Member buckling with tension flange restraint

David Brown of the SCI explains the structural mechanics and design provisions for tension flange restraint for both flexural and lateral torsional buckling.

Introduction

Portal frame designers are familiar with tension flange restraint, often referred to using the shorthand of "Annex G", which is the relevant Annex in BS 5950. The benefit of tension flange restraint is used to increase the lateral torsional buckling resistance. Restraint to one flange can also be used to increase resistance to flexural buckling under compression.

Lateral torsional buckling

A simply supported, unrestrained beam is shown in Figure 1. The unrestrained compression flange buckles laterally, dragging the tension flange with it. The tension flange is reluctant to be displaced at all, resulting in a lateral movement and a rotation of the section. Note that the centre of the rotation is some way distant from the section.

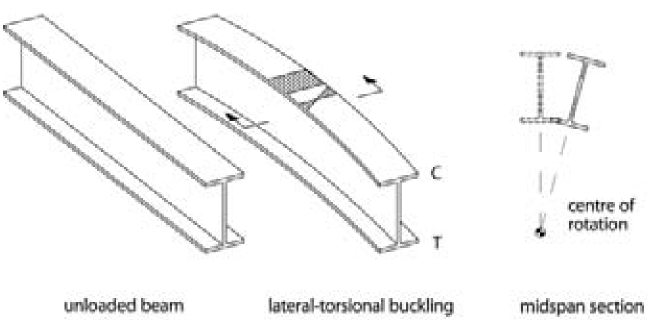

Figure 1 Lateral torsional buckling

The compression flange experiences a direct stress due to the major axis moment $\left[\frac{\text{moment}}{\text{modulus}}\right]$. Because the flange is now also curved on plan, there is a bending stress in the flange, in the section's minor axis. At the extreme fibres, the two stresses combine and collapse results if the combined stresses reach yield. This is obviously a highly simplistic explanation, as plasticity, residual stresses and second-order effects also contribute to the buckling behaviour, but the simple explanation is useful when developing the mechanics of tension flange restraint.

If restraint is provided at, or close to the tension flange, the lateral shift of the compression flange is reduced, as shown in Figure 2. The point of rotation is now forced to be much closer to the tension flange (usually the point of rotation will be some distance off the tension flange, such as the centreline of the restraint). Because the lateral shift is constrained, the additional bending stress is reduced. This translates to an increased resistance.

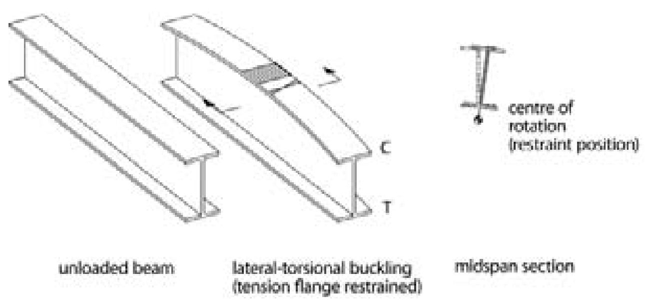

Figure 2 Lateral torsional buckling with tension flange restraint

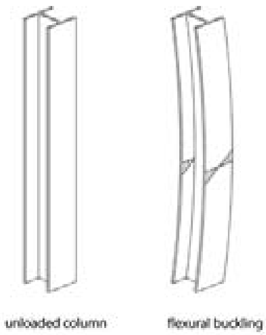

Figure 3 Flexural buckling

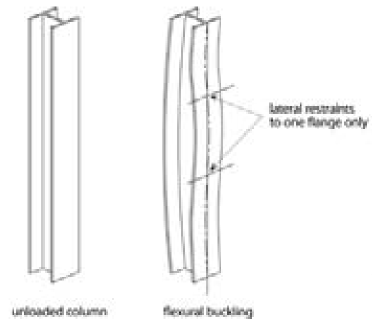

Figure 4 Minor axis flexural buckling with one flange restrained

The background theory assumes that the restraint to the tension flange is continuous, rather like a piano hinge running along the length of the member. In most cases, this sort of continuous restraint cannot be achieved in practice. Therefore design Standards have further checks, to ensure that the restraints to the tension flange are close enough to provide appropriate lateral fixity.

Flexural buckling under compression

In an unrestrained compression member, the section buckles about its minor axis, as shown in Figure 3. The cross section experiences the axial stress, but also a bending stress in the minor axis. This is again a simple explanation of a complex situation.

If intermediate restraints are introduced to one flange only, as shown in Figure 4, the unrestrained flange is free to buckle, but the extent of that displacement is reduced. The bending stress is therefore reduced, which translates to an increased resistance. Like lateral torsional buckling, the restraints to the flange must be located frequently enough to mimic the effect of a continuous "piano hinge" restraint.

What difference does it make?

The answer is always "it depends", but the benefit can be very helpful. In general, the benefit is more significant in flexural buckling compared to lateral torsional buckling. Numerical comparisons are presented in the following paragraphs. The calculations follow the Eurocode nomenclature.

Flexural Buckling Comparison

Example 1.

Consider a 610 × 229 × 113 UKB, S275 with 6.325 m between torsional restraints. The bending moment diagram is triangular, and the axial force N_{Ed} is 188 kN. There are intermediate restraints to one flange (the tension flange) at 1.6 m centres.

The verification is covered by clause BB.3.1.2(2)B. The spacing of the intermediate restraints must not be greater than L_m of BB.3.1.1

$$L_m = \frac{38 i_z}{\sqrt{\frac{1}{57.4}\left(\frac{N_{Ed}}{A}\right) + \frac{1}{756 C_1^2}\frac{W_{pl,y}^2}{A I_t}\left(\frac{F_y}{235}\right)^2}}$$

The C_1 factor relates to the shape of the bending moment diagram. For an overall triangular bending moment diagram, the lowest value (most onerous) of any of the segments in the member is $C_1 = 1.17$

Substituting the various section properties, steel strength, *etc* into the expression,

$$L_m = \frac{38 \times 48.8}{\sqrt{\frac{1}{57.4}\left(\frac{188 \times 10^3}{14400}\right) + \frac{1}{756 \times 1.17^2}\frac{(3280 \times 10^3)^2}{14400 \times 111 \times 10^4}\left(\frac{265}{235}\right)^2}}$$

$L_m = 1819$ mm

With intermediate restraints at 1600 mm, they are considered effective and the benefit of restraint to one flange may be utilised.

The elastic critical force N_{crT} for this situation, with restraints to one flange, is given in BB.3.3.1 by:

$$N_{crT} = \frac{1}{i_s^2}\sqrt{\frac{\pi^2 E I_z a^2}{L_t^2} + \frac{\pi^2 E I_w}{L_t^2} + G I_T}$$

where $i_s^2 = i_v^2 + i_z^2 + a^2$

The key variable is a, which is the distance from the shear centre of the section to the longitudinal axis of the restraints. Assuming the axis of the restraint is 100 mm off the face of the column, the distance a is 403.8 mm.

Substituting the various section properties, steel strength, *etc* into the expression,

$N_{cr} = 2365$ kN

Using curve b, this leads to a buckling resistance, $N_{b,Rd}$ of 1685 kN

Example 2

If the benefit of restraints to one flange had not been taken, the elastic critical force N_{cr} is given by:

$$N_{cr} = \frac{\pi^2 EI}{L^2} = 1770 \text{ kN}$$

Using curve b, this leads to a buckling resistance, $N_{b,Rd}$ of 1336 kN, approximately 20% less than previously calculated.

Lateral torsional buckling comparison

Example 3

Although not given in the Eurocode, the expression for the elastic critical moment of a bi-symmetric section under a uniform moment can be found in Reference **1** and is given by

$$M_{cr0} = \frac{i_s^2}{2a} N_{cr}$$

Using the same details as example 1, $M_{cr0} = \dfrac{225952}{2 \times 403.8} \times 2365 = 662$ kNm

The influence of the shape of the bending moment diagram is incorporated using the C_m factor in BB.3.3.1. Following this procedure gives $C_m = 1.883$.

The elastic critical moment under a linear moment is then given by

$$M_{cr} = M_{cr0} \times C_m = 662 \times 1.883 = 1246 \text{ kNm}$$

Using curve c and expression 6.57, the calculated buckling resistance $M_{b,Rd}$ = 644 kNm

Example 3

If the benefits of tension flange restraint were ignored, the elastic critical moment of a bi-symmetric section is given by

$$M_{cr} = C_1 \frac{\pi^2 EI_z}{L^2} \sqrt{\frac{I_w}{I_z} + \frac{L^2 GI_t}{\pi^2 EI_z}}$$

where C_1 accounts for the shape of the bending moment diagram.

Substituting the various section properties, steel strength, *etc* into the expression, $M_{cr} = 1167$ kNm and $M_{b,Rd} = 629$ kNm. (This calculation has ignored the benefit of the ***f*** factor given in 6.3.2.3(2))

Summary

	Flexural resistance $N_{b,Rd}$	Lateral torsional resistance $M_{b,Rd}$
With tension flange restraint	1685 kN	644 kNm
Without tension flange restraint	1336 kN	629 kNm

Conclusions

Restraint to one flange can be of benefit, especially for flexural buckling. The improvement in lateral torsional buckling resistance is perhaps not as much as commonly thought. In both cases, it is essential that the spacing of the intermediate restraints to the tension flange meet the requirements of the design Standard, to ensure they provide appropriate restraint.

1. **KING, C. M.** *Design of Steel Portal Frames for Europe*, P164, SCI, 2001