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Composite Girders

Poutres mixtes

Verbundträger

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SUMMARY

The rules of Eurocode 4 for the design and construction of composite girders are explained. In doing so it is especially treated with the specialities of such types of structures.

RÉSUMÉ

Les règlements d'Eurocode 4 pour le dimensionnement et la construction des poutres mixtes sont expliqués. En particulier les spécialités de ces constructions sont montrées.

ZUSAMMENFASSUNG

Die Regelungen des Eurocode 4 für die Bemessung und Konstruktion von Verbundträgern werden erläutert. Dabei wird insbesondere auf die Besonderheiten solcher Konstruktionen eingegangen.

1. PRELIMINARY REMARKS

In this report the present rules of Eurocode 4 for the design and construction of composite girders are explained (latest state: May 1990). The planned rules may still vary a bit since discussions have not yet been finished and the new draft of EC 4 is not yet completed.

Herein composite girder means slender composite beams with cross sections in class 3 and 4 (cf. chap. 3.2). Therefore resistance of the cross section and the influence of local buckling and lateral torsional buckling are of great importance for composite girders.

2. LOAD CARRYING BEHAVIOUR OF CONTINUOUS GIRDERS

Increasing the load on a continuous girder according to fig. 1 the load-deformation-curve shown results qualitatively. The following load levels can be marked:

P_1 - crack formation in the concrete flange at the support
P_2 - yielding of steel in one of the edge fibres at the support
P_3 - plastic hinge at the support
P_4 - additional plastic hinges in the span areas, forming a mechanism

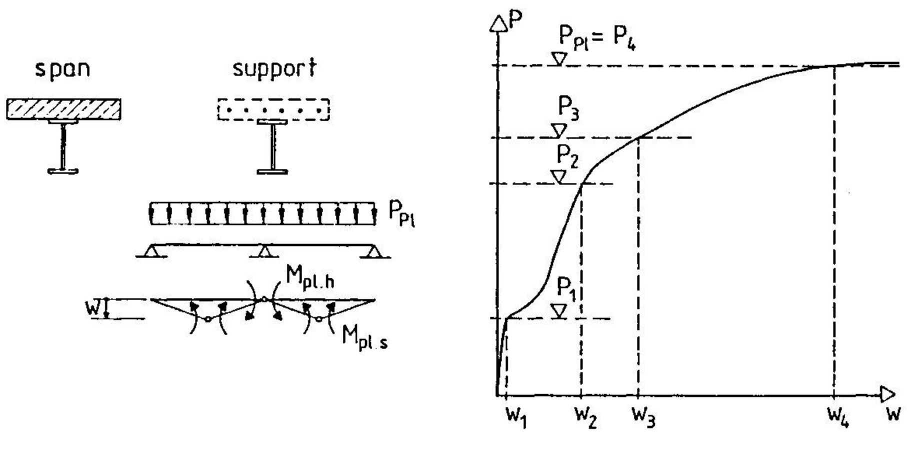


Fig. 1 Example of the load carrying behaviour

The shown load carrying behaviour is completely valid only for composite beams with sufficient rotation capacity. In composite girders (cross section classes 3 and 4) only the maximum load level P_2 can be reached.

3. ULTIMATE LIMIT STATE

3.1 Verifications

For the ultimate limit state it shall be verified that:

$$S_d \leq R_d$$

(S = stress, R = resistance, d = design),

This means that the action effects shall be less than or equal to the resistance capacity. The action effects result from structural analysis with the loads and the respective partial safety factors according to Eurocodes 2, 3 and 4.

The resistance capacities in composite structures are determined with the properties of the different materials. Under consideration of the partial safety factors of the materials results:

$$R_d = \frac{1}{\gamma_{Rd}} \cdot R\left[\frac{f_y}{\gamma_a}, \frac{f_{ck}}{\gamma_c}, \frac{f_{sk}}{\gamma_s}\right]$$

With:

f_y, f_{ck}, f_{sk} - characteristic values of the strengths of structural steel, concrete and reinforcement

γ_a, γ_c, γ_s - partial safety factors
- structural steel $\gamma_a = 1.0$
- concrete $\gamma_c = 1.5$
- reinforcement $\gamma_s = 1.15$

γ_{Rd} - coefficient of the system
- lateral torsional buckling, buckling $\gamma_{Rd} = 1.1$
- in all other cases $\gamma_{Rd} = 1.0$

For the ultimate limit state different influences may be decisive. The example of a typical continuous girder according to fig. 2 provides a general survey.

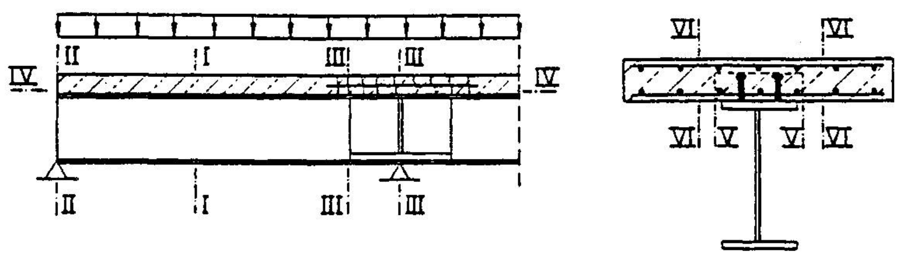


Fig. 2 Decisive sections for the design of a continuous girder

In the marked sections the following checks concerning the load bearing capacity must be carried out:

I-I	Bending moment
II-II	Shear force
III-III	Interaction bending moment-shear force
IV-IV	Shear connectors (longitudinal shear)
V-V	Shear resistance at the studs
VI-VI	Shear resistance in the concrete flange

Moreover the composite girders shall be checked for local buckling and lateral torsional buckling in region of the supports.

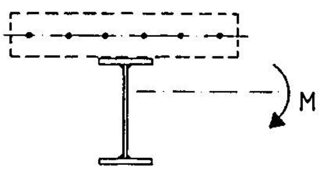
3.2 Classification of cross sections

Cross sections can be grouped in 4 classes:

- Class 1 cross-sections are those which can form a plastic hinge with the rotation capacity required for plastic analysis.
- Class 2 cross-sections are those which can develop their plastic moment resistance, but have limited rotation capacity.
- Class 3 cross-sections are those in which the calculated stress in the extreme compression fibre of the steel member can reach its yield strength, but local buckling is liable to prevent development of the plastic moment resistance.
- Class 4 cross-sections are those in which it is necessary to make explicit allowances for the effects of local buckling when determining their moment resistance or compression resistance.

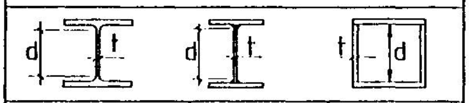
The classification of the cross sections depends on the sign of the bending moment and the proportions of the cross section parts in compression. For sections in hogging bending the adequate class of the compression flange can be determined with fig. 3.

Class	compression flanges : max c/t ratios			
	t, c		t, c	
1	10 ε	10 ε	9 ε	9 ε
2	11 ε	15 ε	10 ε	14 ε
3	15 ε	21 ε	14 ε	20 ε
4	>15 ε	>21 ε	>14 ε	>20 ε

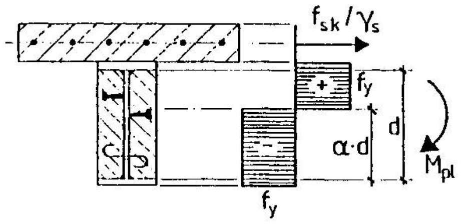


$$\varepsilon = \sqrt{\frac{235}{f_y\ [N/mm^2]}}$$

Fig. 3 Maximum width-to-thickness ratios for steel outstand flanges in compression webs: max d/t ratios



Class	1	2
(tension positive)	$\alpha > 0.5$ $\frac{d}{t} \leq \frac{396\,\varepsilon}{13\alpha - 1}$ $\alpha < 0.5$ $\frac{d}{t} \leq \frac{36\,\varepsilon}{\alpha}$	$\alpha > 0.5$ $\frac{d}{t} \leq \frac{456\,\varepsilon}{13\alpha - 1}$ $\alpha < 0.5$ $\frac{d}{t} \leq \frac{41.5\,\varepsilon}{\alpha}$
Class	3	
	$\psi > -1$ $d/t \leq 42\varepsilon/(0.67+0.33\psi)$ $\psi \leq -1$ $d/t \leq 62\varepsilon(1-\psi)\sqrt{(-\psi)}$	



A web in Class 3 that is encased in concrete may be assumed to be in Class 2!

$$\varepsilon = \sqrt{\frac{235}{f_y}}$$

Fig. 4 Maximum width-to-thickness ratios for steel webs

The maximum ratios of width-to-thickness for steel webs result from fig. 4. Cross sections with a ratio d/t larger than the allowed value for class 3 must be grouped in class 4.

For sections with the compression flange in Class 1 or 2, the class of the web shall be determined from fig. 4, using the plastic neutral axis for the effective composite section. An uncased web in Class 3 may be represented by an effective web in Class 2 as shown in fig. 13.

The class of the web shall be determined from fig. 4, using the elastic neutral axis for sections where the compression flange is in Class 3 or 4.

In beams and girders for buildings, the position of the elastic neutral axis should be determined for the effective concrete flange, neglecting concrete in tension, and the gross cross-section of the steel web. The modular ratio for concrete in compression should be as used in the global analysis for long-term effects.

The classification in 4 cross section classes shall take care for the existing rotation capacity of the cross sections and the maximum load bearing capacity which can be realized. Fig. 5 reveals the influence of the rotation capacity on the load bearing capacity by an example.

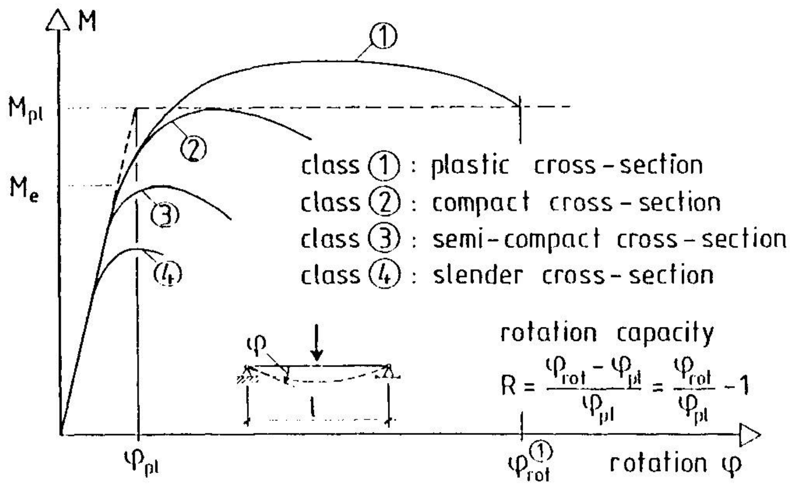


Fig. 5 Rotation capacity and cross section classes

3.3. Analysis for continuous girders

For the cross section classes according to chapter 3.2 results the permitted way of determining the respective action effects according to fig. 6. The respective cross section resistance is also defined there.

cross-section class	calculation of bending moments	resistance of cross-sections
1	plastic hinge analysis or elastic analysis and redistribution of moments	plastic moment
2	elastic analysis and redistribution of moments	plastic moment
3	elastic analysis and redistribution of moments	elastic limiting moment
4		elastic limiting moment taking into account local buckling (web and flange)

Fig. 6 Calculation of action effects and cross section resistance

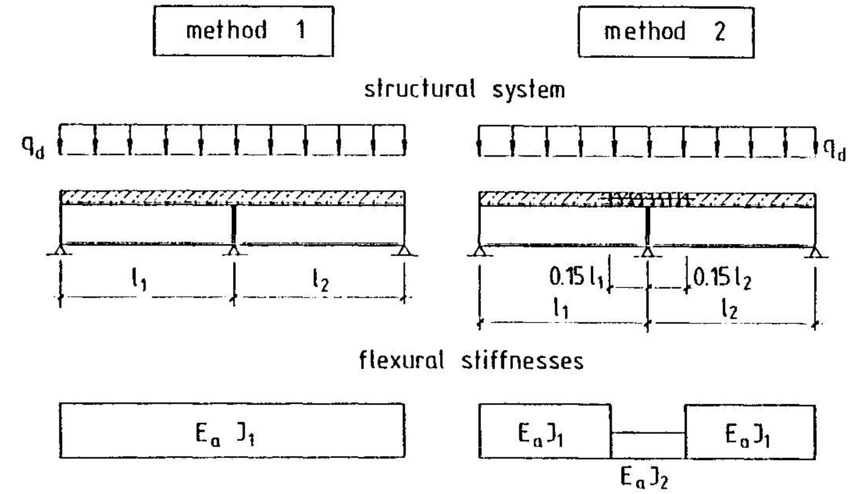


Fig. 7 Calculation of internal moments by elastic global analyis

cross-section class	1	2	3	4
method 1	40	30	20	10
method 2	25	15	10	0

Fig. 8 Permitted redistribution of moments [%] with calculation according to theory of elasticity

For the calculation of internal moments by elastic analysis it can be proceded in two ways (fig. 7). In method 1 the concrete is fully taken into account in all regions of the beam for the determination of the flexural stiffness. In contrast to that the concrete flange in the tension area is neglected in method 2 (cracked concrete flange). In region of the inner support the girder has than only a reduced flexural stiffness which extends into the spans by 0.15 l_i to each side. The distribution of bending moments resulting from method 1 and 2 are of course different. In method 2 the moments at a support are usually less than in method 1.

According to fig. 8 the moments at the supports determined in this way may be reduced by the given percentages. The moments in the span must then be adequately enlarged in the way that equilibrium is satisfied. Moments applied to the steel member should not be redistributed.

Where unpropped construction is used for structures with composite girders that have cross-sections in Class 3 or Class 4, appropriate global analyses shall be made for the separate effects of permanent actions applied to the steel member and actions applied to the composite member.

For cross sections with wide concrete flanges the effective widths of the flange can be determined according to fig. 9. An approximate determination of the distances L_0 between points of zero bending moments is given there. As effective width $L_0/8$ may be applied, but maximally the flange width. Determination of the moments of inertia is than made by means of the effective width. The parts of the cross section, which must be applied for the calculation of moments of inertia, are shown in fig. 10. The following two cases are differentiated therein:

- cross section with concrete flange in compression;
- cross section with concrete flange in tension (cracked concrete neglected).

For global analysis a constant effective width may be assumed over the whole of each span. This value may be taken as the midspan value for a beam, or the support value for a cantilever.

For the verification of the cross-sections, the different effective widths for sections in sagging bending and for sections in hogging bending should be used.

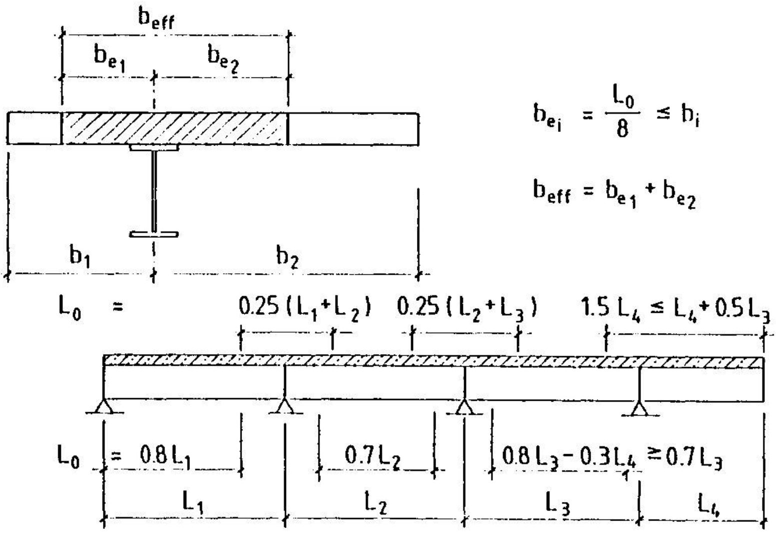


Fig. 9 Effective width of concrete flange

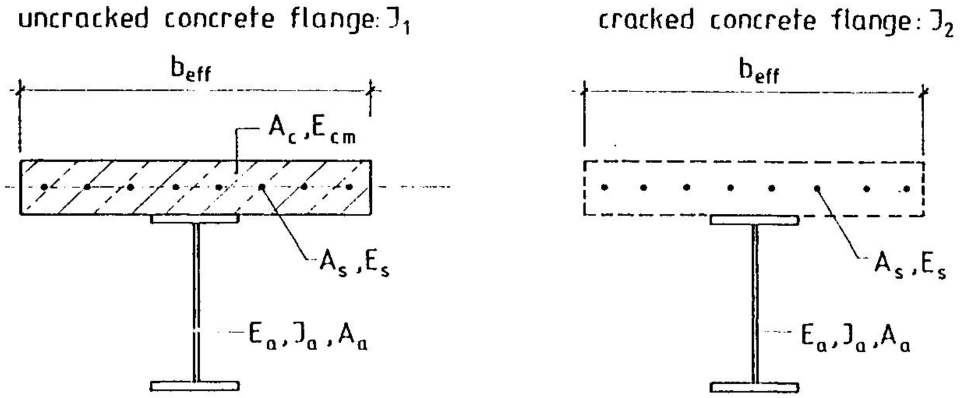


Fig. 10 Cross-section components for the calculation of the second moment of area

The elastic section properties of a composite cross-section should be expressed as those of an equivalent steel cross-section by dividing the contribution of the concrete component by a modular ratio n. The modular ratio may be taken as:

- $n_{co} = E_a/E_{cm}$ for short-term action effects;
- $n_{c\infty} = 3 \cdot n_{co}$ for long-term action effects;

for composite beams in buildings mainly intended for storage. Buildings mainly intended for storage excepted, the modular ratio should be taken equal to $n_{eff} = 2 \cdot n_{co}$ for all action effects. Those methods are accurate enough to take account of creep.

The effects of shrinkage of concrete may be neglected in verifications for ultimate limit states for composite structures in buildings, except global analysis with members having cross-sections in Class 4. No consideration of temperature effects in verifications for ultimate limit states is normally necessary for composite structures for buildings.

3.4 Resistances of cross-sections

3.4.1 Bending moment

The elastic analysis may be applied to cross-sections of any class. Where the effective composite section is in Class 1 or Class 2 the design bending resistance may be determined by plastic theory. The following assumptions shall be made:

- the tensile strength of concrete is neglected,
- plane cross-sections of the structural steel and reinforced concrete parts of a composite member each remain plane.

The typical plastic stress distributions for composite sections in Class 1 or Class 2 are shown in fig. 11 and fig. 12.

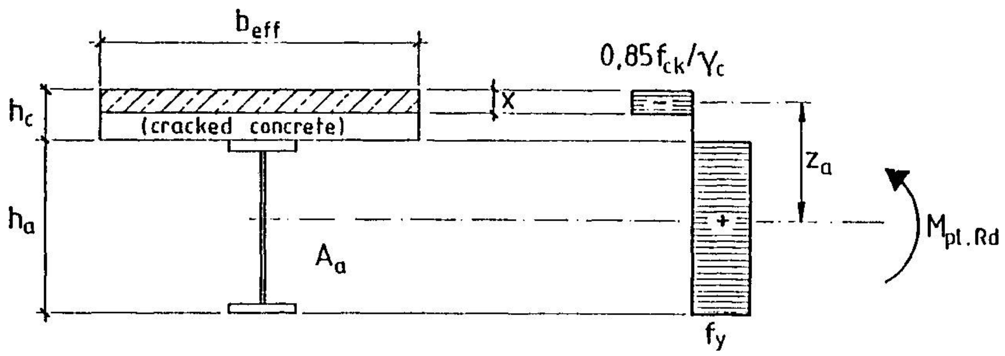


Fig. 11 Plastic stress distributions for sections in sagging bending

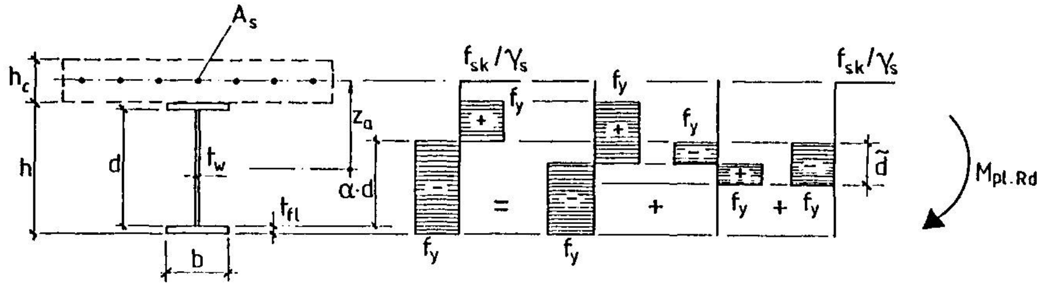


Fig. 12 Plastic stress distributions for sections in hogging bending

The plastic bending resistance for a section in sagging bending can be determined with fig. 11 under consideration of the equilibrium condition:

$$x \cdot b_{eff} \cdot 0.85\ f_{ck}/\gamma_c = A_a \cdot f_y$$

$$\Rightarrow x\ ,\ \text{but}\ \ x \le h_c$$

$$z_a = \frac{1}{2}\ h_a + h_c - \frac{1}{2}\ x$$

$$M_{pl.Rd} = A_a \cdot f_y \cdot z_a$$

In fig. 11 it had been assumed, that the plastic neutral axis lies within the concrete flange. If it lies within the steel beam it can be proceeded in analogy.

For the case of hogging bending results with fig. 12:

$$M_{pl.Rd} = M^{a}_{pl.Rd} - \frac{1}{4}\ t_w \cdot \tilde{d} \cdot f_y + z_a \cdot A_s \cdot f_{sk}/\gamma_s$$

$$\tilde{d} = \frac{A_s \cdot f_{sk}/\gamma_s}{A_w \cdot f_y} \cdot d$$

$$\alpha = \frac{1}{2} + \frac{\tilde{d}}{2 \cdot d}$$

In the classification of cross sections according to chapter 3.2 it may result, that the flange belonges to class 2 and the web to class 3. Nevertheless this cross section may be assigned to class 2, if the effective web area is applied according to fig. 13.

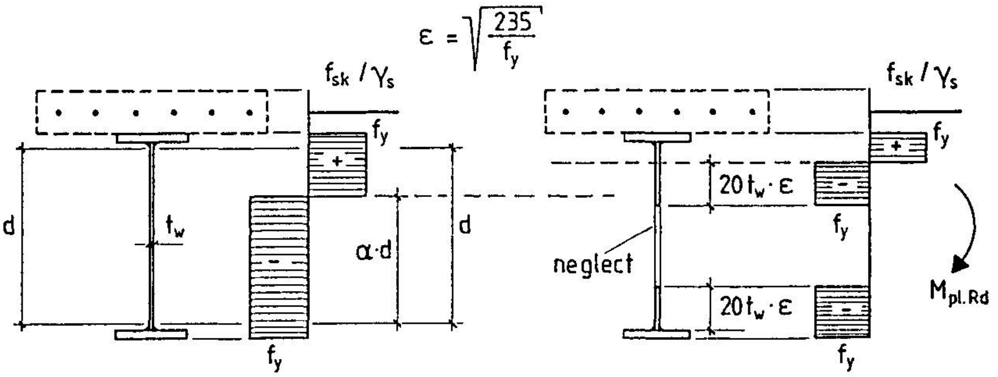


Fig. 13 Plastic stress distributions for sections in hogging bending and use of an effective web

For elastic bending resistances the stresses shall be calculated by elastic theory. Where unpropped construction is used, stresses due to actions on the structural steelwork alone shall be added to stresses due to actions on the composite member. The limiting bending stresses shall be taken as:

- f_y/γ_a in structural steel in tension or compression;
- f_{sk}/γ_s in reinforcing steel in tension or compression;
- $0.85 \cdot f_{ck}/\gamma_c$ in concrete in compression.

The elastic resistance to bending, $M_{e.Rd}$, shall be taken as the sum of the bending moments in the steel section and the composite section when the sum of the stresses due to these bending moments reaches anyone of the above-mentioned limits. The partial stress distribution for cross sections of class 3 is shown in fig. 14.

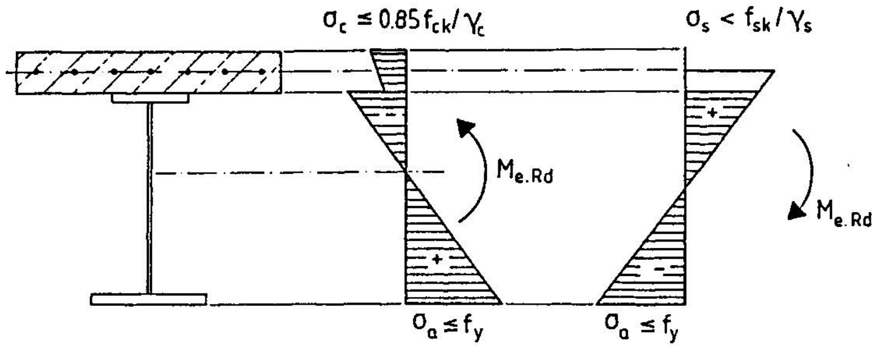


Fig. 14 Elastic stress distributions for sections in Class 3

For cross sections in Class 4 an effective cross section must be used. In doing so the bottom flange and the web panels must be reduced according to the effects of local buckling, fig. 15.

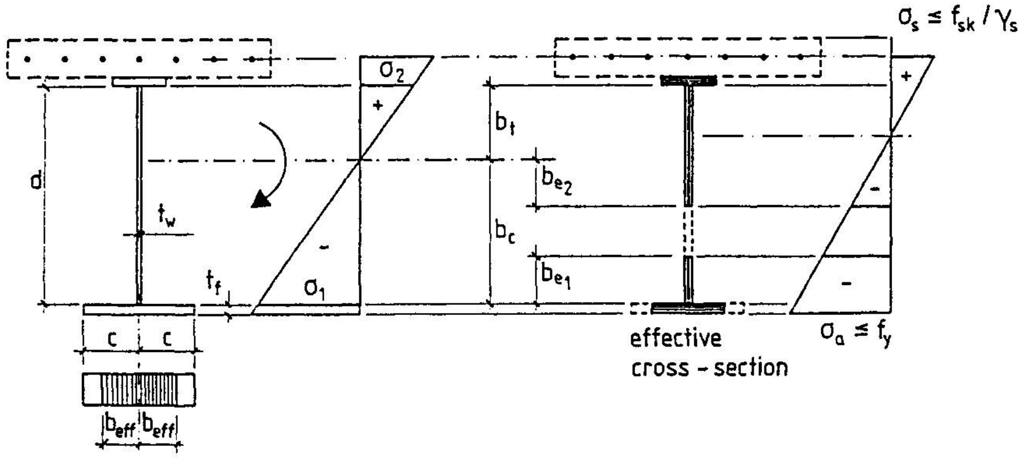
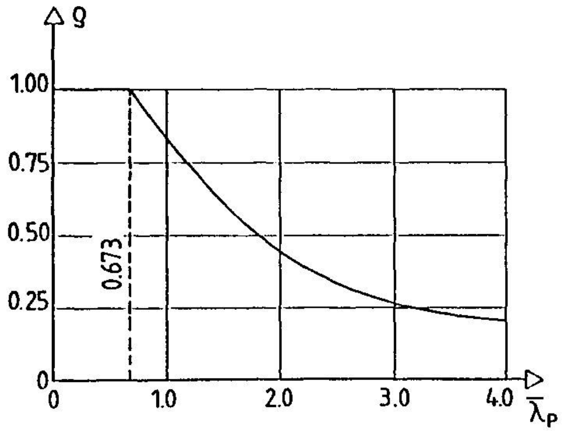


Fig. 15 Elastic stress distributions and effective cross-section for sections in Class 4



$$\bar{\lambda}_p = \frac{d/t_w}{28.4\,\varepsilon\sqrt{k_\sigma}}$$

k_σ - buckling factor

$$\rho = \frac{\bar{\lambda}_p - 0.22}{\bar{\lambda}_p^2} \leq 1.0$$

$$\varepsilon = \sqrt{\frac{235}{f_y\ [N/mm^2]}}$$

Fig. 16 Reduction factor ρ (local buckling)

The influence of local buckling can be taken into account by the reduction factor ρ according to fig. 16. Thus results for the web

$$b_{eff} = \rho \cdot b_c, \quad b_{e1} = 0.4 \cdot b_{eff} \quad \text{and} \quad b_{e2} = 0.6 \cdot b_{eff}$$

With

$$\bar{\lambda}_p = \frac{c/t_f}{28.4 \cdot \varepsilon \cdot \sqrt{k_\sigma}}$$

and ρ according to fig. 15 results for the bottom flange

$$b_{eff} = \rho \cdot c$$

The buckling factors k_σ for the bottom flange and the web can be read off the tables 5.3.2. and 5.3.3. of Eurocode 3.

3.4.2 Vertical shear

The resistance to vertical shear shall be taken as the resistance of the structural steel section. The plastic shear resistance is given by:

$$V_{pl.Rd} = A_v \cdot f_y / \sqrt{3}$$

where A_v is the shear area of the structural steel member as defined in fig. 17.

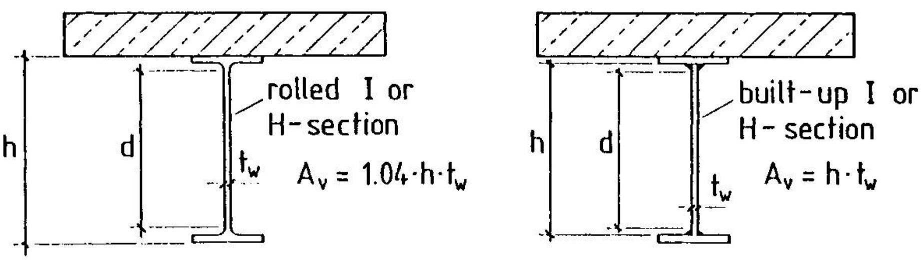


Fig. 17 Shear areas

The shear force resisted by the structural steel section shall satisfy:

$$V_{sd} \leq V_{pl.Rd}$$

The shear buckling resistance of the steel web must not be verified:

- for an unstiffened web with $d/t_w \leq 69\varepsilon$;
- for a transverse stiffened web with

$$d/t_w \leq 30 \cdot \varepsilon \cdot \sqrt{k_\tau} \;;$$

where k_τ is the buckling factor for shear
$\varepsilon = \sqrt{235/f_y}$

If the above mentioned limits are exceeded, it shall be verified that:

$$V_{sd} \leq V_{b.Rd}$$

$V_{b.Rd}$ is the shear buckling resistance

$$V_{b.Rd} = A_v \cdot \tau_{ba}$$

and τ_{ba} the simple post-critical shear strength (Eurocode 3, 5.6.3). τ_{ba} can be determined using fig. 18.

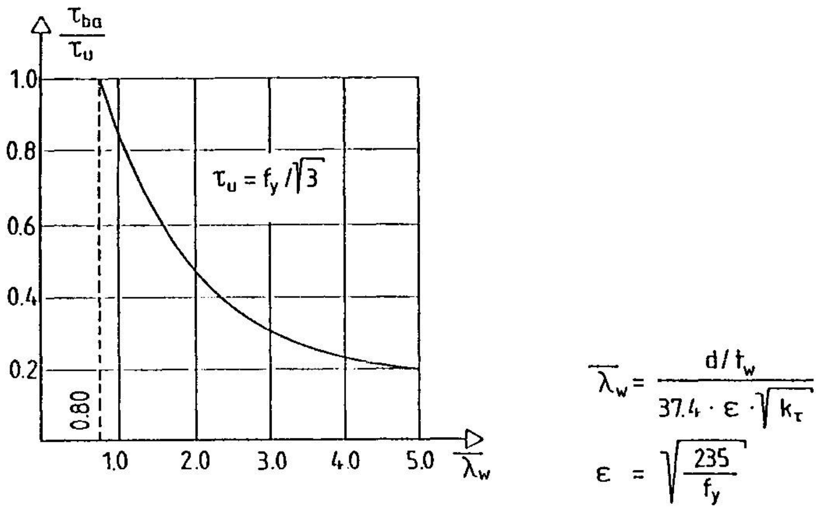


Fig. 18 Simple post-critical shear strength τ_{ba}

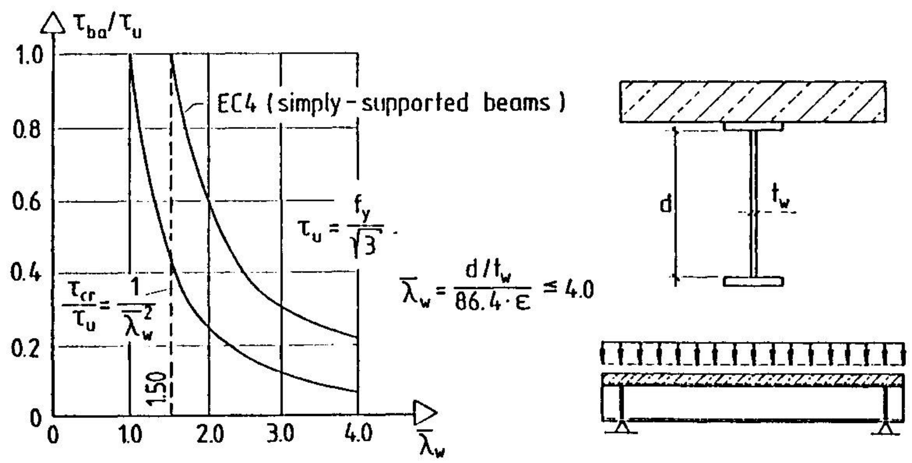


Fig. 19 τ_{ba} for simply-supported beams

For simply-supported beams without internal stiffeners, with full shear connection and subjected to uniformly-distributed loading, the simple post-critical shear strength τ_{ba} may be determined with fig. 19. The number N of shear connectors in each half span should be sufficient to provide full shear connection. They should be uniformly distributed when

$$V_{sd} \leq V_{cr}$$

where $V_{cr} = A_v \cdot \tau_{cr}$

τ_{cr} = elastic critical shear strength.

When $V_{sd} > V_{cr}$, the N connectors should be distributed as shown in fig. 20.

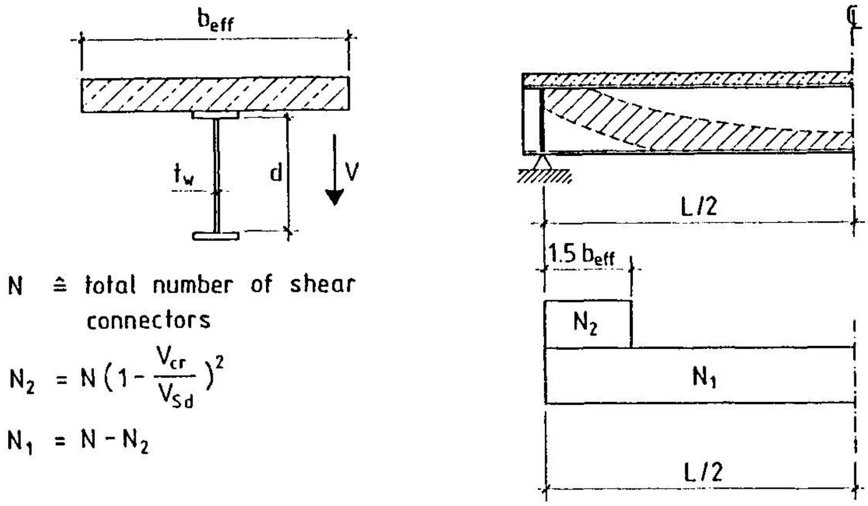


Fig. 20 Distribution of shear connectors, when $V_{sd} > V_{cr}$

Alternatively verification for sufficient shear buckling resistance may also be made by the tension field method of Eurocode 3, 5.6.4.

3.4.3 Bending and vertical shear

Where the vertical shear V_{sd} exceeds half the plastic shear resistance $V_{pl.Rd}$ due allowance shall be made for its effect on the plastic resistance moment. The plastic resistance moment reduced for vertical shear should be calculated by using a reduced design yield strength for the steel web:

$$f_{y.red} = (1 - \rho_1) \cdot f_y$$

where $\rho_1 = [2 \cdot V_{sd}/V_{Rd} - 1]^2$

V_{Rd} should be taken as the plastic shear resistance $V_{pl.Rd}$, or where a check on shear buckling is required, as the lesser of $V_{pl.Rd}$ and $V_{b.Rd}$.

The reduced plastic resistance moment can then be calculated with fig. 21, as described in chapter 3.4.1.

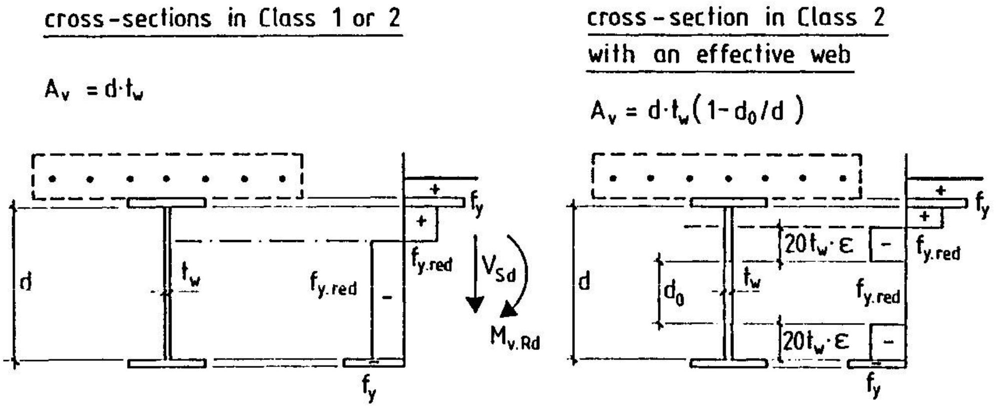


Fig. 21 Reduced yield strength for the steel web

For sections in Class 3 or Class 4, the moment of resistance should not be taken greater than the elastic resistance to bending $M_{e.Rd}$. The rules for the resistances in bending and vertical shear are shown in fig. 22. $M_{fl.Rd}$ is the plastic resistance moment when $\rho_1 = 1$, i.e. for the load bearing capacity of bending moments the steel web is considered as missing.

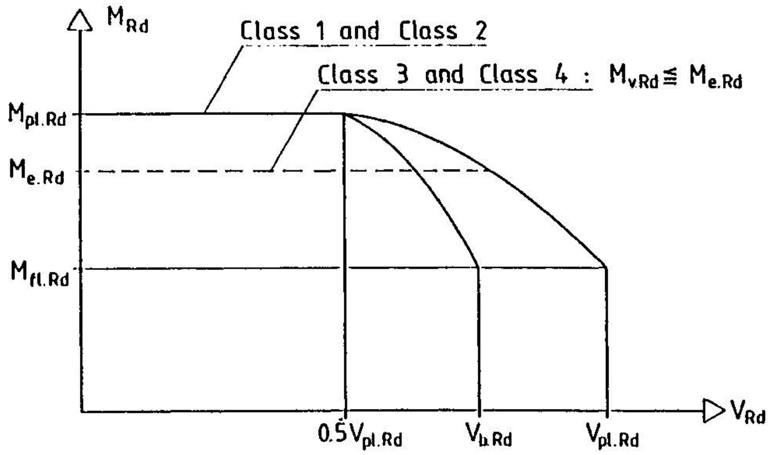


Fig. 22 Resistance in bending and shear

4. LATERAL-TORSIONAL BUCKLING

4.1 General

A steel flange that is attached to a concrete or composite slab by shear connection may be assumed to be laterally stable, provided that the overall width of the slab is not less than the depth of the steel member. All other steel flanges in compression shall be checked for lateral stability. These steel flanges in compression occur in continuous girders at the internal supports (hogging bending). When checking for lateral stability, the bending moment at any cross-section shall be taken as the sum of the moment applied to the composite member and the moment applied to its structural steel component.

4.2 Buckling resistance moment

The buckling resistance moment of a laterally unrestrained beam shall be taken as

$$M_{b.Rd} = \chi_{LT} \cdot M_{pl.Rd}/\gamma_{Rd}$$

for Class 1 or Class 2 cross-sections, and as

$$M_{b.Rd} = \chi_{LT} \cdot M_{e.Rd}/\gamma_{Rd}$$

for Class 3 or Class 4 cross-sections.

Values of χ_{LT} (reduction factor for lateral-torsional buckling) for the appropriate slenderness $\bar{\lambda}_{LT}$ may be determined from fig. 23 or

$$\chi_{LT} = [\varphi_{LT} + (\varphi^2_{LT} - \bar{\lambda}^2_{LT})]^{-1/2} \le 1$$

where

$$\varphi_{LT} = 0.5\ [1 + \alpha_{LT}\ (\bar{\lambda}_{LT} - 0.2) + \bar{\lambda}^2_{LT}]$$

and

$\alpha_{LT} = 0.21$ for rolled sections (buckling curve a)

$\alpha_{LT} = 0.49$ for welded sections (buckling curve c).

The values of $\bar{\lambda}_{LT}$ may be determined from

$\bar{\lambda}_{LT} = (M_{pl}/M_{cr})^{1/2}$ for Class 1 or Class 2 cross-sections,

$\bar{\lambda}_{LT} = (M_e/M_{cr})^{1/2}$ for Class 3 or Class 4 cross-sections,

where

M_{pl} is the value of $M_{pl.Rd}$ when the factors γ_a, γ_c and γ_s are taken as 1.0,

M_e is the value of $M_{e.Rd}$ when the factors γ_a, γ_c and γ_s are taken as 1.0,

M_{cr} is the elastic critical moment for lateral-torsional buckling.

Where the slenderness $\bar{\lambda}_{LT} \leq 0.4$, no alllowance for lateral-torsional buckling is necessary.

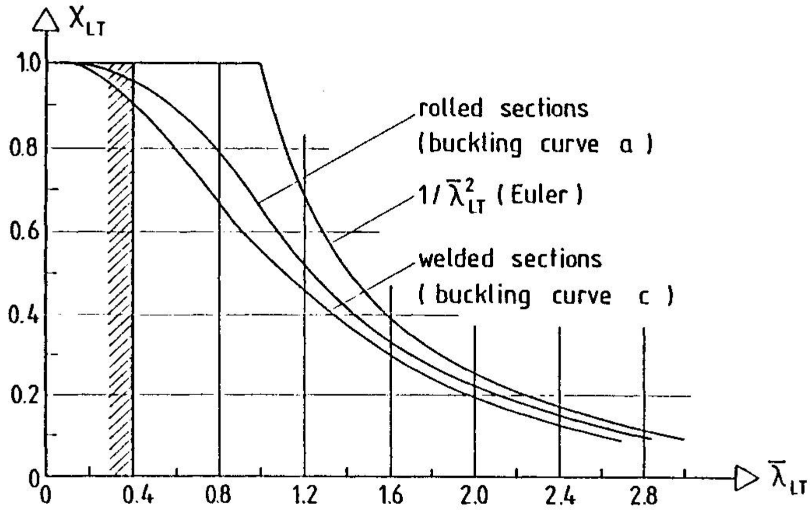


Fig. 23 Reduction factor for lateral-torsional buckling

4.3 Elastic critical moment

The elastic critical hogging moment M_{cr} at an internal support may be taken from fig. 24 for a composite beam with continuity at one or both ends and a restrained top flange.

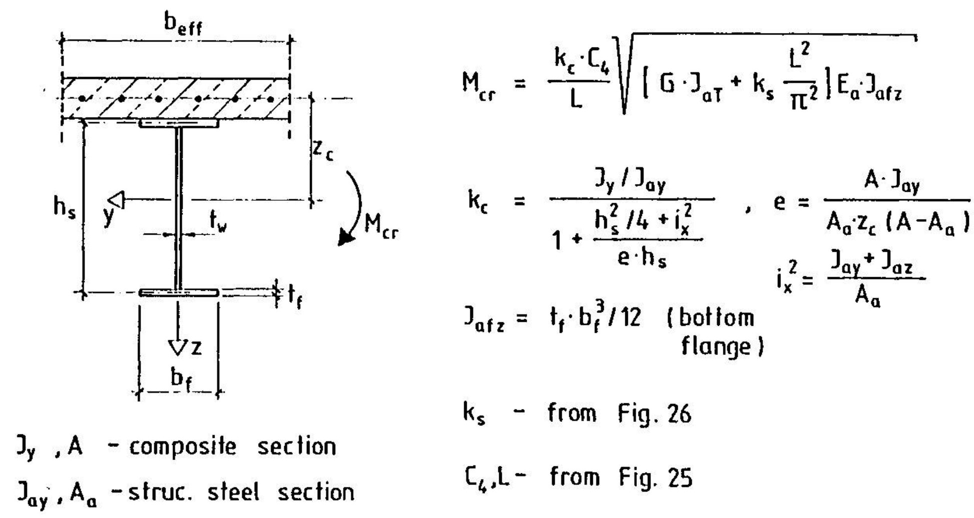
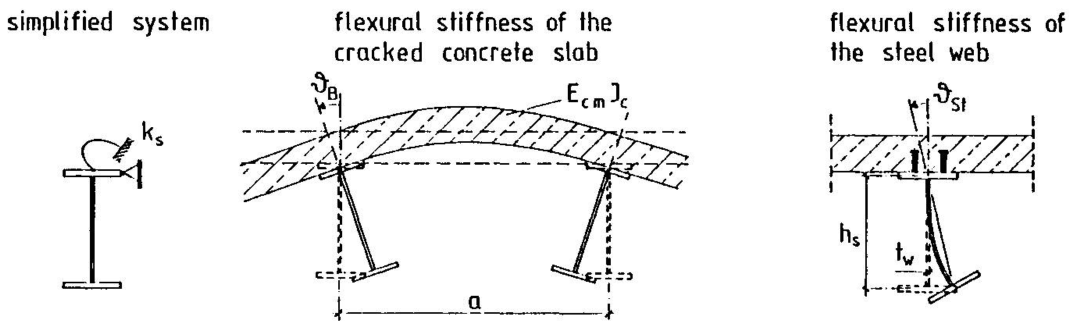


Fig. 24 Elastic critical moment M_{cr}

system	distribution of bending moment	$C_4(\psi)$ $\psi=0.50$	$\psi=0.75$	$\psi=1.00$	$\psi=1.25$	$\psi=1.50$
	$\psi \cdot M_0$, M_0	41.5	30.2	24.5	21.1	19.0
	$\psi \cdot M_0$, M_0, $0.50\ \psi \cdot M_0$	33.9	22.7	17.3	14.1	13.0
	$\psi \cdot M_0$, M_0, $0.75\ \psi \cdot M_0$	28.2	18.0	13.7	11.7	10.6
	$\psi \cdot M_0$, M_0, $\psi \cdot M_0$	21.9	13.9	11.0	9.6	8.8
	$\psi \cdot M_0$, M_0	28.4	21.8	18.6	16.7	15.6
	$\psi \cdot M_0$, M_0, $\psi \cdot M_0$	12.7	9.8	8.6	8.0	7.7

Fig. 25 Factor C_4 (distribution of bending moment)



$$k_s = \frac{k_1 \cdot k_2}{k_1 + k_2} \qquad k_1 = \alpha \frac{E_{cm} J_c}{a} \qquad k_2 = \frac{E_a \cdot t_w^3}{4\,(1 - \nu_a^2)\,h_s}$$

α = 2.0 (simply supported slab)

α = 4.0 (continuous slab)

Fig. 26 Transverse stiffness k_s

The factor k_c in fig. 24 is the value for a double symmetrical steel section. Where the cross-section of the steel member has unequal flanges, the factor k_c is given by:

$$k_c = \frac{J_y/J_{ay}}{\dfrac{z^2_f + i^2_x + z^2_s}{e \cdot h_s} + \dfrac{z_f - z_j}{0.5 \cdot h_s}}$$

where

$$z_f = h_s \cdot J_{afz}/J_{az}$$

$$z_j = z_s - \int_{A_a} \frac{z(y^2 + z^2)}{2\ J_{ay}}\, dA$$

and may be taken as $z_j = 0.4\ h_s\ (2\ J_{afz}/J_{az} - 1)$

when $J_{afz} > 0.5\ J_{az}$.

z_s is the distance from the centroid of the steel section to its shear centre, positive when the shear centre and the compression flange are on the same side of the centroid.

4.4 Check without direct calculation

A simplified method for checking lateral-torsional buckling may be used when the following conditions are satisfied:

- adjacent spans do not differ in length by more than 20 % of the shorter span;
- the loading on each span is uniformly distributed, and the design permanent load exceeds 40 % of the total design load;
- the ratio of the flexural stiffnesses (concrete slab/steel web)

$$\frac{k_1}{k_2} = 3.65 \cdot \frac{E_{cm}}{E_a} \cdot \frac{\alpha}{a} \cdot \frac{h_s}{t^3_w} \cdot J_c \quad \text{is} \quad \leq 0.4$$

In continuous composite beams, where the slenderness

$$\bar{\lambda}_{LT} = \leq 0.4,$$

no allowance for lateral-torsional buckling is necessary. This condition leads to maximum depths for rolled sections, which are given in fig. 27.

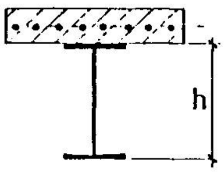
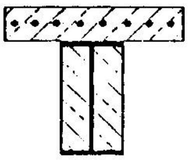
	steel member	$f_y \leq$ 240 N/mm²	$f_y \leq$ 360 N/mm²
	IPE	600	400
	HEA	800	650
	HEB	900	700
	$h \leq 1000$ mm		

Fig. 27 Maximum depth h [mm] for steel members (simplified method)

5. SERVICEABILITY LIMIT STATES

5.1 General

A serviceability limit state is reached when any of the following conditions occur:

- a deflection or change of deflection reaches a limit determined by unfitness for use, possible damage to non-structural components, ponding of rainwater, objectionable appearance, or other form of unserviceability;

- the width of a crack in concrete reaches a limit determined by the risk of corrosion of reinforcement, possible damage to flooring finish or by appearance.

Other limit states (such as vibration) may be of importance in particular structures but these are not covered in Eurocode 4, part 1.

The condition for verification in the serviceability limit state is generally expressed by:

$$E_d \leq C_d$$

where E_d is the design effect of actions and C_d represents a fixed value or a function of certain material properties (and then corresponds to R_d). Three combinations of actions for serviceability limit states are defined (see Eurocode 3, clause 2.3.4):

- rare combination;
- frequent combination;
- quasi-permanent combination.

5.2 Cracking of concrete

Elastic global analysis shall be used for the calculation of the bending moments. In the calculation of bending moments it is proceeded following method 2 which is shown in fig. 7. At each inner support the flexural stiffness is reduced to the value EI_2 over 15% of the span on each side. The concrete flange in this region has no contribution in the flexural stiffness. With this distribution of stiffness the bending moments of the continuous girder are determined.

Regarding a double-span beam with uniform spacing of the supports under uniform load distribution, a reduction of the moments at the supports can be seen from fig. 28. For common composite beams this reduction ranges between 10% and 15%.

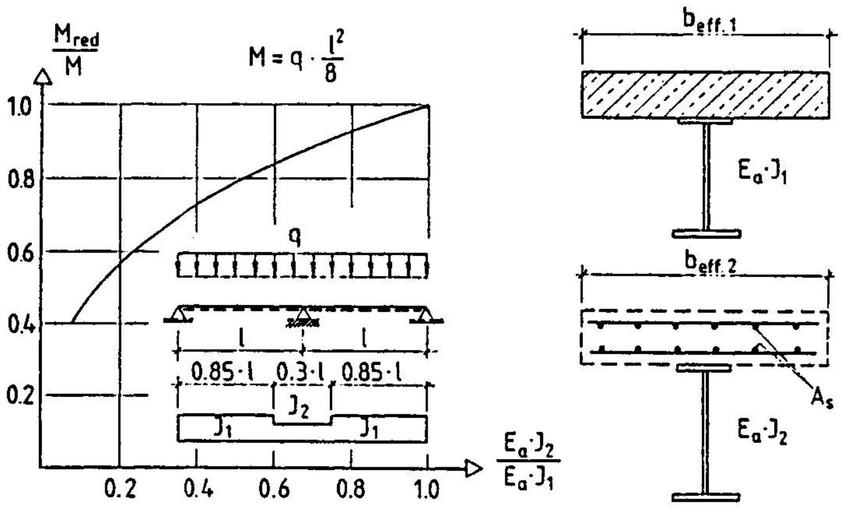


Fig. 28 Reduction of the moment at the support under crack formation in the concrete

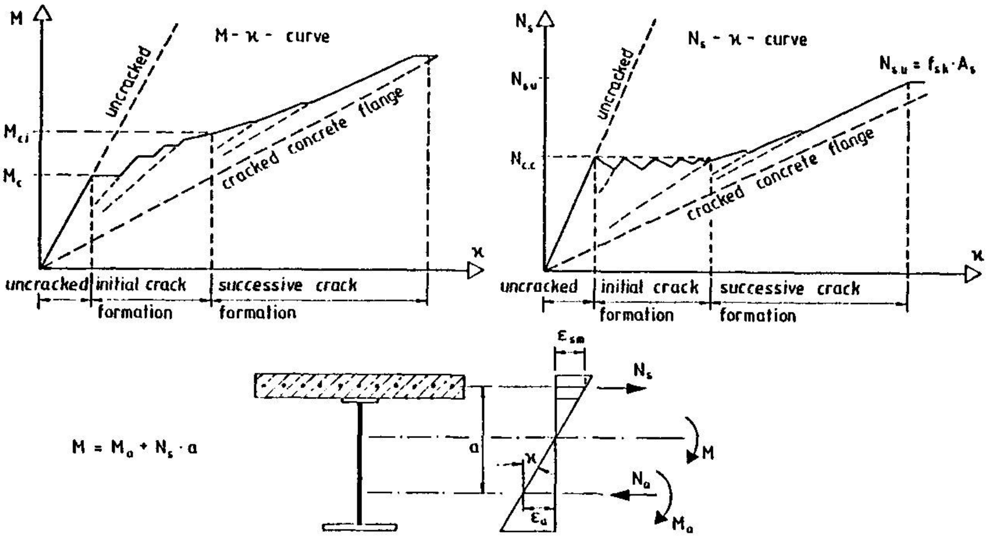


Fig. 29 Relationship between M or N_s and the curvature æ

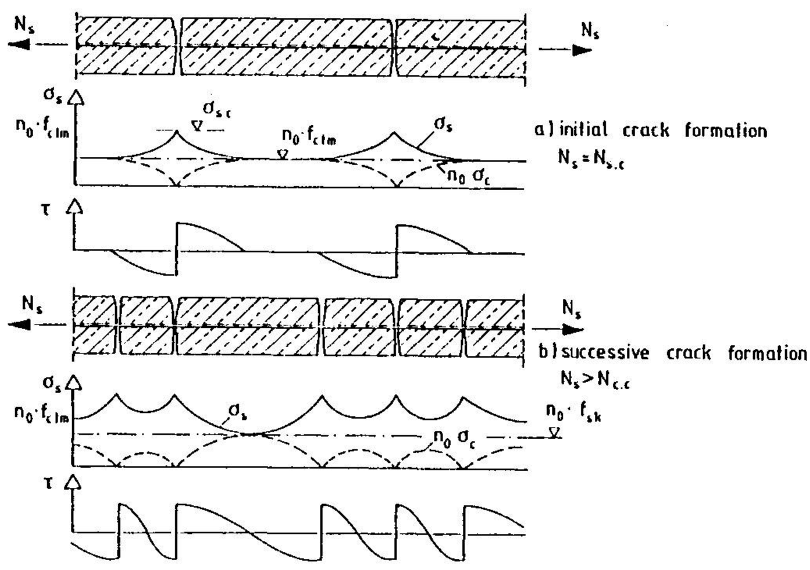


Fig. 30 Stresses in the concrete flange under initial and successive crack formation

The behaviour of composite sections with regard to cracking and curvature for sections in hogging bending shall be explained by means of fig. 29 and 30. Up to reaching the tensile strength of the concrete in the edge fibre of the concrete flanges (cracking moment M_c) the section reacts linear elastically. Now some cracks appear at relatively high distances. Due to the cracks the stiffness is reduced and the strain ε_{sm} but also the bending moment M_a in the steel beam are enlarged. Crack formation effects, that in the surroundings of the cracks a redistribution of forces from the concrete on the reinforcing steel takes place. The shear stresses between these two materials, which result from that, lead to relative displacements between concrete and reinforcing steel, i.e. the crack widens.

If the bending moment is still increased after completed initial crack formation, the crack widths also increase. Since the loading capacity of the bond of reinforcement and concrete is not yet fully exploited, more forces are introduced into the concrete between the cracks. This again leads to reaching the tensile strength of the concrete between the cracks and to further successive crack formation (fig. 30). Also under high stresses a remarkable effect of tension stiffening of the concrete between the cracks is maintained. Thus result increased normal forces of the flange N_s in comparison to a cross section without consideration of the concrete (cracked concrete flange) and smaller curvature of the total cross section. The limit state of crack formation results on reaching the yield limit in the reinforcement.

The cracks, which appear in concrete flanges under tensile stress, must be limited with regard to their width. The permitted crack width depends on the environmental conditions, if it cannot be determined exactly in any single case. The limiting values are summarized in fig. 31 according to Eurocode 2. For the environmental classes 2 to 4, i.e. for the main scope, the permitted crack width is given with w_k = 0.3 mm.
If for the environmental class 1 there is no limiting value given, but $w_k \leq$ 0.5 mm should approximately be maintained for the crack width.

Exposure class	environmental conditions	crack width
1	dry environment	no limitation
2	humid environment	$w_k \leq 0.3$mm
3	humid environment with frost and de-icing salts	
4	seawater environment	
5	aggressive chemical environment	special limitation

<u>Fig. 31</u> Environmental classes and permitted crack widths

	maximum bar diameters d [mm]	
σ_s	$w_k = 0.3$	$w_k = 0.5$
N/mm^2	mm	mm
160	32	36
200	25	36
240	20	36
280	16	30
320	12	22
360	10	18
400	8	14
450	6	12

Fig. 32 Stresses in the reinforcement σ_s in dependence upon the bar diameter and the crack width

For the permitted crack width and with the maximum diameter of the reinforcement the stress in the reinforcement can be read from fig. 32. Thus, the required minimum reinforcement can be calculated with

$$A_s = k_c \cdot k \cdot f_{ct.ef} \cdot A_c/\sigma_s$$

with

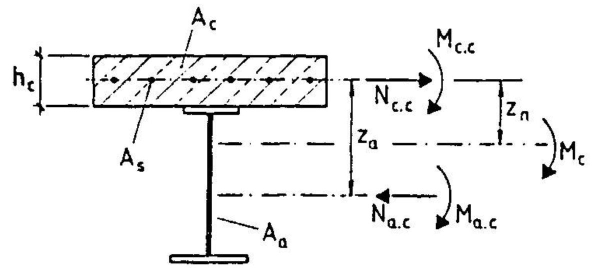
$\sigma_s(d)$ stress in the reinforcement according to fig. 32, but $\sigma_s \le f_{sk}$

A_c area of the concrete flange

$f_{ct.ef}$ effective tensile strength of the concrete (generally equal to f_{ctm}, but > 3 N/mm^2)

$k = 0.8$ coefficient for the effective decrease in tensile strength resulting from self equilibrating stresses

$$k_c = \frac{1}{1 + h_c/2z_n}$$ according to fig. 33



f_{ct} - concrete tensile strength

$n = E_a / E_{cm}$

$z_n = A_a \cdot z_a / (A_a + A_c / n)$

cracking moment : $M_c = n \cdot f_{ct} \dfrac{J_{i.0}}{z_n + h_c/2}$

reinforcing steel stress : $\sigma_{s.c} = N_{c.c} / A_s = f_{ct} \cdot k_c \cdot \dfrac{A_c}{A_s}$

$$k_c = \frac{1}{1 + h_c / 2 z_n}$$

Fig. 33 Initial crack formation and coefficient k_c

σ_s [N/mm²]		160	200	240	280	320	360	400
maximum bar spacings s_s [mm]	w_k = 0.3mm	250	200	160	110	—	—	—
	w_k = 0.5mm	250	250	250	250	200	140	80

Fig. 34 Stresses in the reinforcement σ_s in dependence upon the maximum bar spacing

For the state of completed initial crack formation under loading the bar diameters and/or the bar spacings must be limited. Verification is made by:

$\sigma_s = \sigma_{s.o} + \Delta\sigma_s \le \sigma_s (s_s)$ according to fig. 34 and/or

$\le \sigma_s (d)$ according to fig. 32

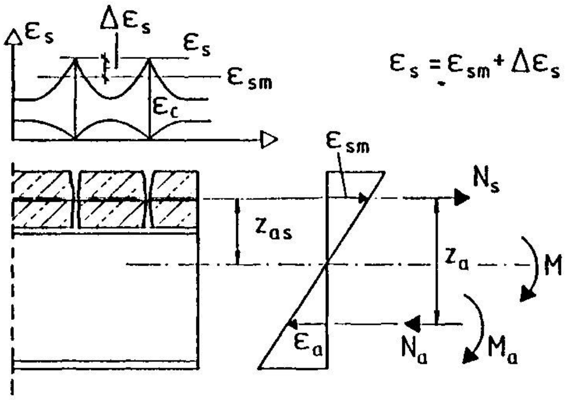
With:

σ_s - stress in the reinforcement under quasi permanent combined loading

$\sigma_{s.o}$ - stress in the reinforcement at the cross section with cracked concrete flange

$\Delta\sigma_s = 0.4 \cdot f_{ctm} / (\rho \cdot \alpha)$ from participation of the concrete

The stresses in the reinforcement $\sigma_{s.o}$ and $\Delta\sigma_s$ result from fig. 35. The values A_{as}, I_{as} and z_{as} are calculated at the effective cross section of the composite beam without participation of the concrete ("cracked").



$$\varepsilon_s = \varepsilon_{sm} + \Delta\varepsilon_s \qquad \Delta\varepsilon_s \approx 0.4 \cdot \frac{f_{ctm}}{\rho \cdot E_s}$$

$$\alpha = \frac{J_{as} \cdot A_{as}}{J_a \cdot A_a}$$

$$\rho = A_s / A_c$$

$$N_s = M \cdot \frac{A_s \cdot z_{as}}{J_{as}} + 0.4 \frac{f_{ctm} \cdot A_s}{\rho \cdot \alpha}$$

$$\sigma_s = \underbrace{\frac{M}{J_{as}} \cdot z_{as}}_{\sigma_{s.0}} + \underbrace{0.4 \cdot \frac{f_{ctm}}{\rho \cdot \alpha}}_{\Delta\sigma_s}$$

Fig. 35 Stresses in the reinforcement under completed crack formation

5.3 Deformations

The deflections of composite beams and girders shall be limited in such way that all of the conditions presented in chapter 5.1 are observed.

For floor and roof construction in buildings, recommended limiting values for vertical deflections are given in Eurocode 3 (clause 4.2.2). The limits given therein range between L/200 and L/500. They may also be applied on composite structures.

The maximum deflections due to loading applied to composite members shall be calculated using elastic analysis with consideration of the effects of:

- cracking of concrete in hogging moment regions;
- creep and shrinkage of concrete;
- local yielding of structural steel (especially when unpropped construction is used).

The effect of cracking of concrete may be taken into account as shown in fig. 7. The hogging bending moment at each internal support and the resulting top-fibre tensile stress in the concrete σ_{ct}, are first calculated using the flexural stiffness $E_a \cdot I_1$. For each support at which σ_{ct} exceeds 0.15 f_{ck}, the stiffness should be reduced to the value $E_a\ I_2$ over 15 % of the length of the span on each side of the internal supports. A new distribution of bending moment is then determined by re-analizing the beam (ct. fig. 28).

For beams with cross-sections in Classes 1, 2 or 3 alternative to the above mentioned method (and to avoid re-analysis), the bending moments at supports may be reduced by the factor f given in fig. 36. Curve A may be used when the loadings per unit length on all spans are equal and the lengths of all spans do not differ by more than 25 %. Otherwise the approximate lower bound value f = 0.6 (line B) should be used.

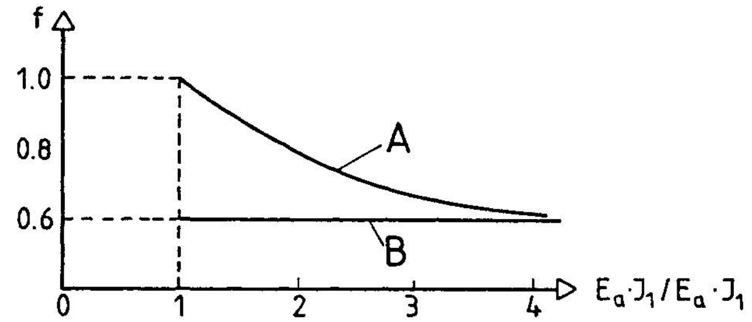


Fig. 36 Reduction factor f for the bending moment at supports

6. CONCLUSION

In this report the rules of Eurocode 4 for the design of composite girders have been explained. The following single subjects had been especially dealt with:

- analysis for continuous girders
- resistances of cross sections
- local buckling
- lateral torsional buckling
- cracking of concrete and serviceability.

With regard to further important questions, which had not been treated herein, for example:

- stud connection between concrete flange and steel beam or
- properties and coefficients of the materials,

it shall be referred to the other reports of the short course.

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