The buckling resistance of laced columns

Figure 2: 33 m tall laced columns in a high bay warehouse

Photo: Winvic Construction, SEGRO Logistics Park East Midlands Gateway Plot 12

David Brown of the SCI uses the example of a laced column to demonstrate useful approaches to member buckling.

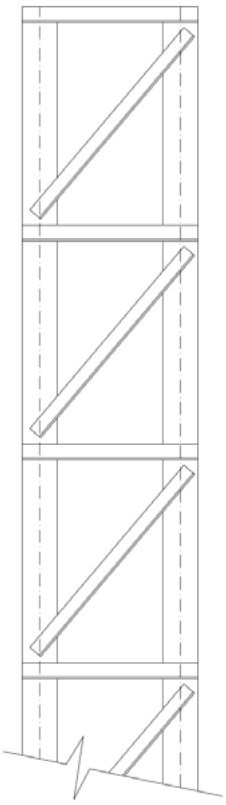

Figure 1: Laced column

Laced columns – a concept past its prime?

The short answer: mostly! A laced column consists of two main members – usually universal sections – acting as chords with a system of diagonal members in a 'N' or 'W' arrangement connecting the two chords, acting as the web of the compound section, as Figure 1. Laced columns involve significant fabrication effort, so enthusiasm for this form of column is influenced by the relative costs of labour and material. In previous decades built-up columns of this form were popular, but in the latest version of the Steel Designers' Manual they receive only a passing reference, perhaps indicating reduced enthusiasm. Laced columns *are* still used – they are very effective for high loads and tall columns found in some buildings, as illustrated in Figure 2. Laced columns may be useful when intermediate restraints are only possible to one axis of a tall, heavily-loaded column.

The primary purpose of this article is to illustrate important concepts relating to member buckling, which are demonstrated in the Eurocode rules.

Flexural buckling of laced compression members in BS 5950

All designers appreciate that buckling must be verified, usually about the two orthogonal axes of the member (a notable exception are angles). For minor axis buckling of a laced column (which is the major axis of the chord members, as shown in Figure 3), there is nothing new. The situation becomes more interesting for major axis buckling of the compound section.

BS 5950 clause 4.7.8 covers the design of laced struts and notes that the compound member may be designed as a single integral member. Table 23 allocates strut curve c when calculating the resistance of a laced strut, about either axis. In compression alone therefore, the process is straightforward. The use of a strut buckling curve allows for initial imperfections and second-order effects – notably the increase in the initial imperfection under the axial load.

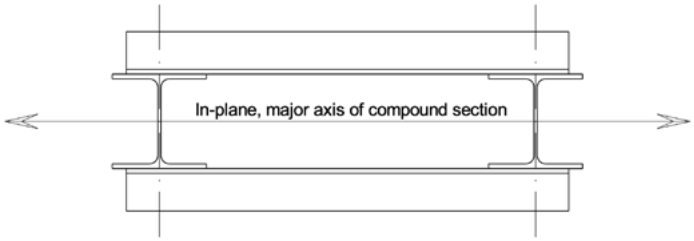

Figure 3: Laced column cross section

The challenge becomes much more complicated if the laced column is subject to an in-plane bending moment in addition to an axial compression. The expressions in section 4.8 all refer to the moment resistance of the section, being primarily suited to single rolled sections. It is not clear how to design a laced column under combined axial load and bending – designers are left to work from first principles.

The effect of shear stiffness

The shear stiffness of a laced column is significantly lower than that of a member with a solid web – much of the "web" is missing. The shear deformation of a member with a solid web is so small that it is usually ignored, but this could be significant in laced columns. Increased deformation leads to an increased moment at mid height due to the eccentricity of the cross section with respect to the ends of the member, and therefore a reduced resistance.

In BS 5950, there is no specific reference to allow for the additional deformation due to the shear flexibility. It could be that this effect is allowed for within the choice of curve c, in combination with the rules about local and overall slenderness.

Buckling of laced compression members in BS EN 1993-1-1

The Eurocode might be considered more helpful than BS 5950, since it has guidance for the design of laced columns subject to combined axial force and bending moment. The Eurocode also explicitly allows for the shear flexibility of a laced column. Perhaps of more interest is the design approach, which rather than using a buckling curve, demonstrates an alternative method to allow for imperfections and second-order effects. The following comments relate to in-plane buckling.

Usually, imperfections in members – in the form of an initial bow – and the amplification of that bow when load is applied, are dealt with through the choice of strut curve. An alternative approach is to determine the initial imperfection, amplify the imperfection and then simply complete a check of the cross section of the form:

$$\frac{\text{Force}}{\text{Area}} \times \frac{\text{Force} \times \text{final imperfection}}{\text{selection modulus}}$$

When this expression equates to the design strength of the section f_y, the buckling resistance has been established. The alternative approaches are described in clause 5.2.2.

Previous articles in New Steel Construction have reminded readers of the relationship between the initial imperfection e_0 and the final imperfection $\hat{e}$ shown in Figure 4 which is given by:

$$\hat{e} = \frac{e_o}{\left(1 - \frac{N_{Ed}}{N_{cr}}\right)}$$

Clause 6.4 of BS EN 1993-1-8 uses this method to amplify an initial imperfection, and also to allow for the reduced shear stiffness of the laced column. Once the imperfections and second-order effects have been allowed for, all that remains are "local" checks. This approach is described in clause 5.2.2(7), where "the individual stability of members should be checked... for the effects not included in the global analysis". The effect remaining to be checked is the buckling of the chord between nodes of the lacing system.

Maximum design force in the chord

With two identical chords, the design force in one chord is obviously half the applied force – but the effects of the member initial imperfection, amplification, shear flexibility and any applied moment must all be added.

The design value of the internal moment (equivalent to Force × final imperfection above) is given by:

$$M_{Ed} = \frac{N_{Ed}e_0 + M^{I}_{Ed}}{1 - \frac{N_{Ed}}{N_{cr}} - \frac{N_{Ed}}{S_v}}$$

In the numerator, $N_{Ed}e_0$ is the applied axial force multiplied by the *initial*

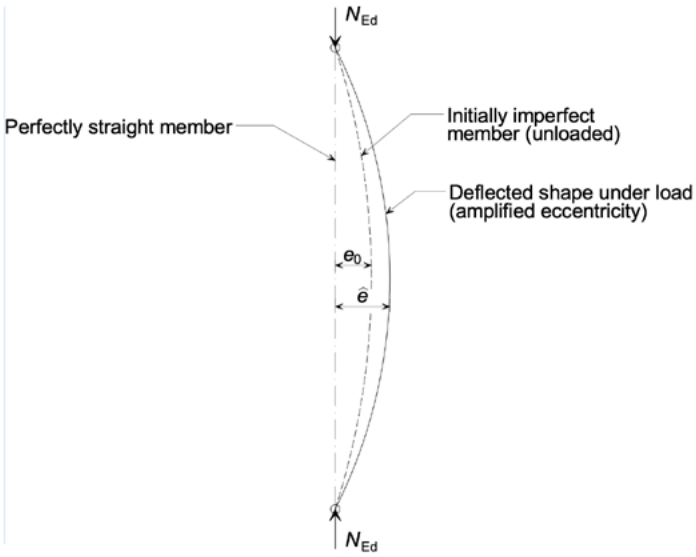

Figure 4: Imperfect strut behaviour

eccentricity. This must be amplified, so the fundamental term becomes:

$$M_{Ed} = \frac{N_{Ed}e_0}{1 - \frac{N_{Ed}}{N_{cr}}}$$ which is the same relationship in the first part of the previous expression.

M^{I}_{Ed} is the moment at the middle of the member (if any), without second-order effects. That too must be amplified.

In the denominator, the final term $\frac{N_{Ed}}{S_v}$ is the amplification of the deformation due to the shear stiffness of the lacings, S_v. Figure 6.9 of the Eurocode gives different values of S_v for various arrangements of lacing.

The design moment M_{Ed}, which allows for the effects described above, is converted into a force in the chord by dividing by the lever arm between the chords, to be added to half the applied compression. Expression 6.69 does not immediately appear to be so straightforward, since it is presented as:

$$N_{ch,Ed} = 0.5N_{Ed} + \frac{M_{Ed}h_oA_{ch}}{2I_{eff}}$$, but since $I_{eff} = 0.5h_o^2A_{ch}$ the expression simplifies to $N_{ch,Ed} = 0.5N_{Ed} + \frac{M_{Ed}}{h_o}$

Perhaps there was some good reason for the more complicated presentation.

How do the codes compare? – BS 5950

A convenient worked example of design to the Eurocode is contained in Reference 1. Each chord is a HE 220 A, in S355, spaced 800 mm apart. The member is 10 m long. Each chord has an area of 6430 mm². For each chord, i_z = 55.1 mm

The in-plane second moment of area I_{eff} of the compound section is 2058 × 106 mm⁴

BS 5950 clause 4.7.8 g) places a limit on the slenderness of the chord between nodes, and on the slenderness of the overall member, so the best starting point is with a chord length between nodes.

Local buckling of chord

In Reference 1 the buckling length is 1125 mm

In the minor axis of the chord, $\lambda_c = \frac{1125}{55.1} = 20.4 < 50$, OK.

The chord flange is 11 mm, so f_y = 355 N/mm²
From Table 23, the strut curve is curve c (the section is a "H" profile)
From Table 24, p_c = 345 N/mm²
P_c = 345 × 6430 × 10⁻³ = 2218 kN per chord, 4436 kN in total.

Overall buckling of compound member

Radius of gyration in-plane = $\sqrt{\frac{2058 \times 10^6}{2 \times 6430}} = 400$ mm

Slenderness = $\frac{10000}{400} = 25$

However, the minimum value is $1.4\lambda_c = 1.4 \times 20.4 = 28.6$
(for convenience, take 30)
From Table 23, the strut curve is curve c
From Table 24, p_c = 324 N/mm²
P_c = 324 × 6430 × 2 × 10⁻³ = 4167 kN
The resistance of the section in this axis is limited by the overall buckling, not the local buckling of the chords between nodes.

How do the codes compare? – BS EN 1993-1-1

From 6.4.1(1) the value of $e_0 = \frac{10000}{500} = 20$ mm

As given in the example, N_{cr} = 42650 kN and S_v = 134100 kN

Also as given, $N_{b,Rd}$ = 2203 kN (reassuringly similar to 2218 kN calculated above in accordance with BS 5950)

Assuming no externally applied moment, to make the example compatible,

$$M_{Ed} = \frac{N_{Ed} \times 0.02}{1 - \frac{N_{Ed}}{42650} - \frac{N_{Ed}}{134100}}$$

and $N_{ch,Ed} = 0.5N_{Ed} + \frac{M_{Ed}}{0.8}$

The maximum resistance is when $N_{ch,Ed} = N_{b,Rd}$, so using "goal seek" within Excel for convenience, N_{Ed} is found to be 4167 kN

To check: $M_{Ed} = \frac{4167 \times 0.02}{1 - \frac{4167}{42650} - \frac{4167}{134100}} = 95.66$kNm

and $N_{ch,Ed} = 0.5 \times 4167 + \frac{95.66}{0.8} = 2203$ kNm, OK.

Somewhat incredibly, the resistance according to BS EN 1993-1-1, is exactly the same as that calculated to BS 5950.

For interest, the terms in the denominator to calculate M_{Ed} are (1 - 0.098 - 0.03) which together lead to a 15% amplifier in the value of e_0. The values give some indication of the effect of shear flexibility – not very significant in this particular example. The shear flexibility obviously varies with the particular arrangement or lacing members proposed. American standards make a further distinction between lacing members that are welded or use preloaded assemblies, and those that are bolted with ordinary bolts in clearance holes. The latter introduces more flexibility into the system and reduces the overall resistance by up to 10%.

The out-of-plane buckling must be verified separately.

Conclusions

The primary purpose of this article was not to promote laced columns, but to demonstrate that buckling behaviour can be addressed either:

- within the member checks, (section 6.3 of the Eurocode) where member imperfections and second-order effects are automatically included, or
- by including the effects of imperfections and second-order effects within the analysis, leaving only local checks (usually just a cross sectional check but in the case of a laced column, the local check is still a buckling check).

A laced column is one example where the second approach is very helpful, as applied moments can be included in the design – a situation not covered in BS 5950. A second more common example is the in-plane buckling of portal frames. In-plane, imperfections and second-order effects are allowed for (if necessary) in the global analysis, meaning only a cross sectional check is needed. ■

1. Single-storey steel buildings Part 6: Detailed design of built-up columns available from arcelormittal.com