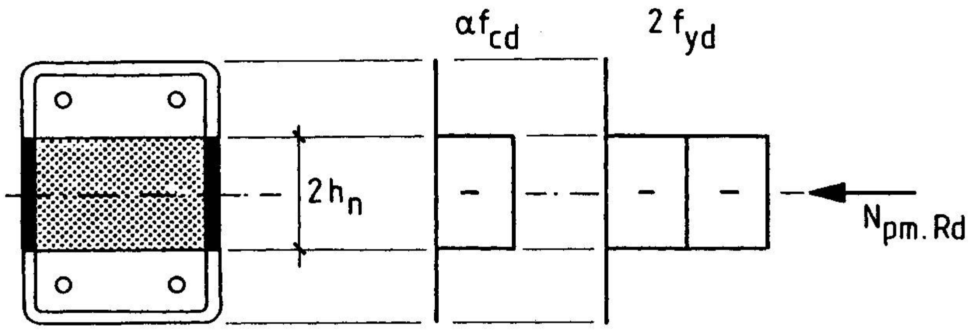


Fig. 8: *Stress difference between point C and point B*

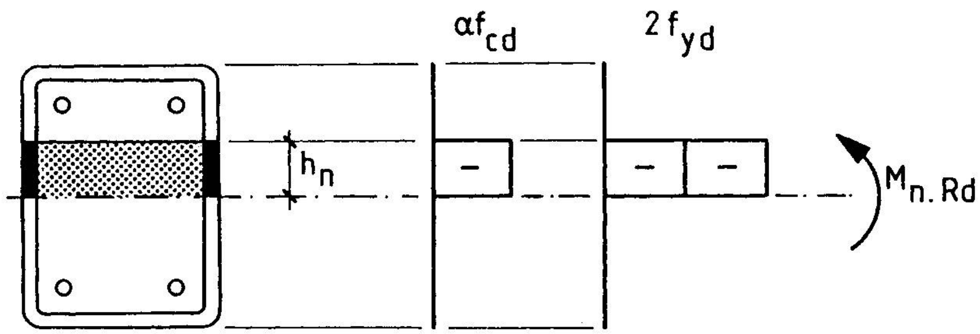


Fig. 9: *Stress difference between point D and point B*

For concrete encased I-profiles and for concrete filled tubes the respective formulae are given in Annex A of Eurocode 4 /5/.

The advantage of this method of calculation is its applicability for any double-symmetrical cross section. Even for joint cross sections (for ex. fig. 1c or f) the characteristic points of the polygonal course can easily be determined.

Although the polygonal course lies beneath the exact interaction curve, the design not always lies on the conservative side.

If the deviation between polygonal course and exact curve is very large in the region of the moment of imperfection (at the height of χ in fig. 5), and small on the other side at the normal force (at χ_d in fig. 5), a too small imperfection is taken into account. In this case the above mentioned point E nearly in the middle between point A and point C has to be determined.

For concrete encased I-sections with bending about the strong axis of the section the exact interaction curve takes an almost linear course between point A and C, so that point E need not be determined in this case.

6.5 Compression and biaxial bending

For the design of a column under compression and biaxial bending first of all verification for each axis has to be performed separately. Thus it is known which of the axes is more endangered to fail. For the combination of both axes the imperfection must only be considered for that axis which is more endangered to fail. For the other axis only the cross section interaction curve is valid.

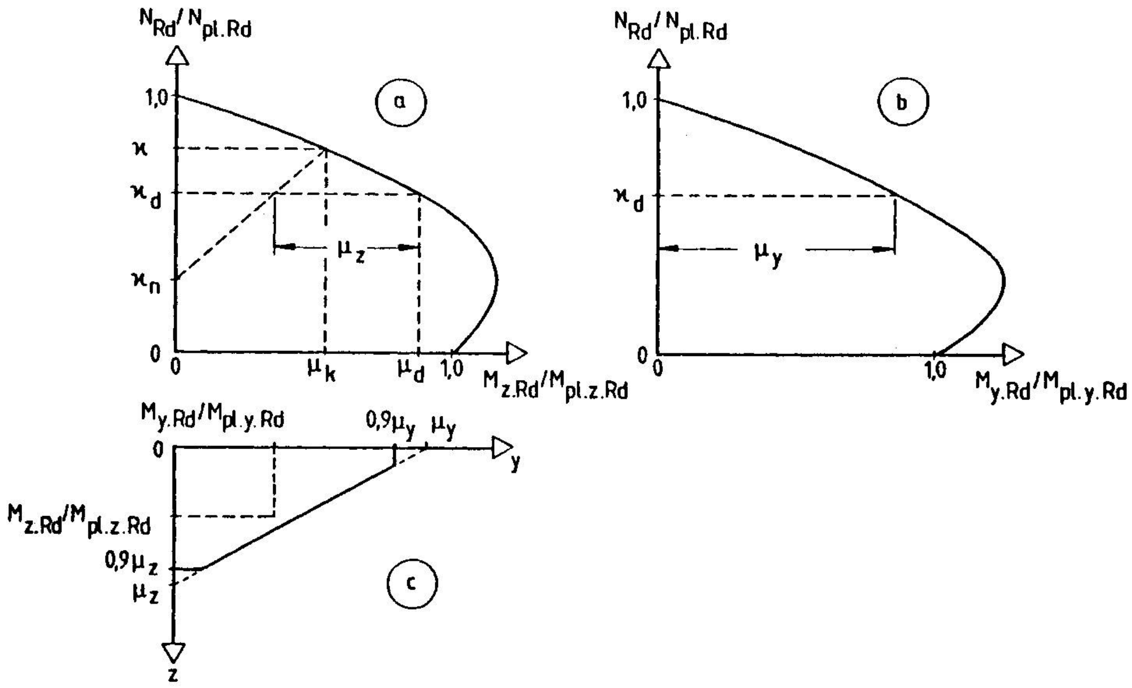


Fig. 10: Verification for compression and biaxial bending

The relative values of moment resistance μ_y and μ_z are used for a new interaction curve. This linear interaction curve (fig. 10c) is cut off at 0.9 μ_y and 0.9 μ_z. The existing moments $M_{y.Sd}$ and $M_{z.Sd}$ related to the respective resistances must lie within the new interaction curve. In form of equation results:

$$\frac{M_{y.Sd}}{\mu_y M_{pl.y.Rd}} + \frac{M_{z.Sd}}{\mu_z M_{pl.z.Rd}} \leq 1.0 \tag{45}$$

and

$$\frac{M_{y.Sd}}{\mu_y M_{pl.y.Rd}} \leq 0.9 \tag{46}$$

$$\frac{M_{z.Sd}}{\mu_z M_{pl.z.Rd}} \leq 0.9 \tag{47}$$

7. SHEAR AND LOAD INTRODUCTION

7.1 Influence of shear forces

It must be ensured that bond stresses between steel profile and concrete do not exceed the following values:

- for completely concrete encased sections — 0.6 N/cm^2
- for concrete filled sections — 0.4 N/cm^2
- for flanges of partially concrete encased sections according to fig. 1b and c — 0.2 N/cm^2
- for the webs of partially concrete filled sections according to fig. 1b and c — 0.0 N/cm^2

An exact determination of bond stresses between structural steel and concrete requires a large calculative effort. In a simplified way composite stresses may be determined according to elastic theory or by help of the plastic resistance of the cross section. The variation of stresses in the concrete member between two critical sections can be used for the determination of bond stresses.

In a similar way the division of the existing shear force in a structural steel part and a concrete part can be made. The share of the concrete in the shear force must be considered in the design for shear in the concrete according to Eurocode 2, whereas the share in shear force of the steel profile can be accounted for, if necessary, by an interactional relation. Fig.11 shows the reduction of normal stresses in the areas which transfer shear stresses. This reduction of the yield stress in the parts which transfer shear can be transformed for engineering practice into a reduction of the thickness.

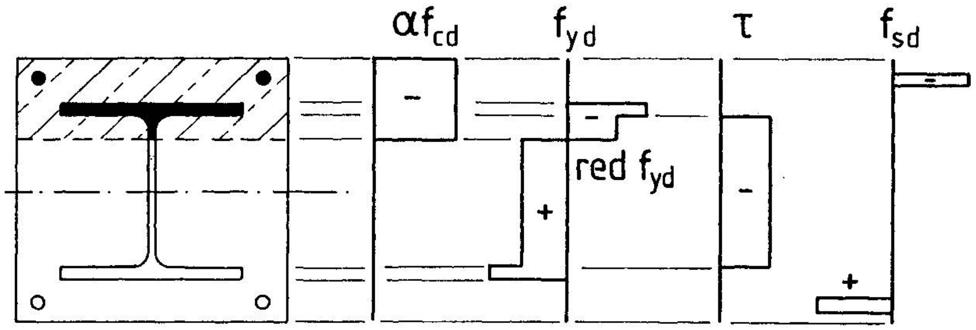


Fig. 11: Reduction of normal stresses by shear stress

The influence of shear must yet only be considered from

$$V_{Sd} > 0.5\ V_{pl.Rd} \tag{48}$$

onwards with

V_{Sd} share of the steel profile in the design transverse force
$V_{pl.Rd}$ cross section resistance of the steel section for shear

$$V_{pl.Rd} = A_V \frac{f_{yd}}{\sqrt{3}} \tag{49}$$

with

A_V shear area of the structural steel part.

As reduction for the area results:

$$\text{red } A_V = A_V \left[1 - \left(\frac{2\ V_{Sd}}{V_{pl.Rd}} - 1 \right)^2 \right] \tag{50}$$

For a concrete encased I-section with bending about the strong axis of the profile results with the relation shown in fig. 1:

$$\text{red } A_V = \text{red } t_w\ h \tag{51}$$

With this reduced thickness the method shown in chapter 6.3 for determination of the cross section interaction curve can be applied without any modification. For the purpose of simplification a division of the shear force in a steel profile part and a concrete part may be neglected. Especially for concrete filled hollow sections the shear force is then attributed to the outer steel part only.

7.2 Regions of load introduction

If loads are introduced into the composite column, it must be ensured, that after a certain introduction length the single components of the cross section are loaded according to their capacity, so that no significant slip occurs between these parts. On this purpose, similar to chapter 7.1, a division of the loads in a steel part and a concrete part must be made.

The exact distribution would have to be made over stress differences between the beginning of the region of load introduction and its end. The length l_e of the region of load introduction should not exceed the value of eqn. 52.

$$l_e \leq 2\,d \tag{52}$$

where d is the smaller of the two cross section dimensions, the dimension vertically to the bending axis respectively.

A simple method of distributing the loads to be introduced can be made by help of the plastic resistance of the different cross section components:

$$N_{c.Sd} = N_{Sd}\left(1 - \frac{N_{pl.a.Rd}}{N_{pl.Rd}}\right) \tag{53}$$

$$N_{a.Sd} = N_{Sd} - N_{c.Sd} \tag{54}$$

$$M_{c.Sd} = M_{Sd}\,\frac{M_{pl.c.Rd}}{M_{pl.Rd}} \tag{55}$$

$$M_{a.Sd} = M_{Sd} - M_{c.Sd} \tag{56}$$

where

$N_{pl.a.Rd}$ is the plastic resistance to normal force of the steel section

$N_{pl.Rd}$ is the plastic resistance to normal force of the total cross section

$M_{pl.c.Rd}$ is the plastic resistance to bending moments of the concrete part including reinforcement and

$M_{pl.Rd}$ is the plastic resistance to bending moments of the total cross section

For the determination of $M_{pl.c.Rd}$ the steel section is neglected. Calculation can be performed following chapter 6.3.

If loads are introduced first over a connection to the steel part, the elements of load introduction (headed studs e.g.) must be designed for the concrete parts of the loading $N_{c.Sd}$ and $M_{c.Sd}$. In the case of load introduction from the concrete into the steel member (e.g. over brackets) the respective steel parts $N_{a.Sd}$ and $M_{a.Sd}$ must be taken as basis for the design.

For columns of storey-height generally head plates are used as elements of load introduction. For doing so usually no other connective devices are needed. For continous columns an arrangement of additional elements for load introduction is necessary.

For open profiles headed studs proved being economic, the use of which activates further load bearing capacities because of frictional forces (fig.12).

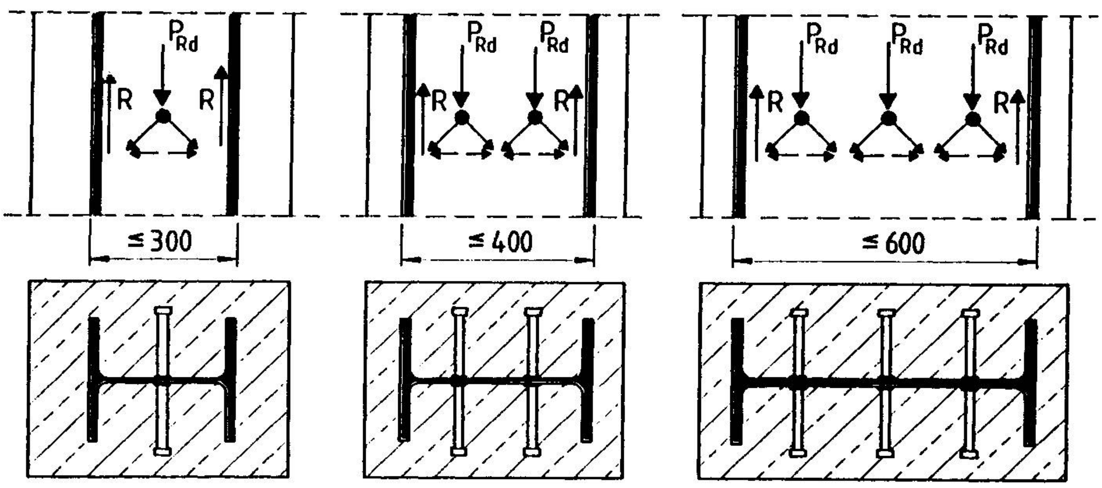


Fig. 12: Shear resistance of headed stud connectors under activation of additional frictional forces

The forces of the outward stud connectors rest on the flanges (fig. 12). As frictional force results:

$$R = \mu \, P_{Rd} / 2 \qquad \text{with} \quad \mu = 0.5 \tag{57}$$

For introducing forces into continuous concrete filled hollow profiles the use of gusset plates punched through the profile is a very economic solution. Due to the effect of confinement, high normal stresses can occur beneath the edge (fig. 13).

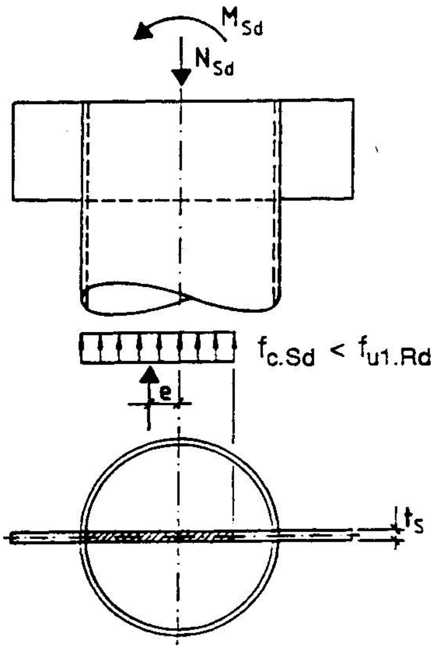


Fig. 13: Load introduction into continuous concrete filled hollow sections

$$f_{u1.Rd} = (f_{ck} + 35.0)\ \frac{1}{\gamma_c} \sqrt{\frac{A}{A_1}} \tag{58}$$

$$f_{u1.Rd} \leq \frac{N_{pm.Rd}}{A_1} \tag{59}$$

where

A is the total area of the concrete core,

A_1 is the area beneath the edge and

$N_{pm.Rd}$ is the plastic resistance of the concrete part of the section

Eqn. 58 has been derived from tests and must still be statistically verified by further investigations.

8. RESTRICTIONS FOR THE APPLICABILITY OF THE SIMPLIFIED METHOD

The application of the above described design method is subjected to various restrictions. One reason is the insufficient amount of tests for the different parameters.

For the maximum slenderness of composite beams it is here valid:

$$\bar{\lambda} \leq 2.0 \tag{60}$$

The steel contribution ratio δ according to eqn. 27 must fulfill the requirement:

$$0.2 \leq \delta \leq 0.9 \tag{61}$$

If δ is less than 0.2 the column shall be designed according to Eurocode 2. If δ exceeds 0.9, design shall be made on the basis of Eurocode 3.

If the longitudinal reinforcement is considered in design, the minimum share of 0.3% of the concrete area must be provided. The maximum share of the reinforcement at the concrete cross section which can be applied in the analysis is 4%. For reasons of fire protection higher shares of the reinforcement can be applied, but shall not be taken into account for the "cold design".

$$0.3\ \% \leq \frac{A_s}{A_c} \leq 4.0\ \% \tag{62}$$

Concrete filled sections may be fabricated without any reinforcement. For concrete encased sections longitudinal reinforcement may also be neglected. In this case only a surface reinforcement is necessary. Steel fabric reinforcement may be used as links.

For the concrete cover of completely encased profiles eqn. 11 is valid. Generally a minimum cover of 40 mm has to be provided. The maximum cover of the steel profile is also restricted. In this case, also as for the share of the reinforcement, it is possible, that the cover may be greater but shall not be taken into account in design as (notations cf. fig. 1):

$$40\ \text{mm} \leq c_z \leq 0.3\ h \tag{63}$$

$$40\ \text{mm} \leq c_y \leq 0.4\ b \tag{64}$$

9. DESIGN OF COLUMNS WITH UNSYMMETRICAL SECTIONS

The method given in /6/ is valid for mono-symmetrical sections. A simplified method for sections without any axis of symmetry is not yet available. Because of the different behaviour of the materials steel and concrete an additional relevant axis occurs in mono-symmetrical sections. Fig. 14 shows the elastic centroidal axis of the section, which is determined using the stiffnesses of the components and the plastic centroidal axis, determined with the strengths of the different materials. The plastic centroidal axis is the centre of stress distributions under pure compression. For double symmetrical sections according to fig. 1 all these axes are the same, the middle line respectively.

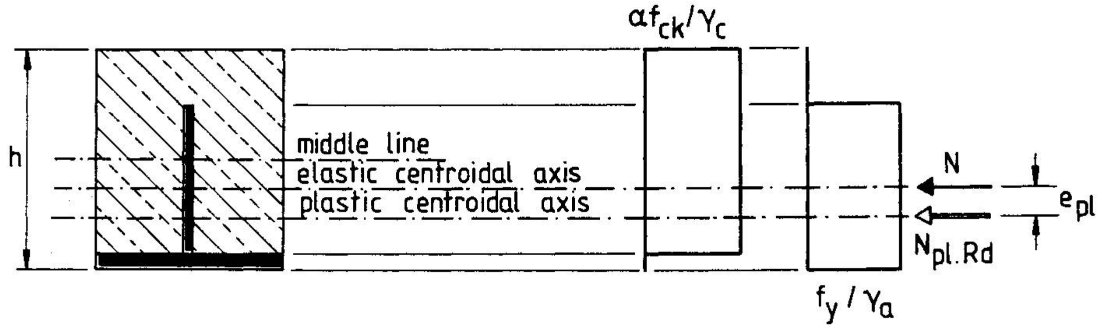


Fig. 14: Axes of mono-symmetrical cross-section

As in common design problems the elastic centroidal axis is the reference axis for the internal forces, this axis is also chosen for the design of composite columns with mono-symmetrical sections. The simplified method relies very close on the simplified method for symmetrical sections. For the determination of the relative slenderness according to eqn. 22, the calculation of deflections and internal forces the stiffness of the section has to be determined using the elastic centroidal axis. The design for axial compression then may be done according to chapter 5.2 but using the European buckling curve d. For this curve d the factor α in eqn. 21 is 0.76.

For the design under combined compression and bending the additional eccentricity e_{pl} according to eqn. 65 has to be regarded, which is the distance between the elastic and the plastic centroidal axis.

$$e_{pl} = \frac{\sum_i (A_i E_i z_i)}{\sum_i (A_i E_i)} - \frac{\sum_i (A_i f_i z_i)}{\sum_i (A_i f_i)} \tag{65}$$

where A_i are the relevant areas
E_i are the stiffness moduli for the areas according to Eurocode 2 and 3
f_i are the design strengths of the materials for the areas, and
z_i are the distances to the reference axis for the calculation

The design is then carried out with eqn. 66. The value of the additional moment caused by the eccentricity e_{pl} should be taken absolutely so that the total moment is enlarged.

$$|M_{Sd}| + |N_{Sd}\, e_{pl}| \leq 0.9\mu\, M_{pl.Rd} \tag{66}$$

Carrying out the design for combined compression and uniaxial bending it should be considered that the cross section interaction curve for the axis of unsymmetry is different for moments with positive or negative sign (fig. 15). The effort for the determination of the resistance of the section may be higher if the moment distribution of the member is of double curvature shape. The determination of the design point is more difficult. In addition the method for the determination of the interaction curve as described in chapter does not work for mono-symmetrical sections.

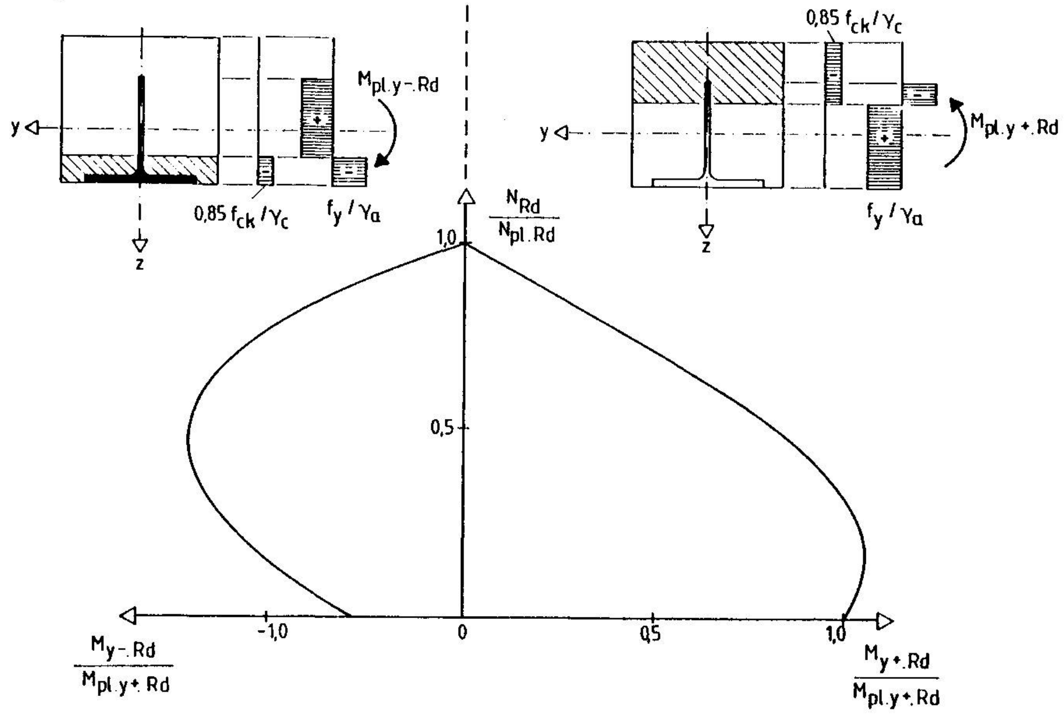


Fig. 15: Complete interaction curve for a mono-symmetrical section

The application of this simplified method for mono-symmetrical sections is restricted by an eccentricity of the elastic centroidal axis to the middle line of the section not greater than h/10, where h ist the overall depth of the section opposite to the bending axis.

For common design problems the unsymmetry may be caused by a different amount of reinforcement in the tension and compression zone of the section or by any hole, which may be necessary for service ducts or similar. In most of these cases the engineer may use the simplified design method for symmetrical sections by cutting the mono-symmetrical section to a symmetrical one for the design.

10. SUMMARY

This paper deals with the simplified design method for composite columns under axial compression and combined compression and bending. The cross sections normally are symmetrical about both main axes. The design for composite columns with mono-symmetrical cross sections is also shortly presented. The simplified design method works by adding up the capacities of the components of the section. This method might be a basis for a design method for columns under fire, because it is very simple to cut parts from the section, which may have failed due to the heat. The developement of a simplified design for fire is one of the tasks of the future in the field of composite columns.

11. REFERENCES

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/6/ Eurocode 4, Design of Composite Structures, Technical Paper R56, Annex C, Design of Composite Columns with mono-symmetrical Cross Sections - Simplified Method, Bochum, Nov. 1989.