Design of Cruciform Sections using BS 5950-1:2000

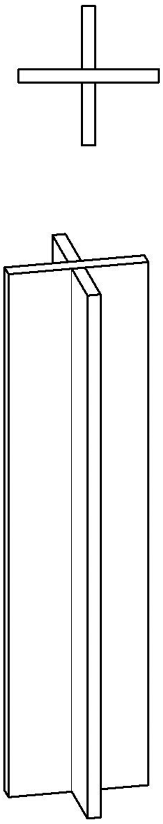

Fig1. Plain Cruciform

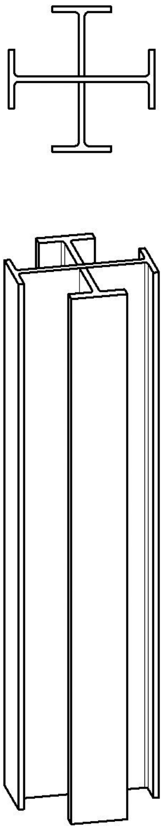

Fig 2. Flanged Cruciform

Many engineers will be relatively unfamiliar with torsional modes of buckling, but they are explicitly covered in the Eurocodes. Steel Construction Institute Senior Manager Charles King explains what designers of bi-symmetric cruciform sections need to know about buckling.

1. CRUCIFORM SECTIONS AND TORSIONAL BUCKLING

1.1 Who's heard of torsional modes of buckling?

There are several modes of buckling that affect structures. Most structural engineers are familiar with:

1. Flexural buckling, commonly referred to as strut buckling, column buckling, compression buckling or Euler buckling to distinguish it from the buckling of a beam due to a bending moment.
2. Lateral torsional buckling, commonly referred to as beam buckling
3. Plate buckling, which one might expect in slender components of box girders
4. Shear buckling, which is a type of plate buckling
5. Local buckling, which is a common term for plate buckling modes that might occur in members.

Many structural engineers working with hot-rolled members or fabricated plate members are less familiar with, or are even unaware of, torsional modes of buckling. Design codes for orthodox structures commonly do not explicitly mention these modes. However, codes do cover such modes; for example the checks in Annex G of BS 5950 1:2000 are for buckling modes that are close relatives of the torsional and torsional-flexural buckling modes described in this article. Torsional modes of buckling are explicitly covered in the Eurocodes.

Torsional buckling modes are better known to engineers designing cold-formed members, where these modes are often critical, and are addressed by design codes.

This article is written to assist in the design of bi-symmetric cruciform sections, as shown in Figure 1 and Figure 2. Such sections are prone to torsional buckling, but not to torsional-flexural buckling because the centroid of the section coincides with the shear centre.

1.2 Does it matter?

A good question is, "If I've never heard of these modes, can they be important?" The answer is that these buckling modes are not the critical modes in the most commonly used compression members with the most common restraint arrangements.

Structural hollow sections have such high torsional stiffness that torsional modes cannot be the critical modes in normal sections. Torsional modes will not be critical in bi-symmetric I-sections (eg UCs) with both flanges restrained at the same points along a member. However, where the flanges are not equally restrained, torsional modes may be the critical modes.

For all other sections, torsional modes are likely to be the critical modes. In the case of angles, channels and tees, this is effectively allowed for in BS 5950-1 by the slenderness calculations to 4.7.10 and Table 25. For other mono-symmetric or asymmetric sections, most codes give no assistance. With the evolution of steel structures towards more exotic structural forms, designers need to be aware of these other modes. It is especially worth noting that in an I-section with different flanges, torsional-flexural buckling is commonly the lowest mode.

1.3 What do these modes look like?

Torsional modes of buckling are commonly divided into two categories:

1. Torsional buckling
2. Torsional-flexural buckling

Cruciform sections and similar sections all have a torsional buckling mode, as shown in Figures 3 and 4.

Asymmetric sections are all prone to torsional-flexural buckling, as shown in Figure 5. Indeed, it is common for torsional-flexural buckling to be the critical mode of buckling, more onerous than pure minor axis buckling (Figure 5)

Sections in which the shear centre does NOT coincide with the centroid will be prone to torsional-flexural buckling and this should be expected to be

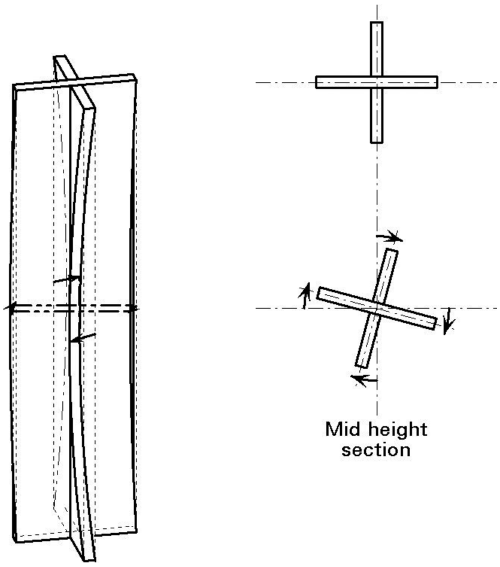

Fig 3. Plain Cruciform deforming in the torsional mode

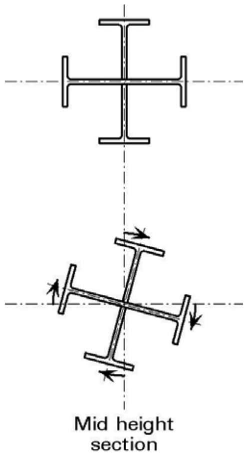

Fig 4. Asymmetric section

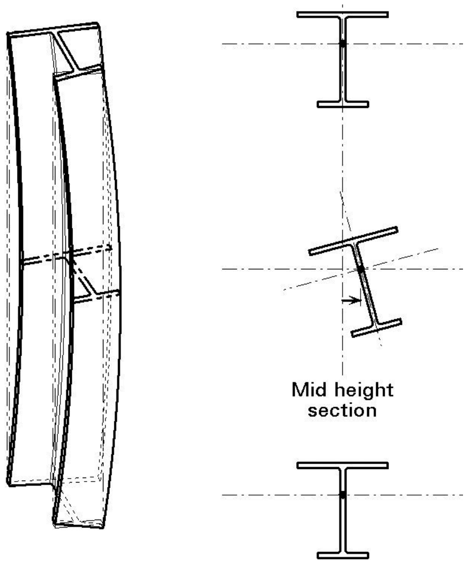

Fig 5. Plain cruciform

the critical mode of buckling for a given effective length. Sections in which the shear centre DOES coincide with the centroid will NOT be prone to torsional-flexural buckling.

This article is intended for the design of bi-symmetric cruciform sections. In these sections, the shear centre coincides with the centroid so they are NOT prone to torsional-flexural buckling.

1.4 What should a designer do?

Designers would be wise to avoid the problem wherever possible by using appropriate sections and restraint arrangements. It is simplest to resist compression either by UCs with both flanges equally restrained or by hollow sections.

If you cannot avoid the problem, calculate the compressive strengths for the torsional modes and compare these with the compressive strength for flexural buckling. Then calculate the compression resistance of the member using the lowest compressive strength and continue with the normal codified design procedures.

2. COMPRESSIVE STRENGTH

2.1 Use of the elastic critical buckling strength

The "elastic critical buckling stress" is the stress at which the member would buckle if it had an infinitely high yield stress and no imperfections. Engineers learn about Euler buckling during their studies and learn that the Euler buckling load of a strut is

$$P_E = \frac{\pi^2 EI}{L^2}$$

The Euler buckling stress is the elastic critical buckling stress of the strut for flexural buckling. In the case of Euler buckling, this can be written using BS 5950 1 symbols as

$$p_E = \frac{P_E}{A} = \frac{\left(\frac{\pi^2 EI}{L^2}\right)}{A} = \frac{\pi^2 E\left(\frac{I}{A}\right)}{L^2} = \frac{\pi^2 E\left(r^2\right)}{L^2} = \frac{\pi^2 E}{\left(L/r\right)^2}$$

Codes use the elastic critical buckling stress to find the compressive strength of a member. In BS 5950-1:2000, this can be seen in Annex C, section C.1. The procedure in Annex C can be adapted to check other modes of buckling by using the elastic critical stress of the mode of buckling considered in place of the elastic critical stress from the usual flexural mode.

To avoid having to use Annex C to calculate p_c from p_E, it is more convenient to calculate the equivalent slenderness, λ, from the equation in Annex C and then find p_c directly from tables in the Standard.

Annex C gives $p_E = \pi^2 E/\lambda^2$. Therefore, λ can be found from $\lambda = \sqrt{\frac{\pi^2 E}{p_E}}$.

This allows the designer to use Table 24 of BS 5950-1:2000 to find p_c easily.

2.2 Torsional buckling

The elastic critical load for torsional buckling can be found in classic references such as Timoshenko, Strength of Materials, Part 2 or Timoshenko and Gere, Theory of Elastic Stability. The elastic critical buckling stress is simply [elastic critical load]/area. Using symbols in BS 5950-1,

$$p_E = \frac{1}{I_0}\left(GJ + \frac{n^2\pi^2}{L^2}EH\right)$$

where

I_0 is the polar moment of area with respect to the shear centre of the section. For bi-symmetric cruciform sections, $I_0 = I_x + I_y$.

G is the torsional modulus

$$= \frac{E}{2(1+v)} = \frac{E}{2(1+0.3)} = \frac{E}{2.6}$$

J is the torsional co $J = \frac{\Sigma bt^3}{3}$.

For flanged cruciforms comprising two rolled sections, J is the sum of the J values of the component rolled sections.

n is the number of half-sine waves along the outstands of the member. For members restrained at both ends, this should be taken as 1.0 unless you are sure you can prove it is

greater. For cantilever columns this should not be taken as greater than 0.5.
L is the length of the member
E is Young's modulus
and
H is the warping constant. For bi-symmetric cruciform sections such as in Figures 1 and 2, this is the sum of the warping constants, H, for the two component sections. H for a plain cruciform is very small, so it is recommended that H is taken as zero.

The formula for members with no warping stiffness (such as a plain cruciform) reduces to:

$$p_E = \frac{1}{I_0} GJ$$

2.3 Selection of the buckling curve

Where there is a torsional component to the buckling, the buckling involves flexure of the flanges similar to minor axis buckling of the component parts of the member. Therefore the buckling curve should be selected from Table 23 of BS 5950-1:2000 for the components as shown in Figure 6.
For example, a flanged cruciform made from UB sections designed to BS 5950-1:2000 would use curves (b) or (c) depending on the thickness of the flange being less than or greater than 40mm. A plain cruciform section would also use curves (b) or (c) depending on the thickness of the flange being less than or greater than 40mm.

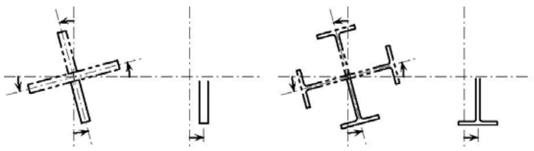

Fig 6. Similarity between torsional buckling and flexural buckling about the minor axis of the component parts

3. BENDING STRENGTH OF CRUCIFORM SECTIONS

When checking the interaction of axial load and bending moment, both axial and bending resistance are required. The axial resistance should be checked as above. The bending resistance can be calculated using the advice in the following sections. Note that the following advice is limited to bi-symmetric sections (torsional-flexural buckling is not critical). A subsequent article will show how torsional flexural buckling can be calculated for other sections

3.1 Flanged cruciform sections

The bending strength of flanged cruciform sections is most conveniently found by calculating the slenderness, λ_{LT}, using the method in BS 5950-1:2000. It is recommended that when using Annex B, section B.2.3, the value of gamma, γ, is taken as 1.0 because the value given in Annex B was derived for single I-sections. It is also recommended that values of "*u*" and "*x*" are calculated using the formula given for channels with equal flanges to avoid assumptions made for single I-sections. Tables 16 or 17 can then be used as for lateral torsional buckling of single I-sections.

3.2 Plain cruciform sections

It is recommended that the derivation of the slenderness for plain cruciform sections, as shown in Figure 1, is calculated assuming that the section is Class 3 and that designers proceed as follows:

1. Calculate the elastic critical stress for lateral torsional buckling for members with no warping stiffness (such as a plain cruciform), which is:

$$p_E = \frac{M_{cr}}{Z_x} = \frac{\left(\frac{\pi}{L}\sqrt{EI_y GJ}\right)}{Z_x}$$

2. Calculate the slenderness, λ_{LT}, from the equation in Annex B relating p_E to slenderness, λ_{LT}, which can be re-arranged as

$$\lambda_{LT} = \sqrt{\frac{\pi^2 E}{p_E}}.$$

3. Use Table 17 of BS 5950-1:2000 to find p_b.

4. DETAILING CRUCIFORM SECTIONS

Where the failure mode is by flexural buckling about one of the rectangular axes of the cruciform, the I-section that is on the axis of flexure relies on the other I-section for stability. If the web of the I-section that is on the axis of flexure is neither stiff enough nor strong enough to make the flanges on the axis of flexure deflect in the same shape as expected for the whole, then these flanges will deflect more and will unload, reducing the resistance of the cruciform.

The simple way to avoid this problem is to add gussets between the webs and flanges (so that all the elements are connected) at centres such that the slenderness about the minor axis of the I-sections (taking an effective length factor of 1.0) is not greater than the slenderness of the full length cruciform. A possible arrangement is shown in Figures 7 and 8.

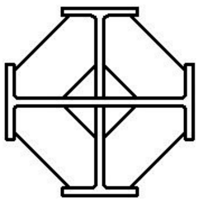

Fig 7.

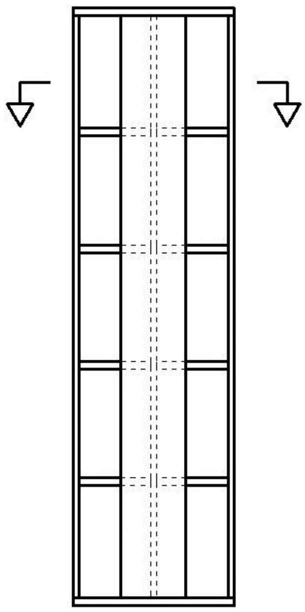

Fig 8.