NOTE 3: Elastic-plastic shear lag effects allowing for limited plastic strains may be taken into account using A_{eff} as follows:

$$A_{eff} = A_{c,eff}\beta^{\kappa} \geq A_{c,eff}\beta \qquad (3.5)$$

where β and κ are taken from Table 3.1.

The expressions in NOTE 2 and NOTE 3 may also be applied for flanges in tension in which case $A_{c,eff}$ should be replaced by the gross area of the tension flange.

4 Plate buckling effects due to direct stresses at the ultimate limit state

4.1 General

(1) This section gives rules to account for plate buckling effects from direct stresses at the ultimate limit state when the following criteria are met:

a) The panels are rectangular and flanges are parallel or nearly parallel (see 2.3);

b) Stiffeners, if any, are provided in the longitudinal or transverse direction or both;

c) Open holes and cut outs are small (see 2.3);

d) Members are of uniform cross section;

e) No flange induced web buckling occurs.

NOTE 1: For compression flange buckling in the plane of the web see section 8.

NOTE 2: For stiffeners and detailing of plated members subject to plate buckling see section 9.

4.2 Resistance to direct stresses

(1) The resistance of plated members may be determined [AC1) using the effective[p] areas (AC1] of plate elements in compression for class 4 sections using cross sectional data (A_{eff}, I_{eff}, W_{eff}) for cross sectional verifications and member verifications for column buckling and lateral torsional buckling according to EN 1993-1-1.

(2) Effective[p] areas should be determined on the basis of the linear strain distributions with the attainment of yield strain in the mid plane of the compression plate.

4.3 Effective cross section

(1) In calculating longitudinal stresses, account should be taken of the combined effect of shear lag and plate buckling using the effective areas given in 3.3.

(2) The effective cross sectional properties of members should be based on the effective areas of the compression elements and on the effective[s] area of the tension elements due to shear lag.

(3) The effective area A_{eff} should be determined assuming that the cross section is subject only to stresses due to uniform axial compression. For non-symmetrical cross sections the possible shift e_N of the centroid of the effective area A_{eff} relative to the centre of gravity of the gross cross-section, see Figure 4.1, gives an additional moment which should be taken into account in the cross section verification using 4.6.

(4) The effective section modulus W_{eff} should be determined assuming the cross section is subject only to bending stresses, see Figure 4.2. For biaxial bending effective section moduli should be determined about both main axes.

NOTE: As an alternative to 4.3(3) and (4) a single effective section may be determined from N_{Ed} and M_{Ed} acting simultaneously. The effects of e_N should be taken into account as in 4.3(3). This requires an iterative procedure.

(5) The stress in a flange should be calculated using the elastic section modulus with reference to the mid-plane of the flange.

(6) Hybrid girders may have flange material with yield strength f_{yf} up to $\phi_h \times f_{yw}$ provided that:

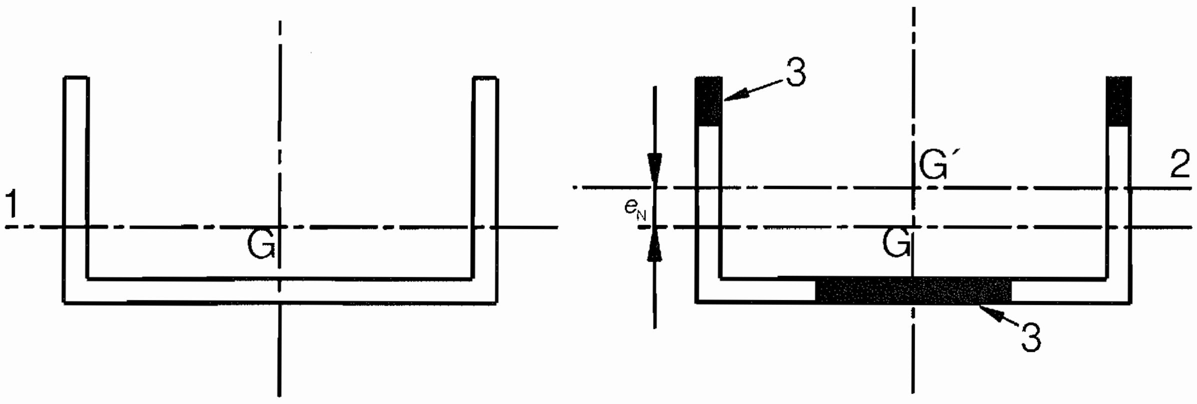
a) the increase of flange stresses caused by yielding of the web is taken into account by limiting the stresses in the web to f_{yw} ;

b) f_{yf} [AC1] *text deleted* [AC1] is used in determining the effective area of the web.

NOTE: The National Annex may specify the value ϕ_h. A value of $\phi_h = 2{,}0$ is recommended.

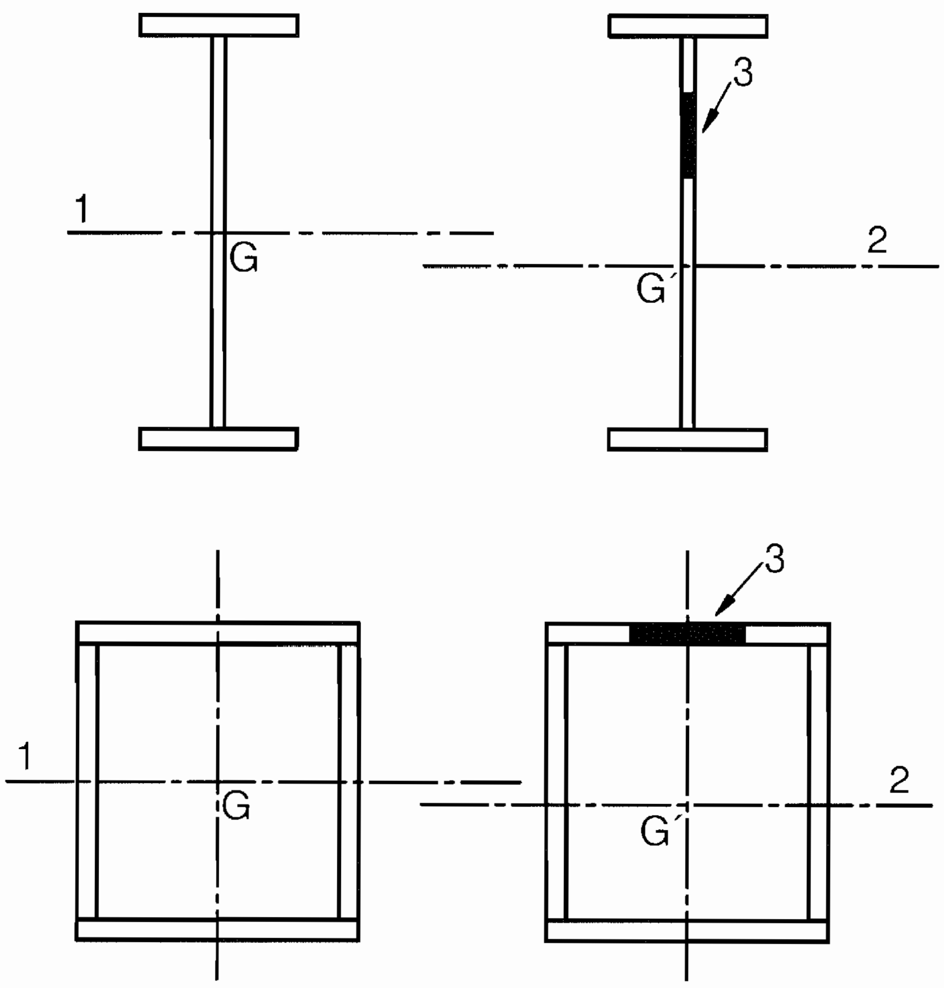
(7) The increase of deformations and of stresses at serviceability and fatigue limit states may be ignored for hybrid girders complying with 4.3(6) including the NOTE.

(8) For hybrid girders complying with 4.3(6) the stress range limit in EN 1993-1-9 may be taken as $1{,}5 f_{yf}$.



G centroid of the gross cross section
G´ centroid of the effective cross section
1 centroidal axis of the gross cross section
2 centroidal axis of the effective cross section
3 non effective zone

Figure 4.1: Class 4 cross-sections - axial force



G centroid of the gross cross section
G´ centroid of the effective cross section
1 centroidal axis of the gross cross section
2 centroidal axis of the effective cross section
3 non effective zone

Figure 4.2: Class 4 cross-sections - bending moment

4.4 Plate elements without longitudinal stiffeners

(1) The effective[p] areas of flat compression elements should be obtained using Table 4.1 for internal elements and Table 4.2 for outstand elements. The effective[p] area of the compression zone of a plate with the gross cross-sectional area A_c should be obtained from:

$$A_{c,eff} = \rho A_c \tag{4.1}$$

where ρ is the reduction factor for plate buckling.

(2) The reduction factor ρ may be taken as follows:

- internal compression elements:

$\rho = 1{,}0$ for [AC1⟩ $\bar{\lambda}_p \leq 0{,}5 + \sqrt{0.085 - 0.055\psi}$ ⟨AC1]

$\rho = \dfrac{\bar{\lambda}_p - 0{,}055(3+\psi)}{\bar{\lambda}_p^2} \leq 1{,}0$ for [AC1⟩ $\bar{\lambda}_p > 0{,}5 + \sqrt{0.085 - 0.055\psi}$, ⟨AC1] [AC1⟩ *text deleted* ⟨AC1] (4.2)

- outstand compression elements:

$\rho = 1{,}0$ for $\bar{\lambda}_p \leq 0{,}748$

$\rho = \dfrac{\bar{\lambda}_p - 0{,}188}{\bar{\lambda}_p^2} \leq 1{,}0$ for $\bar{\lambda}_p > 0{,}748$ (4.3)

where $\bar{\lambda}_p = \sqrt{\dfrac{f_y}{\sigma_{cr}}} = \dfrac{\bar{b}/t}{28{,}4\,\varepsilon\sqrt{k_\sigma}}$

ψ is the stress ratio determined in accordance with 4.4(3) and 4.4(4)

$\bar{b}$ is the appropriate width to be taken as follows (for definitions, see Table 5.2 of EN 1993-1-1)

b_w	for webs;
b	for internal flange elements (except RHS);
b - 3 t	for flanges of RHS;
c	for outstand flanges;
h	for equal-leg angles;
h	for unequal-leg angles;

k_σ is the buckling factor corresponding to the stress ratio ψ and boundary conditions. For long plates k_σ is given in Table 4.1 or Table 4.2 as appropriate;

t is the thickness;

σ_{cr} is the elastic critical plate buckling stress see equation (A.1) in Annex A.1(2) and Table 4.1 and Table 4.2;

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

(3) For flange elements of I-sections and box girders the stress ratio ψ used in Table 4.1 and Table 4.2 should be based on the properties of the gross cross-sectional area, due allowance being made for shear lag in the flanges if relevant. For web elements the stress ratio ψ used in Table 4.1 should be obtained using a stress distribution based on the effective area of the compression flange and the gross area of the web.

NOTE: If the stress distribution results from different stages of construction (as e.g. in a composite bridge) the stresses from the various stages may first be calculated with a cross section consisting of effective flanges and gross web and these stresses are added together. This resulting stress distribution determines an effective web section that can be used for all stages to calculate the final stress distribution for stress analysis.

(4) Except as given in 4.4(5), the plate slenderness $\overline{\lambda}_p$ of an element may be replaced by:

$$\overline{\lambda}_{p,red} = \overline{\lambda}_p \sqrt{\frac{\sigma_{com,Ed}}{f_y / \gamma_{M0}}} \tag{4.4}$$

where $\sigma_{\text{com,Ed}}$ is the maximum design compressive stress in the element determined using the effective[p] area of the section caused by all simultaneous actions.

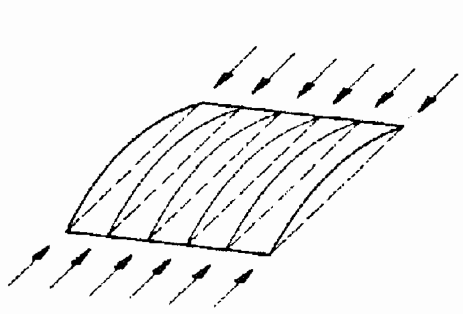
NOTE 1: The above procedure is conservative and requires an iterative calculation in which the stress ratio ψ (see Table 4.1 and Table 4.2) is determined at each step from the stresses calculated on the effective[p] cross-section defined at the end of the previous step.

NOTE 2: See also alternative procedure in Annex E.

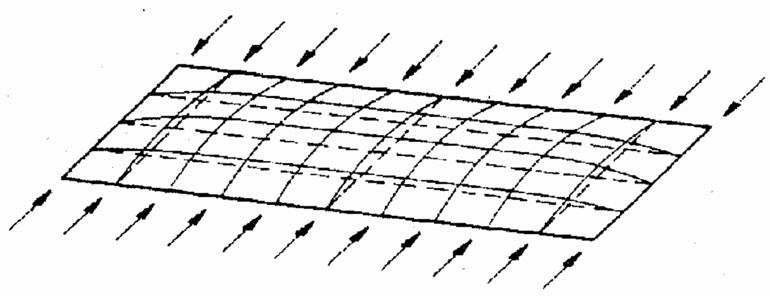
(5) For the verification of the design buckling resistance of a class 4 member using 6.3.1, 6.3.2 or 6.3.4 of EN 1993-1-1, either the plate slenderness $\overline{\lambda}_p$ or $\overline{\lambda}_{p,red}$ with $\sigma_{\text{com,Ed}}$ based on second order analysis with global imperfections should be used.

(6) For aspect ratios $a/b < 1$ a column type of buckling may occur and the check should be performed according to 4.5.4 using the reduction factor ρ_c.

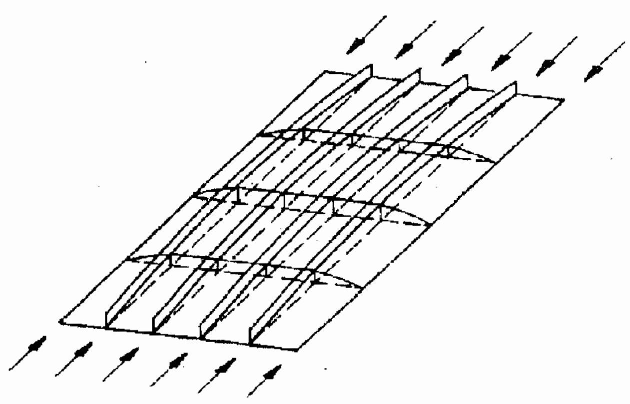
NOTE: This applies e.g. for flat elements between transverse stiffeners where plate buckling could be column-like and require a reduction factor ρ_c close to χ_c as for column buckling, see Figure 4.3 a) and b). For plates with longitudinal stiffeners column type buckling may also occur for $a/b \geq 1$, see Figure 4.3 c).



a) column-like behaviour of plates without longitudinal supports



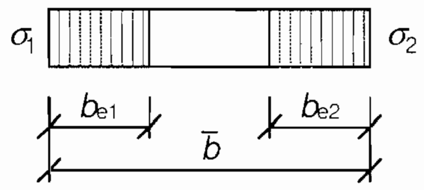
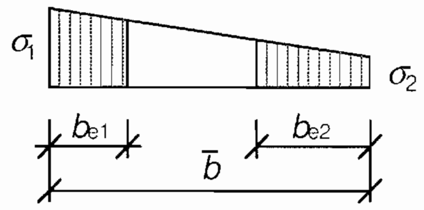
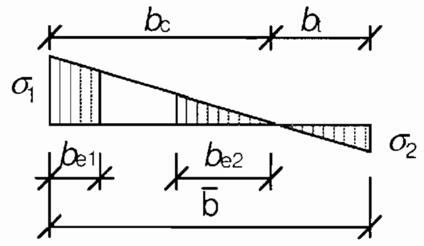
b) column-like behaviour of an unstiffened plate with a small aspect ratio α



c) column-like behaviour of a longitudinally stiffened plate with a large aspect ratio α

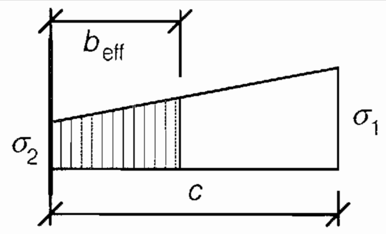
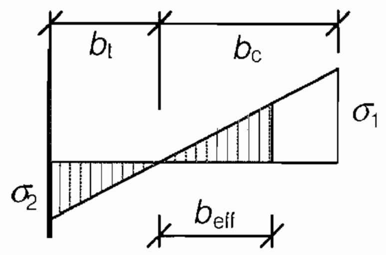
Figure 4.3: Column-like behaviour

Table 4.1: Internal compression elements

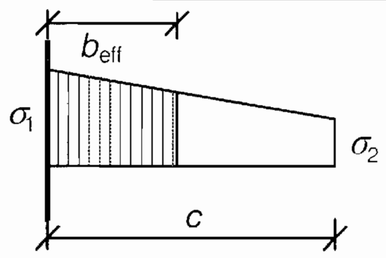
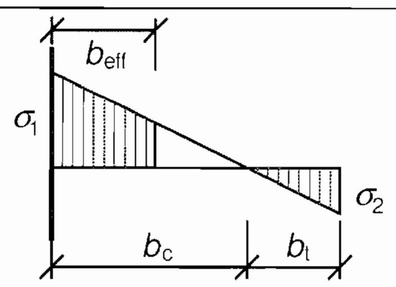
Stress distribution (compression positive)	Effective[p] width b_{eff}
	$\underline{\psi = 1}$: $b_{eff} = \rho\ \bar{b}$ $b_{e1} = 0,5\ b_{eff}$ $\quad b_{e2} = 0,5\ b_{eff}$
	$\underline{1 > \psi \geq 0}$: $b_{eff} = \rho\ \bar{b}$ $b_{e1} = \dfrac{2}{5-\psi} b_{eff}$ $\quad b_{e2} = b_{eff} - b_{e1}$
	$\underline{\psi < 0}$: $b_{eff} = \rho\ b_c = \rho\ \bar{b} / (1-\psi)$ $b_{e1} = 0,4\ b_{eff}$ $\quad b_{e2} = 0,6\ b_{eff}$

$\psi = \sigma_2/\sigma_1$	1	$1 > \psi > 0$	0	$0 > \psi > -1$	-1	AC1⟩ $-1 > \psi \geq -3$ ⟨AC1
Buckling factor k_σ	4,0	$8,2 / (1,05 + \psi)$	7,81	$7,81 - 6,29\psi + 9,78\psi^2$	23,9	$5,98\ (1 - \psi)^2$

Table 4.2: Outstand compression elements

Stress distribution (compression positive)	Effective[p] width b_{eff}
	$\underline{1 > \psi \geq 0}$: $b_{eff} = \rho\ c$
	$\underline{\psi < 0}$: $b_{eff} = \rho\ b_c = \rho\ c / (1-\psi)$

$\psi = \sigma_2/\sigma_1$	1	0	-1	$1 \geq \psi \geq -3$
Buckling factor k_σ	0,43	0,57	0,85	$0,57 - 0,21\psi + 0,07\psi^2$

Stress distribution (compression positive)	Effective[p] width b_{eff}
	$\underline{1 > \psi \geq 0}$: $b_{eff} = \rho\ c$
	$\underline{\psi < 0}$: $b_{eff} = \rho\ b_c = \rho\ c / (1-\psi)$

$\psi = \sigma_2/\sigma_1$	1	$1 > \psi > 0$	0	$0 > \psi > -1$	-1
Buckling factor k_σ	0,43	$0,578 / (\psi + 0,34)$	1,70	$1,7 - 5\psi + 17,1\psi^2$	23,8