Truss joint design – open sections

Following on from the truss joint design presented in the October issue, David Brown of the SCI reviews the design of a joint with a conventional arrangement of open sections.

Conventional – or common?

The October issue of New Steel Construction addressed a heavily loaded joint in a truss, explaining the thought process that led to the decisions firstly to orientate the UC chord members with web horizontal, secondly to use similar sized sections for the web members to facilitate the joint design and lastly to fabricate the node from plate.

In common practice, truss joints between open sections are often simply arranged with the webs vertical and with the web members as smaller sections than the chords.

That arrangement leaves the connection designer to determine how the forces in the members are to be transferred, recognising that elements in the joint are often perpendicular to each other, which is never ideal.

The particular joint considered in this article is shown in Figure 1, although the thought process and element verifications are more important than the actual detail.

The vertical web member has an axial compression of 1800 kN. The diagonal, which is at 45°, therefore has an axial tension of 2545 kN and the joint is in vertical equilibrium. Many connection designers will release anguished howls at this point, since in reality they are unfortunately often given 'envelope' forces which are not in equilibrium and therefore doubly challenging to address.

For the purposes of this example, it is assumed that the force in the chord is 75% of its tension resistance. Because the flange of the 305 UC 158 is 25 mm, the design strength is 345 N/mm^2 (all members are S355) and the axial force is therefore 5200 kN.

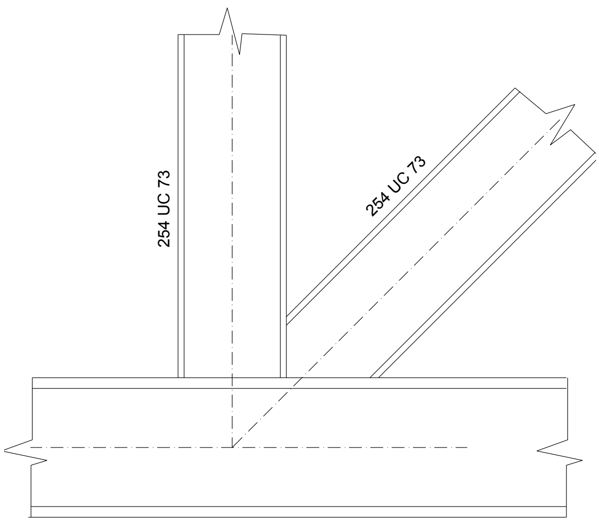

Figure 1: Open section joint

Distribution of forces

A helpful approach is to consider how the forces are distributed within the elements of the cross section. The area of a UC flange is typically 40% of the entire cross section (38.8% for the 254 UC 73), meaning that the element forces in the diagonal and vertical members are as shown in Figure 2.

At the connection points, these element forces have been further split into the two orthogonal components.

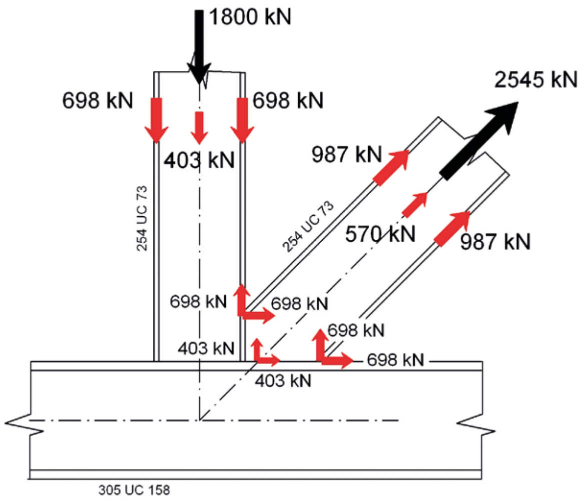

Figure 2: Forces in cross section elements

Connections to unstiffened flanges

Under local loads, webs might need reinforcement under compression, or under tension.

Before those checks are considered, stiffeners might also be required to stiffen the flange so that the full width of connected parts is effective. Stiffeners required for this purpose are more likely to be needed than to reinforce the web, so it is wise to complete these checks first.

There are connections to (potentially) unstiffened flanges at points A and B (in tension) and C and D (in compression) as shown in Figure 3.

If the flange is unstiffened, the more flexible tips of the flange deform and the stress distribution across the connected plate (in this case the flange of the incoming UC) is non-uniform. Design codes calculate an effective breadth, over which the stress is assumed to be uniform.

The verification is covered in clause 4.10 of BS EN 1993-1-8. The effective breadth, b_{eff} must be calculated, which assumes a spread through the flange from the web and root radius. b_{eff} is given by:

▶28

▶26

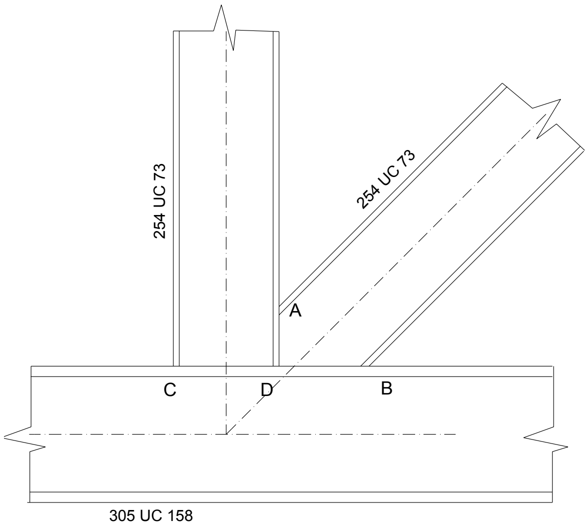

Figure 3: Connections to potentially unstiffened flanges

$b_{eff} = t_w + 2s + 7kt_f$

where $k = \left({t_f}/{t_p}\right)\left({f_{y,f}}/{f_{y,p}}\right)$ but $k \leq 1$

The requirement is then:

$b_{eff} \geq \left({f_{y,p}}/{f_{u,p}}\right) b_p$

It should be noted that both $f_{y,p}$ and $f_{u,p}$ relate to the plate (again in this case, the flange of the incoming UC). This requirement means that:

$b_{eff} \geq 0.75b_p$

If this requirement is not met, then clause 4.10(3) says "Otherwise the joint should be stiffened". Readers will note that the applied force has not featured in this verification – the check is purely geometric, without reference to any force. If the force is small, this requirement seems unreasonable.

BS 5950 had an altogether more sensible approach in clause 6.7.5. The applied force F_x was limited to the resistance P_x obtained from the effective breadth, so connections with low forces could be accommodated without stiffening.

According to BS 5950, stiffening had to be provided if $b_e < 0.5\ (F_x\,/P_x)\ b_p$ but this is a much less onerous requirement than the Eurocode.

At point A, the effective width, b_{eff} is 133mm ($k = 1$)

The limit $= \left({f_{y,p}}/{f_{u,p}}\right) b_p\ = (355/470) \times 254.6 = 193.5$ mm

So according to the Eurocode, stiffening is required at point A.
At point B, the value of k in clause 4.10(2) is calculated as 1.7, but limited to a maximum of 1.0.
The effective width, b_{eff} is 221 mm ($k = 1$)

The limit $= \left({f_{y,p}}/{f_{u,p}}\right) b_p\ = (355/470) \times 254.6 = 193.5$ mm,

which means that stiffening might not be needed – other verifications need to be completed.

Tension stiffener design

At point A, it is convenient simply to assume all the applied horizontal component must be carried into the stiffeners in the vertical member. The resistance of two stiffeners, each 120 × 10 mm in S355 is 852 kN, which exceeds the 698 kN applied.

The weld to the inside of the flange is continued round the root radius, rather than being stopped, so only one leg length (strictly to the Eurocode, a throat length) is deducted from the weld length.

Thus there is 4 × (120 – 8) = 448 mm of weld, assuming an 8 mm fillet weld. This is a transverse weld, so has a resistance of 1.65 kN/mm. The applied force is 698 / 448 = 1.56 kN/mm, so 8 mm fillet weld is OK.

That force must be transferred to the web, between fillets, (it has nowhere to go at the other flange!), so the force in the weld is 698 / (4 × 200) = 0.87 kN/mm. A 6 mm fillet weld would be OK, but practically the same 8 mm fillet weld all round would be specified. Note that this force transferred into the web appears as a shear force in the vertical member.

Web in tension at point B

Although no stiffeners to support the flange are needed, the web of the chord experiences the local tension of 698 kN.

The resistance of the web is given in BS EN 1993-1-8 clause 6.2.6.3, which involves an effective breadth of web, $b_{eff,t,wc}$ and a reduction factor ω due to shear in the web.

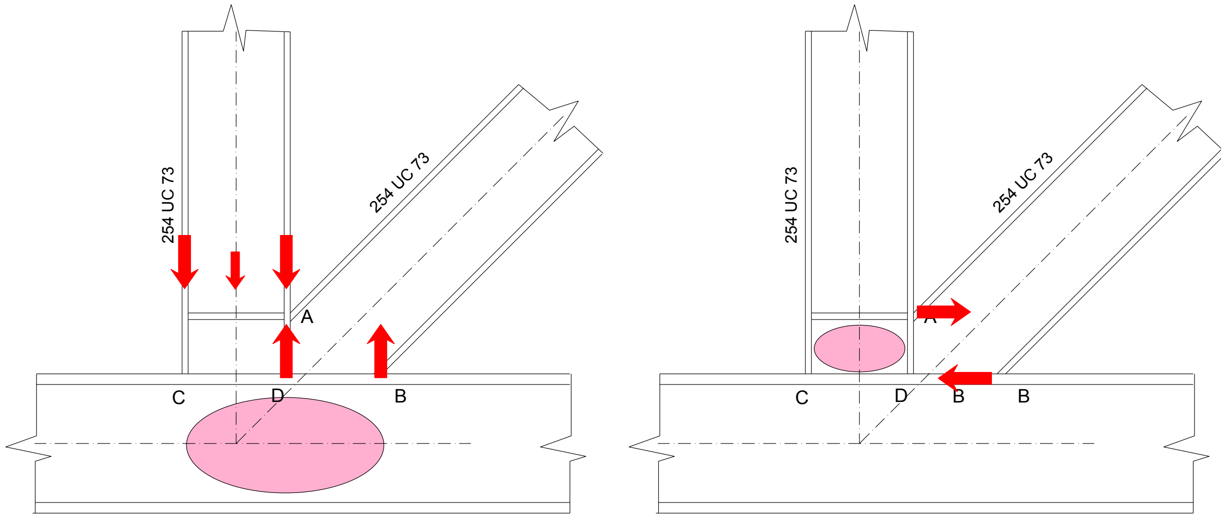

Figure 4: Zones where shear resistance must be verified

This is determined from Table 6.3, which leads backwards to Table 5.4 and a challenging decision on the value of β to be taken. After some consideration, the situation seems most like the shear in a web panel from a one sided moment connection, so $\beta = 1$.

After a frustrating trip back to BS EN 1993-1-1 to calculate the shear area, ω is computed to be 0.82.

The web resistance is computed to be 988 kN, which is more than the applied force of 698 kN, so no stiffener is needed for web tension.

Shear resistance

The shear in the web of the chord and in the web of the vertical member can be calculated by considering the components of force in the appropriate direction, as shown in Figure 4.

A convenient approach is to draw the local shear force diagram due to the applied components of force. Note that this only works if the applied forces are in equilibrium. The shear force diagram for the chord is shown in Figure 5.

Looking in the Blue Book, the shear resistance of the 305 UC 158 is 1130 kN, so it seems highly unlikely that the chord web will be satisfactory when the shear stress is considered in combination with the axial stress.

Shear and axial stress combined

The combination of stresses can be considered using the Von Mises criterion, found in clause 6.2.1 of BS EN 1993-1-1. Designers may not often use this clause, as normal cases have their own specific verifications later in section 6, but this elastic check is useful in unorthodox situations.

Considering just longitudinal and shear stresses, the criterion becomes:

$$\left(\frac{\sigma_{x,Ed}}{f_y / \gamma_{M0}}\right)^2 + 3\left(\frac{\tau_{Ed}}{f_y / \gamma_{M0}}\right)^2 \leq 1$$

The ratio $\left(\frac{\sigma_{x,Ed}}{f_y / \gamma_{M0}}\right) = 0.75$

The value of τ_{Ed} must be calculated as it is the elastic shear stress at the neutral axis. Normally, designers calculate a plastic resistance, so do not know the value of τ_{Ed}.

Designers will use an expression such as $\tau = \frac{SA\bar{y}}{It}$, depending on the form of the mnemonic they use!

The values of $\bar{y}$ and A can be taken directly from the section properties for tees cut from UC sections.

$$\tau_{Ed} = \frac{1101 \times 10^3 \times 10100 \times 13^3}{38700 \times 10^4 \times 15.8} = 242 \text{ N/mm}^2$$

Substituting into the Von Mises criterion:

$(0.75)^2 + 3\left(\frac{242}{345/1}\right)^2 = 2.03$, which is unsatisfactory, as expected.

▶30

▶29

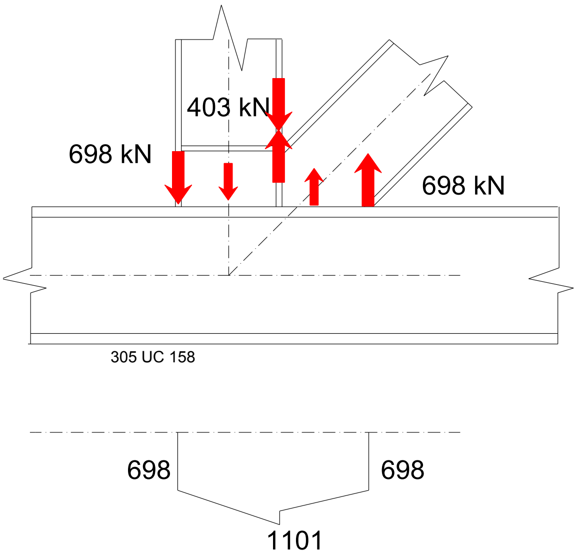

Figure 5: Shear force diagram for the chord

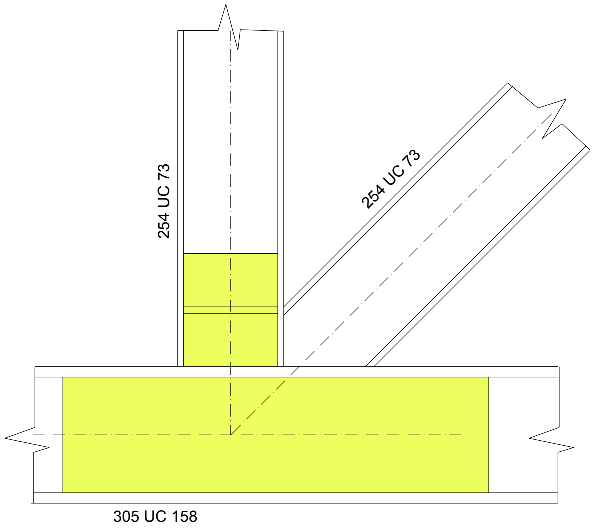

Figure 6: Final joint detail

Supplementary web plate

A supplementary web plate is one option, with the design rules given in clause 6.2.6.1. Because of limited research, the contribution of a supplementary web plate is limited to a maximum thickness equal to the web it reinforces, even if the additional plate is thicker than the web. Adding a plate to the other side of the web makes no further increase in the shear resistance, which seems implausible.

With the objective of using the Von Mises criterion a second time, the shear stress in the compound section must be calculated. Although a thicker plate was selected, the calculated inertia, area and distance to the centre of gravity used only the additional 15.8 mm permitted by the Standard. The longitudinal stress was also reduced by considering the additional area, once again limiting the credited addition to the 15.8 mm, despite specifying a 20 mm plate.

The calculates stresses were

τ_{Ed} = 125 N/mm^2 and σ_x = 218 N/mm^2

Substituting the Von Mises criterion:

$$\left(\frac{218}{345/1}\right)^2 + 3\left(\frac{125}{345/1}\right)^2 = 0.79$$

The length of plate past the critical area needs to be sufficiently long so that the welds can transfer the axial forces assumed in the supplementary web plate.

Other checks

The same process is needed for shear in the vertical member (see Figure 4), where it will be found that reinforcement is also required. Welds between the web and chord members must also be designed.

The compression resistance of the web at point C (Figure 3) requires verification – but is not a problem with the supplementary web plate provided.

The final joint is indicated in Figure 6. Instead of supplementary web plates, a detail using diagonal shear stiffeners could be developed, although the room for diagonal members is rather limited.

Conclusions

As always, a thoughtful consideration of the member selection and member orientation might have avoided some of the more expensive reinforcement required for this particular detail. A second observation is that the necessity to stiffen without any reference to the applied force seems very onerous – it is hoped that some work can be done to modify this requirement.

The good news is that the proposed revisions to BS EN 1993-1-8 do allow more benefit to be taken from supplementary web plates. Finally, the example serves as a reminder that the Von Mises criterion, presented in clause 6.2.1, can be useful when no other option exists.

FROM

Building with Steel

February 1970

High-Rise Trusteel

The Trusteel steel-framed system is well known in the low rise housing field, 37,000 Trusteel homes equally in the Local Authority and Private Housing fields, testify to this.

When the recent requirements for high loadings were introduced, Trusteel decided that the time had come for steel to re-enter the high rise field and by the wide use of Trusteel's existing components, a most economical solution was found.

Advantage was taken of the 'Report of the enquiry into the collapse of flats at Ronan Point', Ministry Circular No. 62/68 and Trusteel found that a steel structure was clearly the best means of meeting the requirements for structural safety. Firstly, hot-rolled sections are used to form a multi-storey portal longitudinally and to tie walls and floors together against explosion and secondly Trusteel's lightweight beams and channels form the floor units.

The possible combination of dead, super and wind loads together with suitable section data was run through a computer to determine the most economical sections, the foundation loading, frame strength and wind sway.

The detailing was such that three dimension units could be assembled on the ground before being lifted into position, thus greatly reducing work at high levels, and greatly increasing safety.

Trusteel believe that there is now no longer any doubt that steel used correctly can be competitive in the high rise field and in every way is a lot safer.