### 6.7.5.2 Design resistance

(1) For un-stiffened or stiffened webs the design resistance $F_{Rd}$ to local buckling under transverse loads should be taken as

$$F_{Rd} = L_{eff}\, t_w\, f_{ow} / \gamma_{M1} \tag{6.134}$$

where:

$f_{ow}$ is the characteristic value of strength of the web material.

$L_{eff}$ is the effective length for resistance to transverse loads, which should be determined from

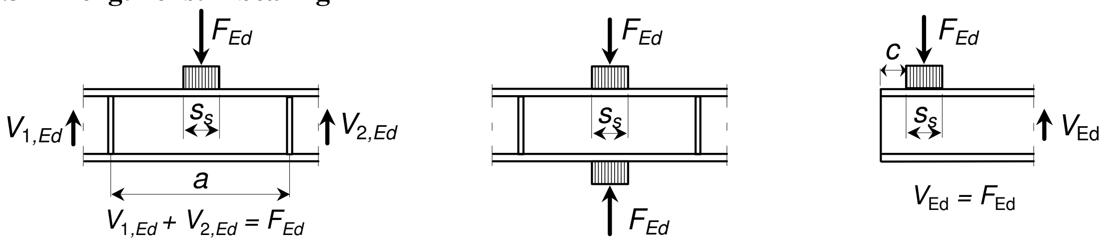
$$L_{eff} = \chi_F l_y \tag{6.135}$$

where:

$l_y$ is the effective loaded length, see 6.7.5.5, appropriate to the length of stiff bearings $s_s$, see 6.7.5.3

$\chi_F$ is the reduction factor due to local buckling, see 6.7.5.4.

### 6.7.5.3 Length of stiff bearing




$$k_F = 6 + 2\left(\frac{b_w}{a}\right)^2$$

Type (a)

$$k_F = 3{,}5 + 2\left(\frac{b_w}{a}\right)^2$$

Type (b)

$$k_F = 2 + 6\frac{s_s + c}{b_w} \le 6$$

Type (c)

**Figure 6.30 - Load applications and buckling coefficients**

(1) The length of stiff bearing, $s_s$, on the flange is the distance over which the applied load is effectively distributed and it may be determined by dispersion of load through solid material at a slope of 1:1, see Figure 6.31. However, $s_s$ should not be taken as larger than $b_w$.

(2) If several concentrated loads are closely spaced ($s_s$ for individual loads > distance between loads), the resistance should be checked for each individual load as well as for the total load with $s_s$ as the centre-to-centre distance between the outer loads.

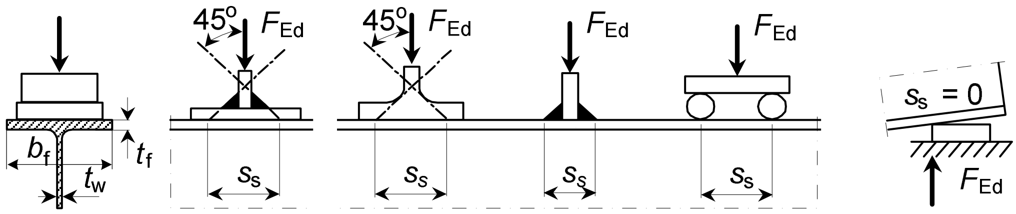



**Figure 6.31 - Length of stiff bearing**

### 6.7.5.4 Reduction factor $\chi_F$ for resistance

(1) The reduction factor $\chi_F$ for resistance should be obtained from:

$$\chi_F = \frac{0{,}5}{\lambda_F} \text{ but not more than } 1{,}0 \tag{6.136}$$

where:

$$\lambda_F = \sqrt{\frac{l_y t_w f_{ow}}{F_{cr}}} \tag{6.137}$$

$$F_{cr} = 0{,}9 k_F E t_w^3 / h_w \tag{6.138}$$

$l_y$ is effective loaded length obtained from 6.7.5.5.

(2) For webs without longitudinal stiffeners the factor $k_F$ should be obtained from Figure 6.30

(3) For webs with longitudinal stiffeners $k_F$ should be taken as

$$k_F = 6 + 2(h_w / a)^2 + (5{,}44 b_1 / a - 0{,}21)\sqrt{\gamma_s} \tag{6.139}$$

where:

$b_1$ is the depth of the loaded sub-panel taken as the clear distance between the loaded flange and the stiffener

$$\gamma_s = 10{,}9 I_{sl} / (h_w t_w^3) \quad \leq 13(a / h_w)^3 + 210(0{,}3 - b_1 / h_w) \tag{6.140}$$

where $I_{sl}$ is the second moment of area (about z–z axis) of the stiffener closest to the loaded flange including contributing parts of the web according to Figure 6.29. Equation (6.140) is valid for $0{,}05 \leq b_1 / h_w \leq 0{,}3$ and loading according to type (a) in Figure 6.30.

#### 6.7.5.5 Effective loaded length

(1) The effective loaded length $l_y$ should be calculated using the two dimensionless parameters $m_1$ and $m_2$ obtained from

$$m_1 = \frac{f_{of} b_f}{f_{ow} t_w} \tag{6.141}$$

$$m_2 = 0{,}02 \left(\frac{h_w}{t_f}\right)^2 \quad \text{if} \quad \lambda_F > 0{,}5 \quad \text{otherwise} \quad m_2 = 0 \tag{6.142}$$

where $b_f$ is the flange width, see Figure 6.31. For box girders, $b_f$ in expression (6.141) is limited to $15 t_f$ on each side of the web.

(2) For cases (a) and (b) in Figure 6.30, $l_y$ should be obtained using:

$$l_y = s_s + 2 t_f \left(1 + \sqrt{m_1 + m_2}\right), \text{ but } l_y \leq \text{ distance between adjacent transverse stiffeners} \tag{6.143}$$

(3) For case (c) in Figure 6.30, $l_y$ should be obtained as the smaller of the values obtained from the equations (6.143), (6.144) and (6.145). However, $s_s$ in (6.143) should be taken as zero if the structure that introduces the force does not follow the slope of the girder, see Figure 6.31.

$$l_y = l_e + t_f \sqrt{\frac{m_1}{2} + \left(\frac{l_e}{t_f}\right)^2 + m_2} \tag{6.144}$$

$$l_y = l_e + t_f \sqrt{m_1 + m_2} \tag{6.145}$$

$$l_e = \frac{k_F E t_w^2}{2 f_{ow} h_w} \le s_s + c \qquad (6.146)$$

## 6.7.6 Interaction

### 6.7.6.1 Interaction between shear force, bending moment and axial force

(1) Provided that the flanges can resist the whole of the design value of the bending moment and axial force in the member, the design shear resistance of the web need not be reduced to allow for the moment and axial force in the member, except as given in 6.7.4.2(10).

(2) If $M_{Ed} > M_{f,Rd}$ the following two expressions (corresponding to curve (2) and (3) in Figure 6.32) should be satisfied:

$$\frac{M_{Ed} + M_{f,Rd}}{2M_{pl,Rd}} + \frac{V_{Ed}}{V_{w,Rd}}\left(1 - \frac{M_{f,Rd}}{M_{pl,Rd}}\right) \le 1{,}00 \qquad (6.147)$$

$$M_{Ed} \le M_{c,Rd}$$

where:

$M_{c,Rd}$ is the design bending moment resistance according to 6.7.2 (4).

$M_{f,Rd}$ is the design bending moment resistance of the flanges only, see 6.7.5(9).

$M_{pl,Rd}$ is the plastic design bending moment resistance

(3) If an axial force $N_{Ed}$ is also applied, then $M_{pl,Rd}$ should be replaced by the reduced plastic moment resistance $M_{N,Rd}$ given by

$$M_{N,Rd} = M_{pl,Rd}\left(1 - \left(\frac{N_{Ed}}{(A_{f1} + A_{f2}) f_o / \gamma_{M1}}\right)^2\right) \qquad (6.148)$$

where $A_{f1}$, $A_{f2}$ are the areas of the flanges.

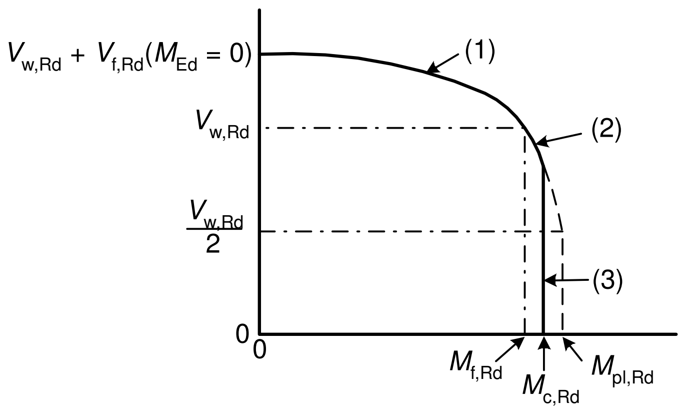



**Figure 6.32 - Interaction of shear force resistance and bending moment resistance**

### 6.7.6.2 Interaction between transverse force, bending moment and axial force

(1) If the girder is subjected to a concentrated force acting on the compression flange in conjunction with bending moment and axial force, the resistance should be verified using 6.2.9, 6.7.5.1 and the following interaction expression

$$\frac{F_{Ed}}{F_{Rd}} + 0{,}8\left(\frac{M_{Ed}}{M_{c,Rd}} + \frac{N_{Ed}}{N_{c,Rd}}\right) \le 1{,}4 \qquad (6.149)$$

where:

$M_{c,Rd}$ is the design bending moment resistance according to 6.7.2 (4).

$N_{c,Rd}$ is the design axial force resistance, see 6.3.1.1.

(2) If the concentrated force is acting on the tension flange the resistance according to 6.7.5 should be verified and in addition also 6.2.1(5)

### 6.7.7 Flange induced buckling

(1) To prevent the possibility of the compression flange buckling in the plane of the web, the ratio $b_w / t_w$ of the web should satisfy the following expression

$$\frac{b_w}{t_w} \leq \frac{k\,E}{f_{of}} \sqrt{\frac{A_w}{A_{fc}}} \tag{6.150}$$

where:

$A_w$ is the cross section area of the web

$A_{fc}$ is the cross section area of the compression flange

The value of the factor $k$ should be taken as follows:

- plastic rotation utilized $k = 0{,}3$
- plastic moment resistance utilized $k = 0{,}4$
- plastic moment resistance utilized $k = 0{,}55$

(2) If the girder is curved in elevation, with the compression flange on the concave face, the ratio $b_w / t_w$ for the web should satisfy the following criterion:

$$\frac{b_w}{t_w} \leq \frac{kE}{f_{of}} \sqrt{\frac{A_w}{A_{fc}}} \frac{1}{\sqrt{1+\dfrac{b_w\,E}{3\,r\,f_{of}}}} \tag{6.151}$$

in which $r$ is the radius of curvature of the compression flange.

(3) If the girder is provided with transverse web stiffeners, the limiting value of $b_w / t_w$ may be increased by the factor $1+(b_w / a)^2$.

### 6.7.8 Web stiffeners

#### 6.7.8.1 Rigid end post

(1) The rigid end post (see Figure 6.27) should act as a bearing stiffener resisting the reaction from bearings at the girder support, and as a short beam resisting the longitudinal membrane stresses in the plane of the web.

(2) A rigid end post may comprise of one stiffener at the girder end and one double-sided transverse stiffener that together form the flanges of a short beam of length $h_f$, see Figure 6.27(b). The strip of web plate between the stiffeners forms the web of the short beam. Alternatively, an end post may be in the form of an inserted section, connected to the end of the web plate.

(4) The double-sided transverse stiffener may act as a bearing stiffener resisting the reaction at the girder support (see 6.2.11).

(5) The stiffener at the girder end should have a cross-sectional area of at least $4 h_f t_w^2 / e$ where $e$ is the centre to centre distance between the stiffeners and $e > 0{,}1 h_f$, see Figure 6.27(b).

(6) If an end post is the only means of providing resistance against twist at the end of a girder, the second moment of area of the end-post section about the centre-line of the web $(I_{ep})$ should satisfy:

$$I_{ep} \geq b_w^3 t_f R_{Ed} /(250 W_{Ed}) \tag{6.152}$$

where:

$t_f$ is the maximum value of flange thickness along the girder

$R_{Ed}$ is the reaction at the end of the girder under design loading

$W_{Ed}$ is the total design loading on the adjacent span.

#### 6.7.8.2 Non-rigid end post and bolted connection

(1) A non-rigid end post may be a single double-sided stiffener as shown in Figure 6.27(c). It may act as a bearing stiffener resisting the reaction at the girder support (see 6.2.11).

(2) The shear force resistance for a bolted connection as shown in Figure 6.27(c) may be assumed to be the same as for a girder with a non-rigid end post provided that the distance between bolts is $p < 40 t_w$.

#### 6.7.8.3 Intermediate transverse stiffeners

(1) Intermediate stiffeners that act as rigid supports of interior panels of the web should be checked for strength and stiffness.

(2) Other intermediate transverse stiffeners may be considered flexible, their stiffness being considered in the calculation of $k_\tau$ in 6.7.4.2.

(3) Intermediate transverse stiffeners acting as rigid supports for web panels should have a minimum second moment of area $I_{st}$:

$$\text{if } a / h_w < \sqrt{2}: \quad I_{st} \geq 1{,}5 h_w^3 t_w^3 / a^2 \tag{6.153}$$

$$\text{if } a / h_w \geq \sqrt{2}: \quad I_{st} \geq 0{,}75 h_w t_w^3 \tag{6.154}$$

The strength of intermediate rigid stiffeners should be checked for an axial force equal to $V_{Ed} - \rho_v b_w t_w f_v / \gamma_{M1}$ where $\rho_v$ is calculated for the web panel between adjacent transverse stiffeners assuming the stiffener under consideration removed. In the case of variable shear forces the check is performed for the shear force at distance $0{,}5 h_w$ from the edge of the panel with the largest shear force.

#### 6.7.8.4 Longitudinal stiffeners

(1) Longitudinal stiffeners may be either rigid or flexible. In both cases their stiffness should be taken into account when determining the relative slenderness $\lambda_w$ in 6.7.4.2(5).

(2) If the value of $\lambda_w$ is governed by the sub-panel then the stiffener may be considered as rigid.

(3) The strength should be checked for direct stresses if the stiffeners are taken into account for resisting direct stress.

#### 6.7.8.5 Welds

(1) The web to flange welds may be designed for the nominal shear flow $V_{Ed} / h_w$ if $V_{Ed}$ does not exceed $\rho_v h_w t_w f_o /(\sqrt{3}\, \gamma_{M1})$. For larger values the weld between flanges and webs should be designed for the shear flow $\eta t_w f_o /(\sqrt{3}\, \gamma_{M1})$ unless the state of stress is investigated in detail.

## 6.8 Members with corrugated webs

(1) For plate girders with trapezoidal corrugated webs, see Figure 6.33, the bending moment resistance is given in 6.8.1 and the shear force resistance in 6.8.2.

NOTE 1 Cut outs are not included in the rules for corrugated webs.

NOTE 2 For transverse loads the rules in 6.7.7 can be used as a conservative estimate.

### 6.8.1 Bending moment resistance

(1) The bending moment resistance may be derived from:

$$M_{\mathrm{Rd}} = \min \begin{Bmatrix} b_2 t_2 h_{\mathrm{f}} f_{\mathrm{o,r}} / \gamma_{\mathrm{M1}} \\ b_1 t_1 h_{\mathrm{f}} f_{\mathrm{o,r}} / \gamma_{\mathrm{M1}} \\ b_1 t_1 h_{\mathrm{f}} \chi_{\mathrm{LT}} f_{\mathrm{o,r}} / \gamma_{\mathrm{M1}} \end{Bmatrix} \begin{matrix} \text{tension flange} \\ \text{compression flange} \\ \text{compression flange} \end{matrix} \qquad (6.155)$$

where $f_{\mathrm{o,r}} = \rho_{\mathrm{z}} f_{\mathrm{o}}$ includes the reduction due to transverse moments in the flanges

$$\rho_{\mathrm{z}} = 1 - 0{,}4 \sqrt{\frac{\sigma_{\mathrm{x}}(M_{\mathrm{z}})}{f_{\mathrm{o}} / \gamma_{\mathrm{M1}}}} \qquad (6.156)$$

$M_{\mathrm{z}}$ is the transverse bending moment in the flange

$\chi_{\mathrm{LT}}$ is the reduction factor for lateral torsional buckling according to 6.3.2.

NOTE The transverse moment $M_{\mathrm{z}}$ may result from the shear flow introduction in the flanges as indicated in Figure 6.33(d).

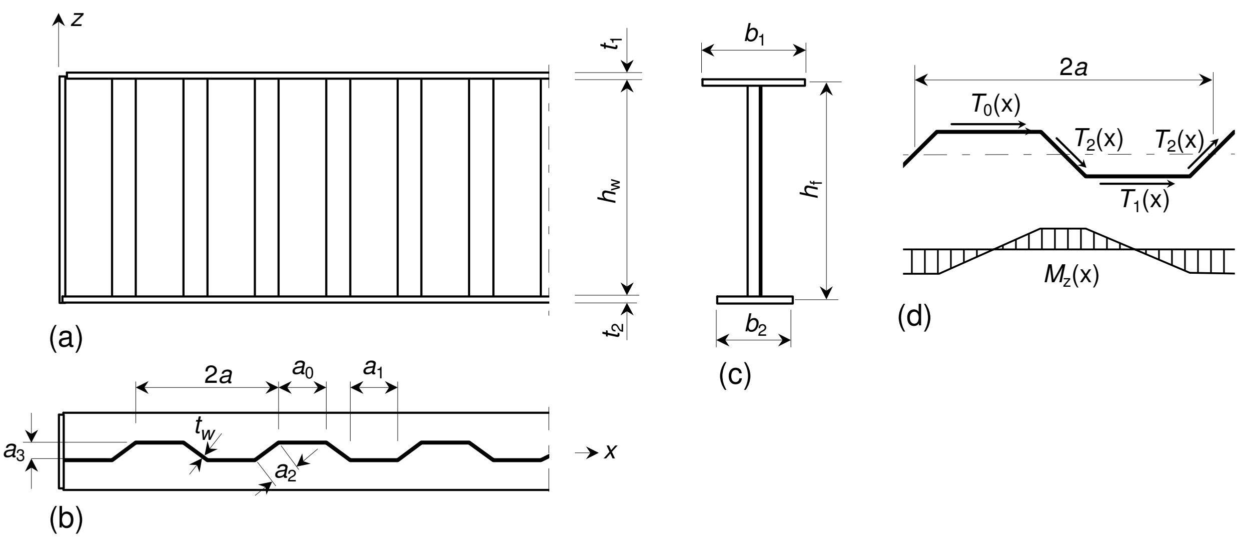



**Figure 6.33 - Corrugated web**

### 6.8.2 Shear force resistance

(1) The shear force resistance $V_{\mathrm{Rd}}$ may be taken as

$$V_{\mathrm{Rd}} = \rho_{\mathrm{c}} t_{\mathrm{w}} h_{\mathrm{w}} \frac{f_{\mathrm{o}}}{\sqrt{3} \cdot \gamma_{\mathrm{M1}}} \qquad (6.157)$$

where $\rho_{\mathrm{c}}$ is the smallest of the reduction factors for local buckling $\rho_{\mathrm{c,l}}$, reduction factor for global buckling $\rho_{\mathrm{c,g}}$ and HAZ softening factor $\rho_{\mathrm{o,haz}}$:

(2) The reduction factor $\rho_{c,l}$ for local buckling may be calculated from:

$$\rho_{c,l} = \frac{1{,}15}{0{,}9 + \lambda_{c,l}} \leq 1{,}0 \qquad (6.158)$$

where the relative slenderness $\lambda_{c,l}$ for trapezoidal corrugated webs may be taken as

$$\lambda_{c,l} = 0{,}35 \frac{a_{max}}{t_w} \sqrt{\frac{f_o}{E}} \qquad (6.159)$$

with $a_{max}$ as the greatest width of the corrugated web plate panels, $a_0, a_1$ or $a_2$, see Figure 6.33.

(3) The reduction factor $\rho_{c,g}$ for global buckling should be taken as

$$\rho_{c,g} = \frac{1{,}5}{0{,}5 + \lambda_{c,g}^2} \leq 1{,}0 \qquad (6.160)$$

where the relative slenderness $\lambda_{c,g}$ may be taken as

$$\lambda_{c,g} = \sqrt{\frac{f_o}{\sqrt{3}\,\tau_{cr,g}}} \qquad (6.161)$$

where the value $\tau_{cr,g}$ may be taken from:

$$\tau_{cr,g} = \frac{32{,}4}{t_w h_w^2} \sqrt[4]{B_x B_z^3} \qquad (6.162)$$

where:

$$B_x = \frac{2a}{a_0 + a_1 + 2a_2} \frac{E t_w^3}{10{,}9}$$

$$B_z = \frac{E I_x}{2a}$$

$2a$ is length of corrugation, see Figure 6.33
$a_0, a_1$ and $a_2$ are widths of folded web panels, see Figure 6.33
$I_x$ is second moment of area of one corrugation of length $2a$, see Figure 6.33.

NOTE Equation (6.162) applies to plates with hinged edges.

(4) The reduction factor $\rho_{o,haz}$ in HAZ is given in 6.1.6.

# 7 Serviceability Limit States

## 7.1 General

(1)P A aluminium structure shall be designed and constructed such that all relevant serviceability criteria are satisfied.

(2) The basic requirements for serviceability limit states are given in 3.4 of EN 1990.

(3) Any serviceability limit state and the associated loading and analysis model should be specified for a project.

(4) Where plastic global analysis is used for the ultimate limit state, plastic redistribution of forces and moments at the serviceability limit state may occur. If so, the effects should be considered.

NOTE The National Annex may give further guidance.

## 7.2 Serviceability limit states for buildings

### 7.2.1 Vertical deflections

(1) With reference to EN 1990 – Annex A1.4 limits for vertical deflections according to Figure A1.1 in EN 1990 should be specified for each project and agreed with the owner of the construction work.

NOTE The National Annex may specify the limits.

### 7.2.2 Horizontal deflections

(1) With reference to EN 1990 – Annex A1.4 limits for horizontal deflections according to Figure A1.2 in EN 1990 should be specified for each project and agreed with the owner of the construction work.

NOTE The National Annex may specify the limits.

### 7.2.3 Dynamic effects

(1) With reference to EN 1990 – Annex A1.4.4 the vibrations of structures on which the public can walk should be limited to avoid significant discomfort to users, and limits should be specified for each project and agreed with the owner of the construction work.

NOTE The National Annex may specify limits for vibration of floors.

### 7.2.4 Calculation of elastic deflection

(1) The calculation of elastic deflection should generally be based on the properties of the gross cross-section of the member. However, for slender sections it may be necessary to take reduced section properties to allow for local buckling (see section 6.7.5). Due allowance of effects of partitioning and other stiffening effects, second order effects and changes in geometry should also be made.

(2) For class 4 sections the following effective second moment of area $I_{ser}$, constant along the beam may be used

$$I_{ser} = I_{gr} - \frac{\sigma_{gr}}{f_o}(I_{gr} - I_{eff}) \tag{7.1}$$

where:

$I_{gr}$ is the second moment of area of the gross cross-section

$I_{eff}$ is the second moment of area of the effective cross-section at the ultimate limit state, with allowance for local buckling, see 6.7.5

$\sigma_{gr}$ is the maximum compressive bending stress at the serviceability limit state, based on the gross cross-section (positive in the formula).

(3) Deflections should be calculated making also due allowance for the rotational stiffness of any semi-rigid joints, and the possible recurrence of local plastic deformation at the serviceability limit state.

# 8 Design of joints

## 8.1 Basis of design

### 8.1.1 Introduction

(1)P All joints shall have a design resistance such that the structure remains effective and is capable of satisfying all the basic design requirements given in 2.

(2) The partial safety factors $\gamma_M$ for joints should be applied to the characteristic resistance for the various types of joints.

NOTE Numerical values for $\gamma_M$ may be defined in the National Annex. Recommended values are given in Table 8.1

**Table 8.1 - Recommended partial factors $\gamma_M$ for joints**

| | |
|---|---|
| Resistance of members and cross-sections | $\gamma_{M1}$ and $\gamma_{M2}$ see 6.1.3 |
| Resistance of bolt connections | $\gamma_{M2} = 1{,}25$ |
| Resistance of rivet connections | |
| Resistance of plates in bearing | |
| Resistance of pin connections | $\gamma_{Mp} = 1{,}25$ |
| Resistance of welded connections | $\gamma_{Mw} = 1{,}25$ |
| Slip resistance | |
| - for hybrid connections or connections under fatigue loading | $\gamma_{M3} = 1{,}1$, see 8.5.9.3 |
| - for other design situations | $\gamma_{M3} = 1{,}1$, see 8.5.9.3 |
| - for ultimate limit states | $\gamma_{M3} = 1{,}25$ |
| Resistance of adhesive bonded connections | $\gamma_{Ma} \geq 3{,}0$ |
| Bearing resistance of an injection bolt | $\gamma_{M4} = 1{,}0$ |
| Resistance of joints in hollow section lattice girder | $\gamma_{M5} = 1{,}0$ |
| Resistance of pins at serviceability limit state | $\gamma_{M6,ser} = 1{,}0$ |
| Preload of high strength bolts | $\gamma_{M7} = 1{,}1$ |

(3) Joints subject to fatigue should also satisfy the rules given in EN 1999-1-3.

### 8.1.2 Applied forces and moments

(1) The forces and moments applied to joints at the ultimate limit state should be determined by global analysis conforming to 5.

(2) These applied forces and moments should include:

- second order effects;
- the effects of imperfections (see 5.3);
- the effects of connection flexibility

NOTE For the effect of connection flexibility, see Annex L.

### 8.1.3 Resistance of joints

(1) The resistance of a joint should be determined on the basis of the resistances of the individual fasteners, welds and other components of the joint.

(2) Linear-elastic analysis should generally be used in the design of the joint. Alternatively non-linear analysis of the joint may be employed provided that it takes account of the load deformation characteristics of all the components of the joint.

(3) If the design model is based on yield lines such as block shear i.e., the adequacy of the model should be demonstrated on the basis of physical tests.

### 8.1.4 Design assumptions

(1) Joints may be designed by distributing the internal forces and moments in whatever rational way is best, provided that:

(a) the assumed internal forces and moments are in equilibrium with the applied forces and moments;

(b) each parts in the joint is capable of resisting the forces or stresses assumed in the analysis;

(c) the deformations implied by this distribution are within the deformation capacity of the fasteners or welds and of the connected parts, and

(d) the deformations assumed in any design model based on yield lines are based on rigid body rotations (and in-plane deformations) which are physically possible.

(2) In addition, the assumed distribution of internal forces should be realistic with regard to relative stiffness within the joint. The internal forces will seek to follow the path with the greatest rigidity. This path should be clearly identified and consistently followed throughout the design of the joint.

(3) Residual stresses and stresses due to tightening of fasteners and due to ordinary accuracy of fit-up need not usually be allowed for.

### 8.1.5 Fabrication and execution

(1) Ease of fabrication and execution should be considered in the design of all joints and splices.

(2) Attention should be paid to:

- the clearances necessary for safe execution;
- the clearances needed for tightening fasteners;
- the need for access for welding;
- the requirements of welding procedures, and
- the effects of angular and length tolerances on fit-up.

(3) Attention should also be paid to the requirements for:

- subsequent inspection;
- surface treatment, and
- maintenance.

Requirements to execution of aluminium structures are given in prEN 1090-3.

## 8.2 Intersections for bolted, riveted and welded joints

(1) Members meeting at a joint should usually be arranged with their centroidal axes intersecting at a point.

(2) Any kind of eccentricity in the nodes should be taken into account, except in the case of particular types of structures where it has been demonstrated that it is not necessary.