6.2 Design Resistance

6.2.1 Internal forces

(1) The stresses due to the internal forces and moments in a member may be assumed not to affect the design resistances of the basic components of a joint, except as specified in 6.2.1(2) and 6.2.1(3).

(2) The longitudinal stress in a column should be taken into account when determining the design resistance of the column web in compression, see 6.2.6.2(2).

(3) The shear in a column web panel should be taken into account when determining the design resistance of the following basic components:

- column web in transverse compression, see 6.2.6.2;
- column web in transverse tension, see 6.2.6.3.

6.2.2 Shear forces

(1) In welded connections, and in bolted connections with end-plates, the welds connecting the beam web should be designed to transfer the shear force from the connected beam to the joint, without any assistance from the welds connecting the beam flanges.

(2) In bolted connections with end-plates, the design resistance of each bolt-row to combined shear and tension should be verified using the criterion given in Table 3.4, taking into account the total tensile force in the bolt, including any force due to prying action.

NOTE: As a simplification, bolts required to resist in tension may be assumed to provide their full design resistance in tension when it can be shown that the design shear force does not exceed the sum of:

a) the total design shear resistance of those bolts that are not required to resist tension and;

b) (0,4/1,4) times the total design shear resistance of those bolts that are also required to resist tension.

(3) In bolted connections with angle flange cleats, the cleat connecting the compression flange of the beam may be assumed to transfer the shear force in the beam to the column, provided that:

- the gap g between the end of the beam and the face of the column does not exceed the thickness t_a of the angle cleat;
- the force does not exceed the design shear resistance of the bolts connecting the cleat to the column;
- the web of the beam satisfies the requirement given in EN 1993-1-5, section 6.

(4) The design shear resistance of a joint may be derived from the distribution of internal forces within that joint, and the design resistances of its basic components to these forces, see Table 6.1.

(5) In base plates if no special elements for resisting shear are provided, such as block or bar shear connectors, it should be demonstrated [AC2⟩ that the design friction resistance of the base plate, see 6.2.2(6), and, in cases where the bolt holes are not oversized, the design shear resistance of the anchor bolts, see 6.2.2(7), added up is sufficient ⟨AC2] to transfer the design shear force. The design bearing resistance of the block or bar shear connectors with respect to the concrete should be checked according to EN 1992.

(6) In a column base the design friction resistance $F_{f,Rd}$ between base plate and grout should be derived as follows:

$$F_{f,Rd} = C_{f,d} \; N_{c,Ed} \qquad \text{... (6.1)}$$

where:

$C_{f,d}$ is the coefficient of friction between base plate and grout layer. The following values may be used:

- for sand-cement mortar $C_{f,d} = 0{,}20$;
- for other types of grout the coefficient of friction $C_{f,d}$ should be determined by testing in accordance with EN 1990, Annex D;

$N_{c,Ed}$ is the design value of the normal compressive force in the column.

NOTE: If the column is loaded by a tensile normal force, $F_{f,Rd} = 0$.

(7) In a column base the design shear resistance of an anchor bolt $F_{vb,Rd}$ should be taken as the smaller of $F_{1,vb,Rd}$ and $F_{2,vb,Rd}$ where:

- $F_{1,vb,Rd}$ is the design shear resistance of the anchor bolt, see 3.6.1
- AC2 $F_{2,vb,Rd} = \dfrac{\alpha_{bc} f_{ub} A_s}{\gamma_{M2}}$ AC2 ... (6.2)

where:

α_b $= 0{,}44 - 0{,}0003 f_{yb}$

f_{yb} is the yield strength of the anchor bolt, where 235 N/mm$^2 \leq f_{yb} \leq$ 640 N/mm^2

(8) AC2 The design shear resistance $F_{v,Rd}$ between a column base plate and a grout layer AC2 should be derived as follows:

$$F_{v,Rd} = F_{f,Rd} + n F_{vb,Rd} \quad \text{... (6.3)}$$

where:

n is the number of anchor bolts in the base plate.

(9) The concrete and reinforcement used in the base should be designed in accordance with EN 1992.

6.2.3 Bending moments

(1) The design moment resistance of any joint may be derived from the distribution of internal forces within that joint and the design resistances of its basic components to these forces, see Table 6.1.

(2) Provided that the axial force N_{Ed} in the connected member does not exceed 5% of the design resistance $N_{p\ell,Rd}$ of its cross-section, the design moment resistance $M_{j,Rd}$ of a beam-to column joint or beam splice may be determined using the method given in 6.2.7.

(3) The design moment resistance $M_{j,Rd}$ of a column base may be determined using the method given in 6.2.8.

(4) In all joints, the sizes of the welds should be such that the design moment resistance of the joint $M_{j,Rd}$ is always limited by the design resistance of its other basic components, and not by the design resistance of the welds.

(5) In a beam-to-column joint or beam splice in which a plastic hinge is required to form and rotate under any relevant load case, the welds should be designed to resist the effects of a moment at least equal to the smaller of:

- the design plastic moment resistance of the connected member $M_{p\ell,Rd}$
- α times the design moment resistance of the joint $M_{j,Rd}$

where:

$\alpha = 1{,}4$ - for frames in which the bracing system satisfies the criterion (5.1) in EN 1993-1-1 clause 5.2.1(3) with respect to sway;

$\alpha = 1{,}7$ - for all other cases.

(6) In a bolted connection with more than one bolt-row in tension, as a simplification the contribution of any bolt-row may be neglected, provided that the contributions of all other bolt-rows closer to the centre of compression are also neglected.

6.2.4 Equivalent T-stub in tension

6.2.4.1 General

(1) In bolted connections an equivalent T-stub in tension may be used to model the design resistance of the following basic components:

- column flange in bending;
- end-plate in bending;
- flange cleat in bending;
- base plate in bending under tension.

(2) Methods for modelling these basic components as equivalent T-stub flanges, including the values to be used for e_{min}, ℓ_{eff} and m, are given in 6.2.6.

(3) The possible modes of failure of the flange of an equivalent T-stub may be assumed to be similar to those expected to occur in the basic component that it represents.

(4) The total effective length $\sum\ell_{eff}$ of an equivalent T-stub, see Figure 6.2, should be such that the design resistance of its flange is equivalent to that of the basic joint component that it represents.

NOTE: The effective length of an equivalent T-stub is a notional length and does not necessarily correspond to the physical length of the basic joint component that it represents.

(5) The design tension resistance of a T-stub flange should be determined from Table 6.2.

NOTE: Prying effects are implicitly taken into account when determining the design tension resistance according to Table 6.2.

(6) In cases where prying forces may develop, see Table 6.2, the design tension resistance of a T-stub flange $F_{T,Rd}$ should be taken as the smallest value for the three possible failure modes 1, 2 and 3.

(7) In cases where prying forces may not develop the design tension resistance of a T-stub flange $F_{T,Rd}$ should be taken as the smallest value for the two possible failure modes according to Table 6.2.

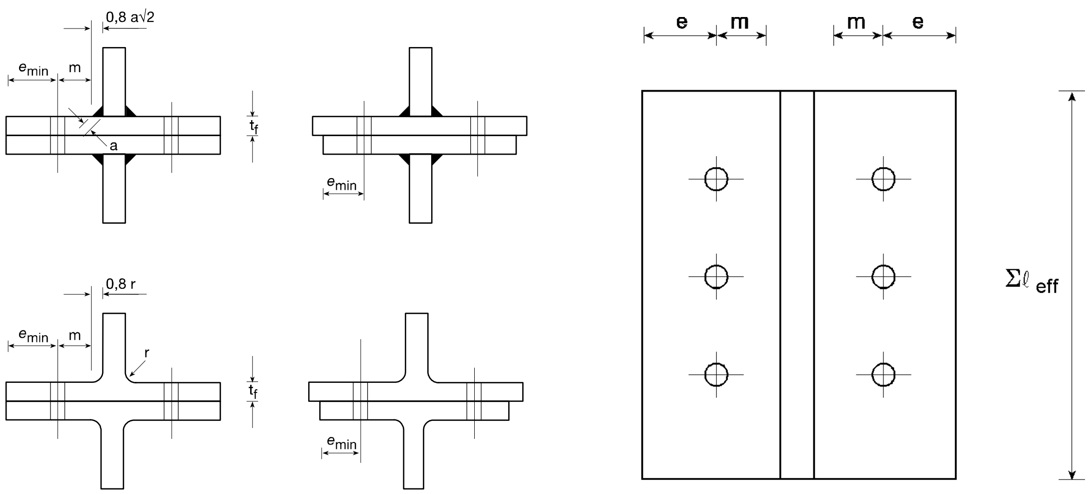


Figure 6.2: Dimensions of an equivalent T-stub flange

Table 6.2: Design Resistance $F_{T,Rd}$ of a T-stub flange

<table>
<tr><td></td><td colspan="2">Prying forces may develop, i.e. $L_b \le L_b^*$</td><td>No prying forces</td></tr>
<tr><td>Mode 1</td><td>Method 1</td><td>Method 2 (alternative method)</td><td rowspan="4">$F_{T,1\text{-}2,Rd} = \dfrac{2M_{pl,1,Rd}}{m}$</td></tr>
<tr><td>without backing plates</td><td>$F_{T,1,Rd} = \dfrac{4M_{pl,1,Rd}}{m}$</td><td>$F_{T,1,Rd} = \dfrac{(8n-2e_w)M_{pl,1,Rd}}{2mn-e_w(m+n)}$</td></tr>
<tr><td>with backing plates</td><td>$F_{T,1,Rd} = \dfrac{4M_{pl,1,Rd}+2M_{bp,Rd}}{m}$</td><td>$F_{T,1,Rd} = \dfrac{(8n-2e_w)M_{pl,1,Rd}+4nM_{bp,Rd}}{2mn-e_w(m+n)}$</td></tr>
<tr><td>Mode 2</td><td colspan="2">$F_{T,2,Rd} = \dfrac{2M_{pl,2,Rd}+n\Sigma F_{t,Rd}}{m+n}$</td></tr>
<tr><td>Mode 3</td><td colspan="3">$F_{T,3,Rd} = \Sigma F_{t,Rd}$</td></tr>
</table>

Mode 1: Complete yielding of the flange
Mode 2: Bolt failure with yielding of the flange
Mode 3: Bolt failure

L_b is - the bolt elongation length, taken equal to the grip length (total thickness of material and washers), plus half the sum of the height of the bolt head and the height of the nut or
- the anchor bolt elongation length, taken equal to the sum of 8 times the nominal bolt diameter, the grout layer, the plate thickness, the washer and half the height of the nut

AC2 $L_b^* = \dfrac{8{,}8m^3 A_s n_b}{\Sigma \ell_{eff,1} t_f^{\,3}}$ AC2

$F_{T,Rd}$ is the design tension resistance of a T-stub flange
Q is the prying force

$M_{p\ell,1,Rd} = 0{,}25\Sigma \ell_{eff,1} t_f^{\,2} f_y / \gamma_{M0}$

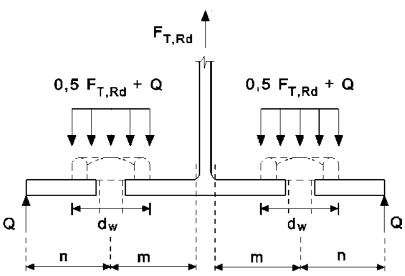
$M_{p\ell,2,Rd} = 0{,}25\Sigma \ell_{eff,2} t_f^{\,2} f_y / \gamma_{M0}$

$M_{bp,Rd} = 0{,}25\Sigma \ell_{eff,1} t_{bp}^{\,2} f_{y,bp} / \gamma_{M0}$

$n = e_{min}$ but $n \le 1{,}25m$

AC2 n_b is the number of bolt rows (with 2 bolts per row) AC2

$F_{t,Rd}$ is the design tension resistance of a bolt, see Table 3.4;
$\sum F_{t,Rd}$ is the total value of $F_{t,Rd}$ for all the bolts in the T-stub;
$\sum \ell_{eff,1}$ is the value of $\sum \ell_{eff}$ for mode 1;
$\sum \ell_{eff,2}$ is the value of $\sum \ell_{eff}$ for mode 2;
e_{min}, m and t_f are as indicated in Figure 6.2.
$f_{y,bp}$ is the yield strength of the backing plates;
t_{bp} is the thickness of the backing plates;
$e_w = d_w / 4$;
d_w is the diameter of the washer, or the width across points of the bolt head or nut, as relevant.



NOTE 1: In bolted beam-to-column joints or beam splices it may be assumed that prying forces will develop.

NOTE 2: In method 2, the force applied to the T-stub flange by a bolt is assumed to be uniformly distributed under the washer, the bolt head or the nut, as appropriate, see figure, instead of concentrated at the centre-line of the bolt. This assumption leads to a higher value for mode 1, but leaves the values for $F_{T,1\text{-}2,Rd}$ and modes 2 and 3 unchanged.

6.2.4.2 Individual bolt-rows, bolt-groups and groups of bolt-rows

(1) Although in an actual T-stub flange the forces at each bolt-row are generally equal, when an equivalent T-stub flange is used to model a basic component listed in 6.2.4.1(1), allowance should be made for the different in forces at each bolt-row.

(2) When using the equivalent T-stub approach to model a group of bolt rows it may be necessary to divide the group into separate bolt-rows and use an equivalent T-stub to model each separate bolt-row.

(3) When using the T-stub approach to model a group of bolt rows the following conditions should be satisfied:

a) the force at each bolt-row should not exceed the design resistance determined considering only that individual bolt-row;

b) the total force on each group of bolt-rows, comprising two or more adjacent bolt-rows within the same bolt-group, should not exceed the design resistance of that group of bolt-rows.

(4) When determining the design tension resistance of a basic component represented by an equivalent T-stub flange, the following parameters should be calculated:

a) the design resistance of an individual bolt-row, determined considering only that bolt-row;

b) the contribution of each bolt-row to the design resistance of two or more adjacent bolt-rows within a bolt-group, determined considering only those bolt-rows.

(5) In the case of an individual bolt-row $\sum \ell_{eff}$ should be taken as equal to the effective length ℓ_{eff} tabulated in 6.2.6 for that bolt-row taken as an individual bolt-row.

(6) In the case of a group of bolt-rows $\sum \ell_{eff}$ should be taken as the sum of the effective lengths ℓ_{eff} tabulated in 6.2.6 for each relevant bolt-row taken as part of a bolt-group.

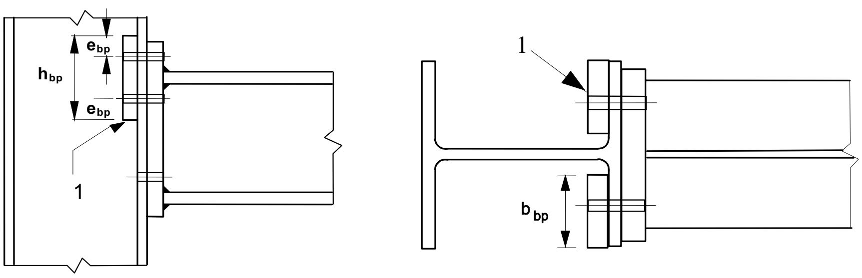
6.2.4.3 Backing plates

(1) Backing plates may be used to reinforce a column flange in bending as indicated in Figure 6.3.

(2) Each backing plate should extend at least to the edge of the column flange, and to within 3 mm of the toe of the root radius or of the weld.

(3) The backing plate should extend beyond the furthermost bolt rows active in tension as defined in Figure 6.3.

(4) Where backing plates are used, the design resistance of the T-stub $F_{T,Rd}$ should be determined using the method given in Table 6.2.



$h_{bp} \geq \sum \ell_{eff,1}$

$e_{bp} \geq 2d$

1 Backing plate

Figure 6.3: Column flange with backing plates

6.2.5 Equivalent T-stub in compression

(1) In steel- to-concrete joints, the flange of an equivalent T-stub in compression may be used to model the design resistances for the combination of the following basic components:

- the steel base plate in bending under the bearing pressure on the foundation;
- the concrete and/or grout joint material in bearing.

(2) The total effective length l_{eff} and the total effective width b_{eff} of an equivalent T-stub should be such that the design compression resistance of the T-stub is equivalent to that of the basic joint component it represents.

NOTE: AC2 The values for the effective length and the effective width of an equivalent T-stub are notional values for these lengths AC2 and may differ to the physical dimensions of the basic joint component it represents.

(3) The design compression resistance of a T-stub flange $F_{C,Rd}$ should be determined as follows:

$$F_{C,Rd} = f_{jd}\, b_{eff}\, l_{eff} \qquad \text{... (6.4)}$$

where:

b_{eff} is the effective width of the T-stub flange, see 6.2.5(5) and 6.2.5(6)

l_{eff} is the effective length of the T-stub flange, see 6.2.5(5) and 6.2.5(6)

f_{jd} is the design bearing strength of the joint, see 6.2.5(7)

(4) The forces transferred through a T-stub should be assumed to spread uniformly as shown in Figure 6.4(a) and (b). The pressure on the resulting bearing area should not exceed the design bearing strength f_{jd} and the additional bearing width, c, should not exceed:

$$c = t\,[f_y / (3\, f_{jd}\, \gamma_{M0})]^{0.5} \qquad \text{... (6.5)}$$

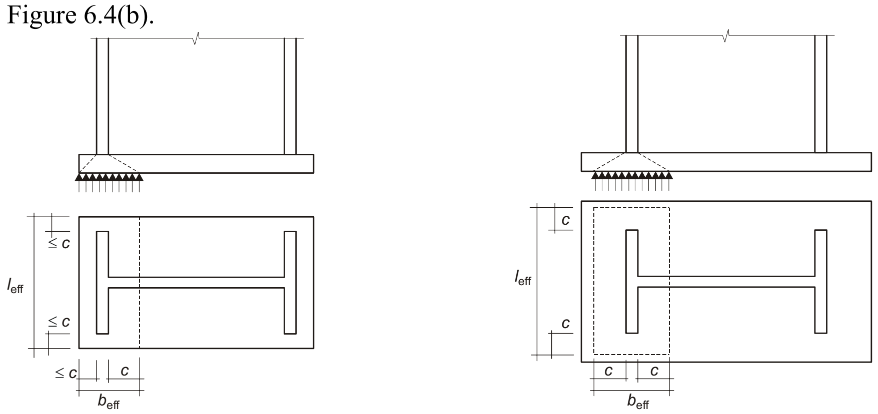
where:

t is the thickness of the T-stub flange;

f_y is the yield strength of the T-stub flange.

(5) Where the projection of the physical length of the basic joint component represented by the T-stub is less than c, the effective area should be taken as indicated in Figure 6.4(a)

(6) Where the projection of the physical length of the basic joint component represented by the T-stub exceeds c on any side, the part of the additional projection beyond the width c should be neglected, see Figure 6.4(b).



(a) Short projection (b) Large projection

Figure 6.4: Area of equivalent T-Stub in compression

(7) The design bearing strength of the joint f_{jd} should be determined from:

$$f_{jd} = \beta_j F_{Rdu} / (b_{eff} l_{eff}) \qquad \text{... (6.6)}$$

where:

β_j is the foundation joint material coefficient, which may be taken as 2/3 provided that the characteristic strength of the grout is not less than 0,2 times the characteristic strength of the concrete foundation and the thickness of the grout is not greater than 0,2 times the smallest width of the steel base plate. In cases where the thickness of the grout is more than 50 mm, the characteristic strength of the grout should be at least the same as that of the concrete foundation.

F_{Rdu} is the concentrated design resistance force given in EN 1992, where A_{c0} is to be taken as (b_{eff} l_{eff}).

6.2.6 Design Resistance of basic components

6.2.6.1 Column web panel in shear

(1) The design methods given in 6.2.6.1(2) to 6.2.6.1(14) are valid provided the column web slenderness satisfies the condition [AC2⟩ $d_c/t_w \leq 69\varepsilon$ ⟨AC2].

(2) For a single-sided joint, or for a double-sided joint in which the beam depths are similar, the design plastic shear resistance $V_{wp,Rd}$ of an unstiffened column web panel, subject to a design shear force $V_{wp,Ed}$, see 5.3(3), should be obtained using:

$$V_{wp,Rd} = \frac{0{,}9 f_{y,wc} A_{vc}}{\sqrt{3}\,\gamma_{M0}} \qquad \text{... (6.7)}$$

where:

A_{vc} is the shear area of the column, see EN 1993-1-1.

(3) The design shear resistance may be increased by the use of stiffeners or supplementary web plates.

(4) Where transverse web stiffeners are used in both the compression zone and the tension zone, the design plastic shear resistance of the column web panel $V_{wp,Rd}$ may be increased by $V_{wp,add,Rd}$ given by:

$$V_{wp,add,Rd} = \frac{4 M_{pl,fc,Rd}}{d_s} \quad \text{but} \quad V_{wp,add,Rd} \leq \frac{2 M_{pl,fc,Rd} + 2 M_{pl,st,Rd}}{d_s} \qquad \text{... (6.8)}$$

where:

d_s is the distance between the centrelines of the stiffeners;

$M_{p\ell,fc,Rd}$ is the design plastic moment resistance of a column flange

$M_{p\ell,st,Rd}$ is the design plastic moment resistance of a stiffener.

NOTE: In welded joints, the transverse stiffeners should be aligned with the corresponding beam flange.

(5) When diagonal web stiffeners are used the design plastic shear resistance of a column web should be determined according to EN 1993-1-1.

NOTE: In double-sided beam-to-column joint configurations without diagonal stiffeners on the column webs, the two beams are assumed to have similar depths.

(6) Where a column web is reinforced by adding a supplementary web plate, see Figure 6.5, the shear area A_{vc} may be increased by $b_s t_{wc}$. If a further supplementary web plate is added on the other side of the web, no further increase of the shear area should be made.

(7) Supplementary web plates may also be used to increase the rotational stiffness of a joint by increasing the stiffness of the column web in shear, compression or tension, see 6.3.2(1).

(8) The steel grade of the supplementary web plate should be equal to that of the column.

(9) The width b_s should be such that the supplementary web plate extends at least to the toe of the root radius or of the weld.

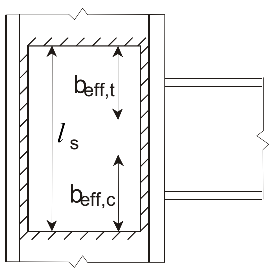
(10) The length ℓ_s should be such that the supplementary web plate extends throughout the effective width of the web in tension and compression, see Figure 6.5.

(11) The thickness t_s of the supplementary web plate should be not less than the column web thickness t_{wc}.

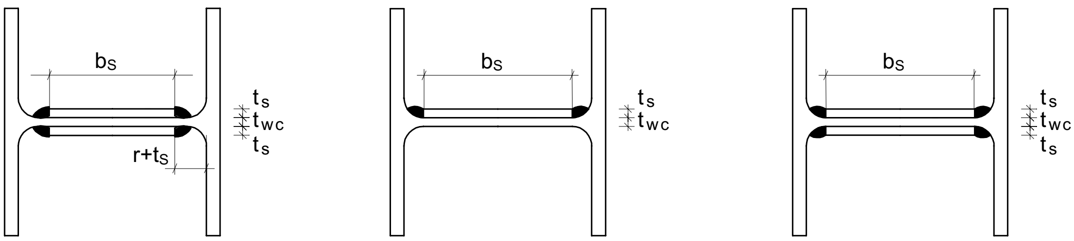
(12) The welds between the supplementary web plate and profile should be designed to resist the applied design forces.

(13) The width b_s of a supplementary web plate should be less than $40\varepsilon\, t_s$.

(14) Discontinuous welds may be used in non corrosive environments.



a) Layout



NOTE: Weldability at the corner should be taken into account.

b) Examples of cross-section with longitudinal welds

Figure 6.5: Examples of supplementary web plates

6.2.6.2 Column web in transverse compression

(1) The design resistance of an unstiffened column web subject to transverse compression should be determined from:

$$F_{c,wc,Rd} = \frac{\omega\, k_{wc}\, b_{eff,c,wc}\, t_{wc}\, f_{y,wc}}{\gamma_{M0}} \quad \text{but} \quad F_{c,wc,Rd} \le \frac{\omega\, k_{wc}\, \rho\, b_{eff,c,wc}\, t_{wc}\, f_{y,wc}}{\gamma_{M1}} \qquad \text{... (6.9)}$$

where:

ω is a reduction factor to allow for the possible effects of interaction with shear in the column web panel according to Table 6.3;

$b_{\mathrm{eff,c,wc}}$ is the effective width of column web in compression:

– for a welded connection:

$$b_{\mathrm{eff,c,wc}} = t_{fb} + 2\sqrt{2}\, a_b + 5(t_{fc} + s) \qquad \text{... (6.10)}$$

a_c, r_c and a_b are as indicated in Figure 6.6.

– for bolted end-plate connection:

$$b_{\mathrm{eff,c,wc}} = t_{fb} + 2\sqrt{2}\, a_p + 5(t_{fc} + s) + s_p \qquad \text{... (6.11)}$$

s_p is the length obtained by dispersion at 45° through the end-plate (at least t_p and, provided that the length of end-plate below the flange is sufficient, up to $2t_p$).

– for bolted connection with angle flange cleats:

$$b_{\mathrm{eff,c,wc}} = 2t_a + 0{,}6r_a + 5(t_{fc} + s) \qquad \text{... (6.12)}$$

– for a rolled I or H section column: $s = r_c$

– for a welded I or H section column: $s = \sqrt{2}a_c$

ρ is the reduction factor for plate buckling:

– if $\bar{\lambda}_p \leq 0{,}72$: $\rho = 1{,}0$ ... (6.13a)

– if $\bar{\lambda}_p > 0{,}72$: $\rho = (\bar{\lambda}_p - 0{,}2)/\bar{\lambda}_p^{\,2}$ ... (6.13b)

$\bar{\lambda}_p$ is the plate slenderness:

$$\bar{\lambda}_p = 0{,}932\sqrt{\frac{b_{eff,c,wc}\, d_{wc}\, f_{y,wc}}{E t_{wc}^{\,2}}} \qquad \text{... (6.13c)}$$

– for a rolled I or H section column: $d_{wc} = h_c - 2(t_{fc} + r_c)$

– for a welded I or H section column: $d_{wc} = h_c - 2(t_{fc} + \sqrt{2}a_c)$

k_{wc} is a reduction factor and is given in 6.2.6.2(2).

Table 6.3: Reduction factor ω for interaction with shear

Transformation parameter β	Reduction factor ω
$0 \leq \beta \leq 0{,}5$	$\omega = 1$
$0{,}5 < \beta < 1$	$\omega = \omega_1 + 2(1-\beta)(1-\omega_1)$
$\beta = 1$	$\omega = \omega_1$
$1 < \beta < 2$	$\omega = \omega_1 + (\beta - 1)(\omega_2 - \omega_1)$
$\beta = 2$	$\omega = \omega_2$
$\omega_1 = \dfrac{1}{\sqrt{1 + 1{,}3(b_{eff,c,wc}\, t_{wc} / A_{vc})^2}}$	$\omega_2 = \dfrac{1}{\sqrt{1 + 5{,}2(b_{eff,c,wc}\, t_{wc} / A_{vc})^2}}$
A_{vc} is the shear area of the column, see 6.2.6.1; β is the transformation parameter, see 5.3(7).	

(2) Where the maximum longitudinal compressive stress $\sigma_{com,Ed}$ due to axial force and bending moment in the column exceeds $0,7 f_{y,wc}$ in the web (adjacent to the root radius for a rolled section or the toe of the weld for a welded section), its effect on the design resistance of the column web in compression should be allowed for by multiplying the value of $F_{c,wc,Rd}$ given by expression (6.9) by a reduction factor k_{wc} as follows:

– when $\sigma_{com,Ed} \leq 0,7 f_{y,wc}$: $k_{wc} = 1$

– when $\sigma_{com,Ed} > 0,7 f_{y,wc}$: $k_{wc} = 1,7 - \sigma_{com,Ed} / f_{y,wc}$ ... (6.14)

NOTE: Generally the reduction factor k_{wc} is $1,0$ and no reduction is necessary. It can therefore be omitted in preliminary calculations when the longitudinal stress is unknown and checked later.

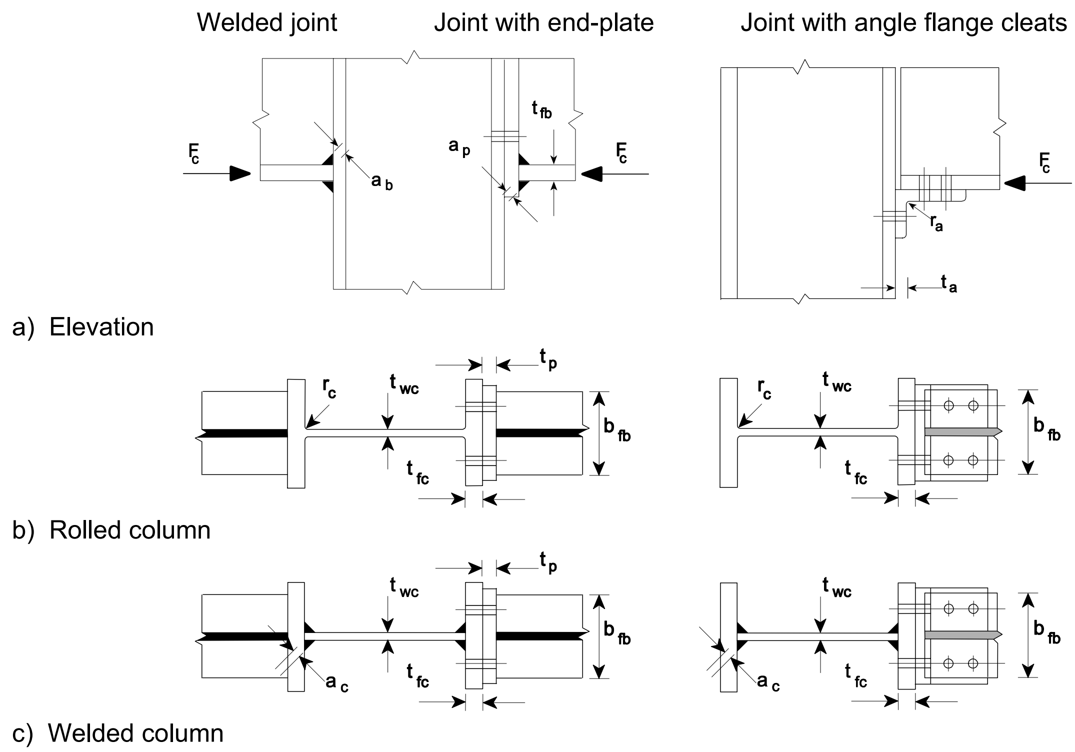


Figure 6.6: Transverse compression on an unstiffened column

(3) The 'column-sway' buckling mode of an unstiffened column web in compression illustrated in Figure 6.7 should normally be prevented by constructional restraints.

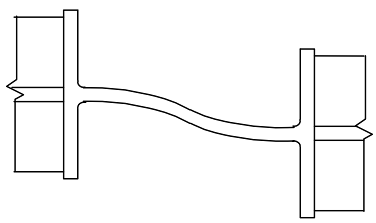


Figure 6.7: 'Column-sway' buckling mode of an unstiffened web

(4) Stiffeners or supplementary web plates may be used to increase the design resistance of a column web in transverse compression.