Restraint to chords

BS 5950 indicates that purlins can be assumed to provide out-of-plane restraint to trusses. David Brown of the SCI discusses the intended scope of the advice and warns against straying outside the intended application.

Clause 4.10 of BS 5950 covers members in lattice frames and trusses. The clause contains a series of assumptions that designers may adopt, notably about buckling lengths, joint fixity and approximate bending moments in the rafters. The subject of this article is part (a) of that clause, which notes that the out-of-plane (buckling) lengths may be taken as the distance between purlins. It is tempting for designers to apply this guidance to all types of trusses, not appreciating that the original intent was relatively lightweight roof trusses.

In long span roofs, it is relatively common to provide a truss solution, perhaps with secondary trusses spanning onto primary trusses, so that internal column-free space is maximised. Some of the larger trusses carry significant loading and may therefore be fabricated with UC section chords (typically), or sometimes UB section chords, if other steelwork members connect to the chord. The eventual solution may be something like that shown in Figure 1. The chords are both UC members and the internal members are hollow sections. The exact details are immaterial – the key point is that there are purlins at the node points, and because of the proposed geometry and member selection, there are purlin connections at intermediate positions between the nodes.

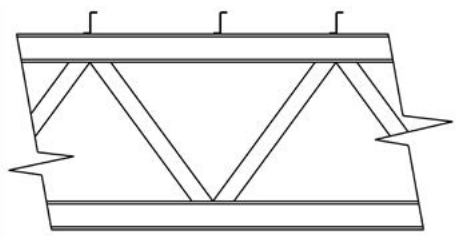

Figure 1: Assumed truss arrangement

Assuming that the top chord is in compression, the buckling resistance must be calculated, demanding an assessment of the buckling lengths in each axis. Designers may refer to clause 4.10 of BS 5950 and conclude from that clause that the out-of-plane buckling lengths may be taken as the spacing of the purlins. But is a connection to only one flange providing the assumed restraint, particularly at the intermediate location? Would the restraint be satisfactory for a UC section? Would it be equally satisfactory for a UB section, if one had been chosen?

The original intent of the clause

Colin Taylor, the primary drafter of BS 5950 has been consulted and his advice is acknowledged with gratitude. Colin comments that the clause was intended to be applied to small roof trusses (note the word "rafter" used in the clause) and similar triangulated lattices. The members themselves would have typically been angles, back-to-back angles or tees. At the time of drafting, purlins were angles, channels or even hollow sections. The use of light gauge purlins came later. Colin also notes that designers would have naturally provided restraint to the "inside" flange of compression chords.

It is interesting to look back even further, at the provisions in BS 449. Diagrams are provided giving the buckling lengths for stanchions, including those with tie beams attached to one flange only. Figure 14 from BS 449 is reproduced below as Figure 2, and the "diaphragms" shown providing restraint to the inside flange a clearly an important feature.

Figure 15 of BS 449 is equally instructive. In that Figure, a single storey stanchion has a number of intermediate angle side

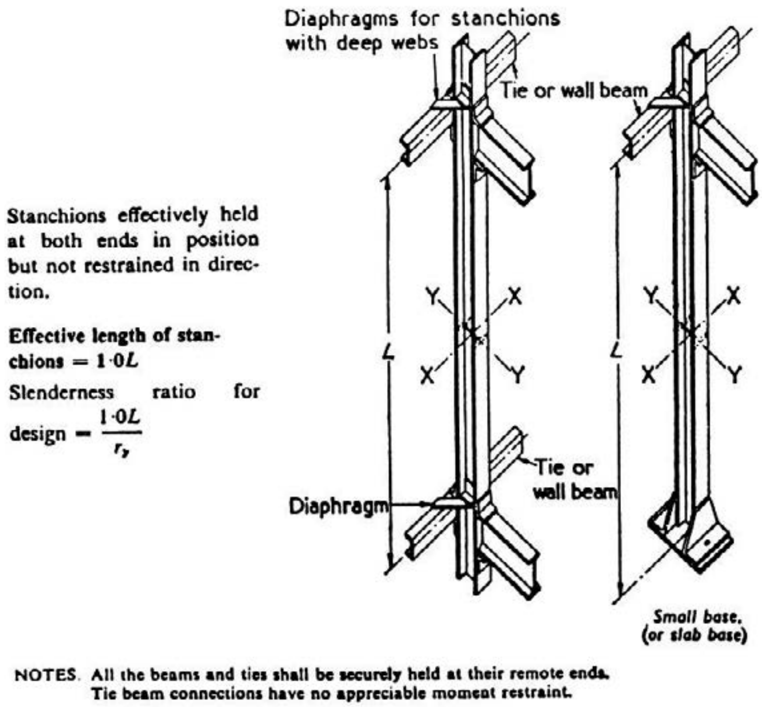

Figure 2: Figure 14 from BS 449 – Stanchion with tie beams attached to one flange

rails, attached to one flange only. The out-of-plane effective length is specified as 0.75L, where *L* is the overall height of the column, despite the intermediate angle rails.

Perhaps we might say that those provisions were unduly conservative, but it is clear that much more attention was paid to restraining both flanges, rather than assuming restraint to one side only was sufficient to produce pure flexural buckling in the minor axis. This article aims to encourage designers to think carefully about such arrangements and consider how the member will buckle.

But how *does* the member behave?

Jumping forward from BS 449 to today, designers have a range of tools which can be used to investigate structural behaviour. Colin Taylor mentions making Perspex models, but today's solution is invariably software.

For the second part of this article, the software *LTBeamN* has been used, as this tool allows restraints to be placed anywhere within (or outside) the member depth and allows the fixity (both laterally and rotationally) to be specified. To investigate the behaviour in a truss, a member has been modelled with fork ends at the nodes. At the intermediate purlin position, a lateral restraint can be modelled. Specifying full lateral and rotational fixity in the software will produce the results for a fully effective lateral and torsional restraint – the chord buckling will be minor axis flexural bending between the purlin positions. The "real" situation can also be modelled, with a lateral restraint some distance outside the flange (assumed to be the centre of the bolt group to the purlin) and a varying degree of rotational fixity. The software reports the elastic critical buckling load, $N_{cr,}$ but also gives a useful graphical output of the buckling mode.

Buckling examples

The following examples are based on a 254 UC 89, arbitrarily chosen as a typical section. The nodes are at 4 m centres, and a single restraint is provided at the mid-point.

With no intermediate restraint, the member (as expected) buckles in the minor axis, between the supports. The buckled form is shown in Figure 3

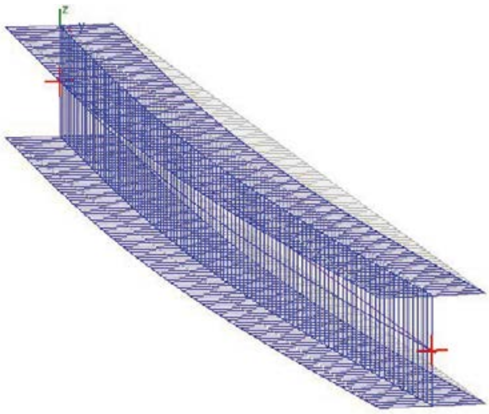

Figure 3: Buckling between supports

For the arrangement in Figure 3, N_{cr} is given as 6264 kN. For those interested, the intermediate steps and the buckling resistance in S355 are as follows: $\lambda = 0.789$; $\chi = 0.669$; $N_{b,z,Rd} = 2610$ kN

If a midspan restraint is introduced with full torsional fixity, the result is shown in Figure 4.

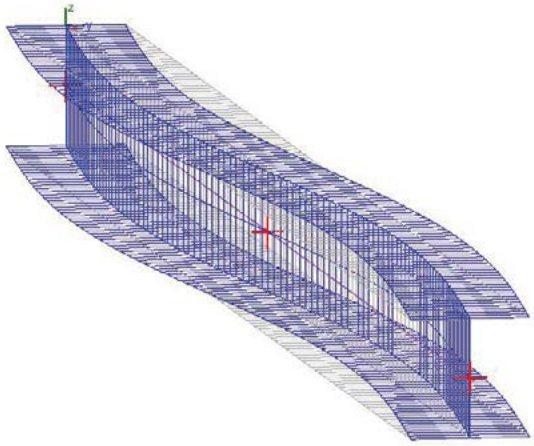

Figure 4: Buckling with lateral torsional support at midspan

For the arrangement in Figure 4, N_{cr} is given as 25069 kN. The intermediate steps and the buckling resistance in S355 are as follows: $\lambda = 0.394$; $\chi = 0.9$; $N_{b,z,Rd} = 3510$ kN

The values of 2610 kN and 3510kN can be confirmed in the Blue Book.

If a midspan restraint is provided 100 mm outside one flange only, with no torsional fixity, the result is shown in Figure 5

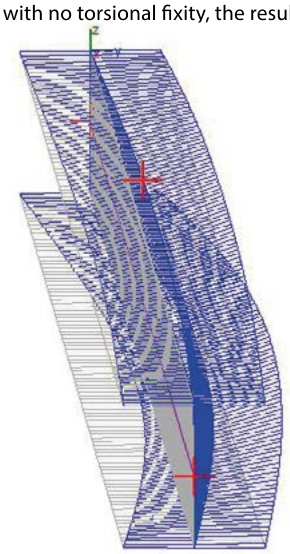

Figure 5: Buckling with lateral support at midspan, 100 mm outside the flange

In this case, both flanges have buckled laterally, not the double curvature bending shown in Figure 4 that one might have hoped for. In the case illustrated in Figure 5, N_{cr} is given as 7273 kN. The intermediate steps and the buckling resistance in S355 are as follows: $\lambda = 0.732$; $\chi = 0.705$; $N_{b,z,Rd} = 2747$ kN, which is significantly less than the resistance with an effective lateral torsional restraint.

The benefit of stiffness at the connection

The buckling form in Figure 5 resulted from a lateral restraint which was modelled to provide zero rotational stiffness. It could be argued that there is some rotational stiffness delivered by the secondary member. If this case is to be made, designers must credit the connection itself with stiffness and the ability to transfer moment, as this provides the torsional fixity to the main member. Without doing any analysis, it seems rather brave to credit a connection to a light gauge steel member with too much stiffness, as the bolts are in oversize holes and the material is thin.

With a secondary member each side of the chord, with lengths L_1, L_2 and Inertias I_1 and I_2, the stiffness can be calculated as

$$4E \frac{I_1}{L_1} + \frac{I_2}{L_2}$$

With a typical purlin length taken as 7m and a typical purlin inertia of 175 cm^4, the stiffness at the joint is calculated as 420 kNm/radian. Assuming the joint itself is infinitely stiff (which must be too optimistic, as discussed above) the midspan restraint can be credited with some rotational stiffness.

Figure 6 shows the results for the identical situation described in Figure 5, but with rotational stiffness at the restraint of 420 kNm/radian.

In fact, even with some degree of stiffness, the buckling form has not changed significantly. In this case, N_{cr} is given as 12079 kN. The intermediate steps and the buckling resistance in S355 are as follows: $\lambda = 0.568$; $\chi = 0.804$; $N_{b,z,Rd} = 3135$ kN

It may be observed that the resistance (3135 kN) appears to be approaching that when a fully effective lateral torsional restraint is provided (3510 kN). However, the rotational stiffness must be increased from 420 kNm/radian to 1660 kNm/radian before double curvature bending results. In other words, the secondary members must be around four times as stiff as is typical, before the assumption of a lateral torsional restraint is realised – and that still depends on the unlikely assumption that the connection itself is infinitely stiff.

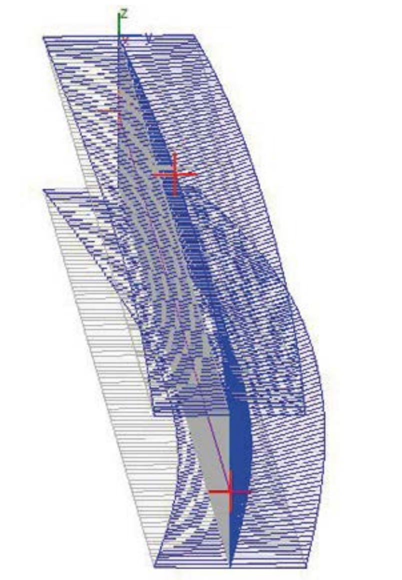

Figure 6: Buckling with lateral and rotationally stiff support at midspan, 100 mm outside the flange

Conclusion

It is hoped that this article has illustrated that restraints to only one flange of compression members should not be assumed to provide effective torsional restraint, unless carefully assessed. The advice in clause 4.10 of BS 5950 should not be used to justify such an assumption for large, heavily loaded members, as it is clear that the intended scope was limited to quite different forms of construction. If there is uncertainty about the effectiveness of the restraint, freely available software may be used to examine the behaviour of the member, modelling the location and fixity of the connecting steelwork.