5.4 Beams

As stated previously, the effect of local buckling should invariably be taken into account in thin walled members, using methods described already. Laterally stable beams are beams, which do not buckle laterally. Designs may be carried out using simple beam theory, making suitable modifications to take account of local buckling of the webs. This is done by imposing a maximum compressive stress, which may be considered to act on the bending element. The maximum value of the stress is given by

$$p_0 = \left[1.13 - 0.0019\frac{D}{t}\sqrt{\frac{f_y}{280}}\right] p_y \ngtr f_y \qquad \textbf{\textit{(5.11)}}$$

Where p_o = the limiting value of compressive stress in N/mm^2

D/t = web depth/thickness ratio

f_y = material yield stress in N/mm^2.

p_y = design strength in N/mm^2

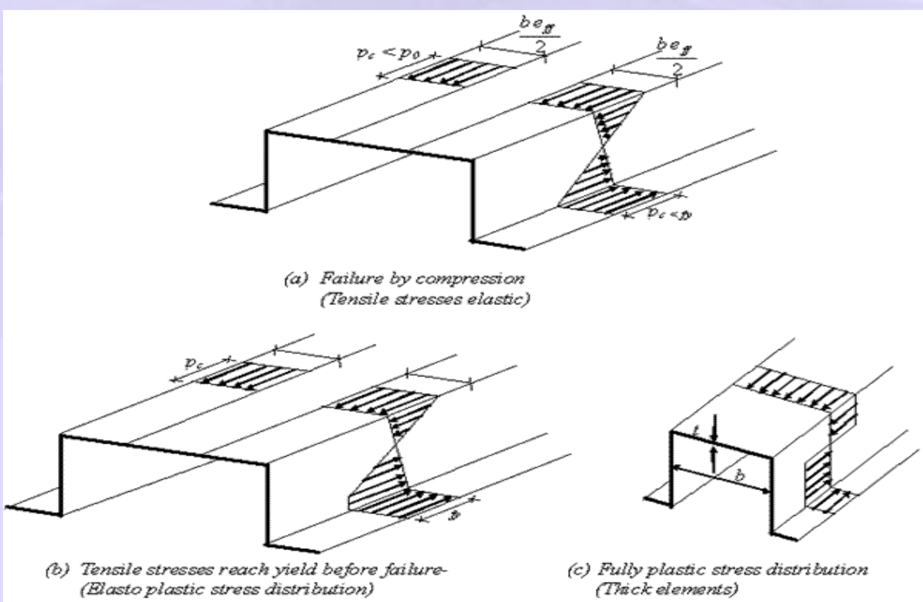

Fig.5.11 Laterally stable beams: Possible stress patterns

For steel with f_y = 280 N/mm^2, $p_0 = f_y$ when (D/t) ? 68.

For greater web slenderness values, local web buckling has a detrimental effect. The moment capacity of the cross section is determined by limiting the maximum stress on the web to p_0. The effective width of the compression element is evaluated using this stress and the effective section properties are evaluated. The ultimate moment capacity (M_{ult}) is given by

$$M_{ult} = Z_c.p_0 \qquad \textit{(5.11a)}$$

Where Z_c = effective compression section modulus

This is subject to the condition that the maximum tensile stress in the section does not exceed f_y (see Fig.5.11a).

If the neutral axis is such that the tensile stresses reach yield first, then the moment capacity is to be evaluated on the basis of elasto-plastic stress distribution (see Fig.5.11b). In elements having low (width/thickness) ratios, compressive stress at collapse can equal yield stress (see Fig. 5.11c). In order to ensure yielding before local buckling, the maximum (width/thickness) ratio of stiffened elements is $\leq 25\sqrt{\frac{280}{f_y}}$ and for unstiffened elements, it is $\leq 8\sqrt{\frac{280}{f_y}}$

5.4.1 Other beam failure criteria

5.4.1.1 Web crushing

This may occur under concentrated loads or at support point when deep slender webs are employed. A widely used method of overcoming web crushing problems is to use web cleats at support points (See Fig.5.12).

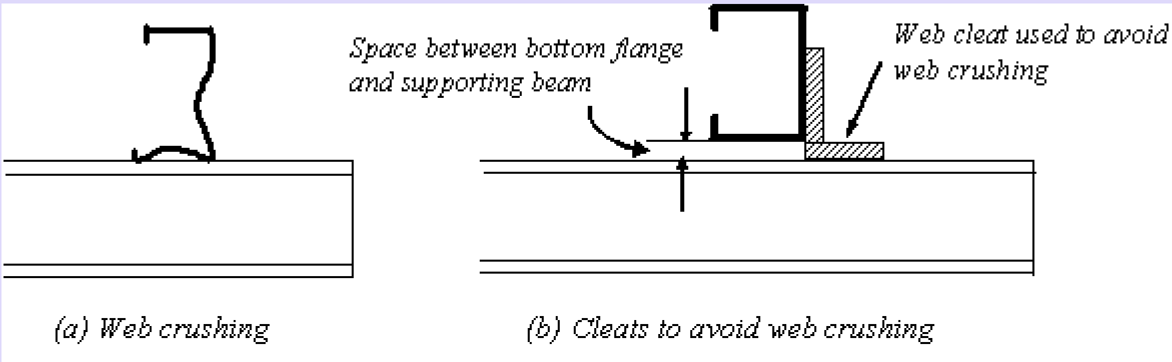

Fig.5.12 Web crushing and how to avoid it

5.4.1.2 Shear buckling

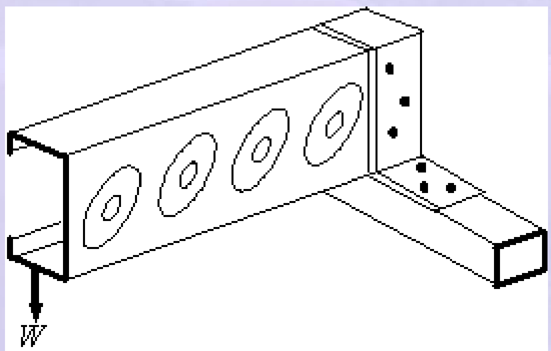

Fig. 5.13 Web buckling

The phenomenon of shear buckling of thin webs has been discussed in detail in the chapter on "Plate Girders". Thin webs subjected to predominant shear will buckle as shown in Fig.5.13. The maximum shear in a beam web is invariably limited to 0.7 times yield stress in shear. In addition in deep webs, where shear buckling can occur, the average shear stress (p_v) must be less than the value calculated as follows:

$$p_v \le \left(\frac{1000\,t}{D}\right)^2 \qquad \textbf{\textit{(5.12)}}$$

Where p = average shear stress in N/mm^2.

t and D are the web thickness and depth respectively (in mm)

5.4.2 Lateral buckling

The great majority of cold formed beams are (by design) restrained against lateral deflections. This is achieved by connecting them to adjacent elements, roof sheeting or to bracing members. However, there are circumstances where this is not the case and the possibility of lateral buckling has to be considered.

Lateral buckling will not occur if the beam under loading bends only about the minor axis. If the beam is provided with lateral restraints, capable of resisting a lateral force of 3% of the maximum force in the compression flange, the beam may be regarded as restrained and no lateral buckling will occur.

As described in the chapter on "Unrestrained Beams"', lateral buckling occurs only in "long" beams and is characterised by the beam moving laterally and twisting when a transverse load is applied. This type of buckling is of importance for long beams with low lateral stiffness and low torsional stiffness (See Fig.5.14); such beams under loading will bend about the major axis.

The design approach is based on the "effective length" of the beam for lateral buckling, which is dependent on support and loading conditions. The effective length of beams with both ends supported and having restraints against twisting is taken as 0.9 times the length, provided the load is applied at bottom flange level. If a load is applied to the top flange which is unrestrained laterally, the effective length is increased by 20%. This is considered to be a "destabilising load", i.e. a load that encourages lateral instability.

The elastic lateral buckling moment capacity is determined next. For an I section or symmetrical channel section bent in the plane of the web and loaded through shear centre, this is

$$M_E = \frac{\pi^2 A.E.D}{2\left(l_e / r_y\right)^2}.C_b\sqrt{1+\frac{1}{20}\left(\frac{l_e}{r_y}.\frac{t}{D}\right)^2} \qquad \textbf{(5.13)}$$

Where,

A = cross sectional area, in mm^2

D = web depth, in mm

t = web thickness, in mm

r_y = radius of gyration for the lateral bending of section

C_b = 1.75 - 1.05 β + 0.3 β^2 x 2.3.

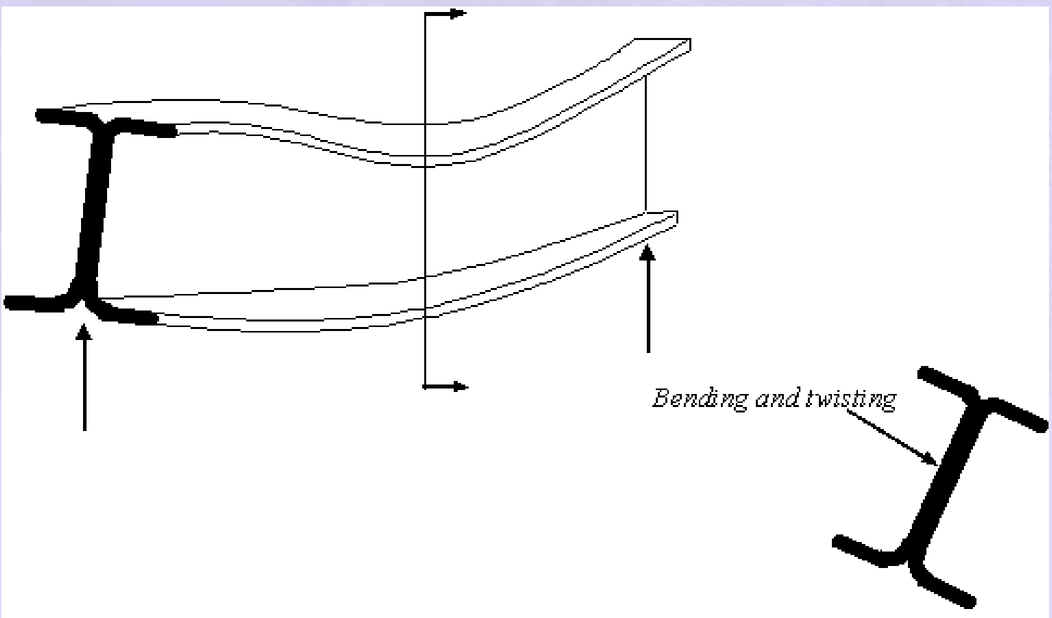

Fig.5.14 Lateral buckling

Where β = ratio of the smaller end moment to the larger end moment M in an unbraced length of beam. β is taken positive for single curvature bending and negative for double curvature (see Fig. 5.15)

To provide for the effects of imperfections, the bending capacity in the plane of loading and other effects, the value of M_E obtained from eq. (5.13) will need to be modified. The basic concept used is explained in the chapter on Column Buckling where the failure load of a column is obtained by employing the Perry-Robertson equation for evaluating the collapse load of a column from a knowledge of the yield load and Euler buckling load.

M_E = Elastic lateral buckling resistance moment given by equation (5.13)

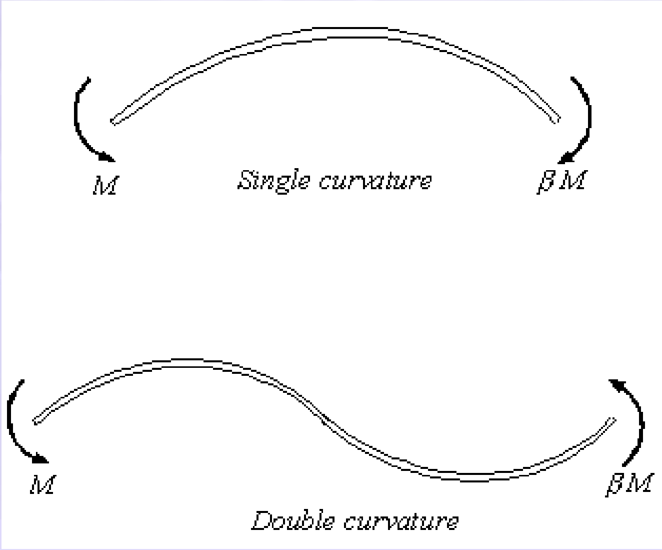

Fig. 5.15 Single and double curvature bending

A similar Perry-Robertson type equation is employed for evaluating the Moment Resistance of the beam

$$M_b = \frac{1}{2}\left[\left\{M_y + (1+\eta)M_E\right\} - \sqrt{\left[M_y + (1+\eta)M_E\right]^2 - 4M_y.M_E}\right] \qquad (5.14)$$

M_y = First yield moment given by the product of yield stress (f_y) and the Elastic Modulus (Z_c) of the gross section.

η = Perry coefficient, given by

When $\frac{l_e}{r_y} < 40C_b$, $\eta = 0$.

When $\frac{l_e}{r_y} < 40C_b$, $\eta = 0.002\left(\frac{l_e}{r_y} - 40C_b\right)$

l_e = effective length

r_y = radius of gyration of the section about the y - axis.

When the calculated value of M_b exceed M_{ult} calculated by using equation (5.11.a), then M_b is limited to M_{ult}. This will happen when the beams are "short".