The design of hybrid fabricated girders - part 2

In Part two of the article, David Brown of the SCI discusses the lateral torsional buckling resistance and shear resistance of hybrid sections.

Cross sectional moment resistance – made easy

In Part 1, the cross section was classified as Class 4 (due to the web), leading the designer to BS EN 1993-1-5 and the challenge of calculating effective section properties. There is an easier approach, but the penalty is a conservative resistance.

Clause 5.5.2(12) of BS EN 1993-1-1 allows the class of the cross section to be based on that of the flanges alone, as long as it is assumed that the web is designed for shear alone and does not contribute to the bending resistance. This simple approach does not relieve the designer of considering shear lag and plate buckling, as clause 6.2.1(2) of BS EN 1993-1-1 makes clear.

In the example presented in Part 1, the flanges were Class 3. If the web is assumed to make no contribution to the bending resistance, then the resulting stress diagram is shown in Figure 1.

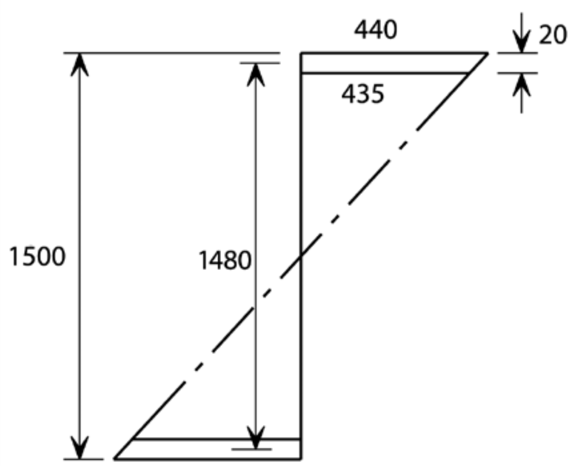

Figure 1: Class 3 stress diagram, neglecting the web

As demonstrated in Part 1, the flange does not suffer from shear lag effects or plate buckling effects.

Thus the bending resistance of the cross section = 400 × 20 × 437.5 × 1480 × 10^{-6} = 5180 kNm

The resistance calculated in Part 1 was 6485 kNm; an increase of 25% over the simple approach.

Lateral-torsional buckling

There is no real challenge in LTB resistance. The section properties must be calculated, or the on-line M_{cr} tool on *steelconstruction.info* can be used to calculate the section properties – and to compute M_{cr}, of course.

For those who prefer to see the calculations:

$$I_z = 2 \times \frac{20 \times 400^3}{12} + \frac{1600 \times 12^3}{12} = 213 \times 10^6 \text{ mm}^4$$

$$I_w = \frac{I_z \times h_0^{\,2}}{4} = \frac{213 \times 10^6 \times (500 - 20)^2}{4} = 1.17 \times 10^{14} \text{ mm}^6$$

$$I_t = \frac{2}{3} b_f t_f^{\,3} + \frac{1}{3} h_w t_w^{\,3} = \frac{2}{3} \times 400 \times 20^3 + \frac{1}{3} \times 1460 \times 12^3 = 2.97 \times 10^6 \text{ mm}^4$$

(the calculation for I_t is a simplification; online tools give 2.92 × 10^6 mm^4)

Assuming the loading is uniformly distributed, then C_1 = 1.13. At this point, the value of M_{cr} can be calculated using the steel designer's favourite (or maybe not?) expression:

$$M_{cr} = C_1 \frac{\pi^2 E I_z}{L^2} \sqrt{\frac{I_w}{I_z} + \frac{L^2 G I_t}{\pi^2 E I_z}}$$, which computes to 5966 kNm.

It does seem rather odd to use online tools to calculate section properties and then compute M_{cr} by hand, when the same software will calculate M_{cr} to be 5970 kNm.

The general case given in clause 6.3.2.2 of BS EN 1993-1-1 is used for fabricated girders.

h/*b* = 1500/400 = 3.75, so curve *d* is used and α_{LT} = 0.76

Working through the expressions in clause 6.3.2.2, the reduction factor χ_{LT} = 0.447 and M_b = 0.447 × 6485 = 2899 kNm.

Web resistance

The web must resist shear, of course, but must also prevent the flange buckling in the plane of the web (a possibility for very tall, thin webs). When calculating the shear resistance, the presence of stiffeners (or not) makes a significant difference, as will be demonstrated. The shear resistance comprises a contribution from the web, but also an additional contribution from the flanges. The flanges can span between stiffeners and mobilise a tension field mechanism (see Figure 2) – essentially like a tension member in a truss. The contribution from the flanges is generally small and can only be used if the flanges are not fully utilised in carrying moment – so a simple solution is to neglect the additional resistance.

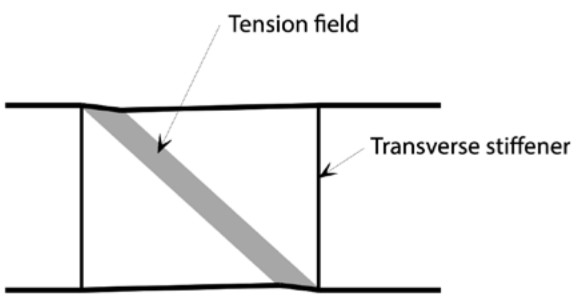

Figure 2: Flange contribution to web shear resistance (from Hendy and Murphy)

Shear resistance

The consideration of shear resistance starts in BS EN 1993-1-1, with an expectation that for tall thin webs, shear buckling will be critical (clause 6.2.6(6)).

For the web, $\varepsilon = \sqrt{\frac{235}{355}} = 0.81$. With η=1, then

$$\frac{h_w}{t_w} = \frac{1460}{12} = 121.7\,; \quad 72\frac{\varepsilon}{\eta} = 72 \times \frac{0.81}{1} = 53.3$$

▶27

▶28 A check of shear buckling is therefore required (the web would need to be 25 mm thick before a check of shear buckling is not needed), which takes designers to BS EN 1993-1-5 clause 5.2. Initially, the resistance of an unstiffened web will be calculated.

Unstiffened section - contribution from the web – clause 5.3

A series of intermediate values are required, determined from Annex A.3 and Annex A.1 of BS EN 1993-1-5

From Annex A.3, if the unstiffened span is 8 m, then

$\frac{a}{h_w} = \frac{8000}{1460} = 5.48$. Because this value is greater than 1.0, the shear buckling coefficient, k_τ is given by: $k_\tau = 5.34 + 4\left(\frac{h_w}{a}\right)^2 + k_{\tau sl}$

$k_{\tau sl}$ relates to longitudinal stiffeners, so is zero in this example. Therefore, $k_\tau = 5.47$

From Annex A.1, $\sigma_E = 190000\left(\frac{t}{b}\right)^2 = 190000 \times \left(\frac{12}{1460}\right)^2 = 12.84$

From clause 5.3(3), $\tau_{cr} = k_\tau \sigma_E = 12.84 \times 5.47 = 70.2$

Then $\bar{\lambda}_w = 0.76\sqrt{\frac{f_{yw}}{\tau_{cr}}} = 0.76\sqrt{\frac{355}{70.7}} = 1.71$

Because $\bar{\lambda}_w > 1.08$ (From Table 5.1), $\chi_w = \frac{0.83}{\bar{\lambda}_w} = \frac{0.83}{1.71} = 0.49$

Thus the contribution from the web =

$\frac{0.49 \times 355 \times 1460 \times 12}{\sqrt{3} \times 1.0} \times 10^{-3} = 1745$ kN

The contribution from the flange, assuming $M_{Ed} = 0$ at the supports, is 33.5 kN, so less than 2% of the contribution from the web – and small enough to be neglected.

The plastic shear resistance should be verified in accordance with clause 6.2.6(2) of BS EN 1993-1-2, and in this example is found to be 1980 kN – as expected, shear buckling is critical.

Stiffened section - contribution from the web – clause 5.3

If intermediate transverse stiffeners are provided, the shear resistance increases. In this case, it is assumed that the transverse stiffeners are spaced such that $a/h_w = 2$ (Figure 3).

Then k_τ=6.34; σ_E=12.84; τ_{cr}=81.4; $\bar{\lambda}_w$=1.59; χ_w=0.522 and the web resistance increases to 1874 kN

In this stiffened case, the additional contribution from the flanges increases to 96 kN, so is more significant.

Moment resistance or shear resistance – which is critical?

If, as calculated above, the LTB resistance, M_b is 2899 kNm, and

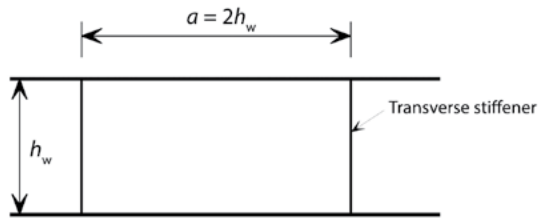

Figure 3: Aspect ratio of web panel

assuming that M_{Ed} is 90% of M_b, then the UDL is 326 kN/m

The end shear is therefore 1304 kN, so in this example, even the unstiffened shear resistance of 1745 kN is sufficient.

Flange induced buckling (clause 8)

To prevent flange induced buckling, the following criteria must be satisfied:

$$\frac{h_w}{t_w} \le k\frac{E}{f_{yf}}\sqrt{\frac{A_w}{A_{fc}}}$$

Since the elastic moment resistance has been calculated, the factor $k = 0.55$

Then $\frac{1460}{12} \le 0.55\,\frac{210000}{440}\sqrt{\frac{1460 \times 12}{7999}}$ or $121.7 \le 388.5$

The criteria is satisfied, so there is no flange induced buckling of the web.

Further guidance

The Designer's Guide to EN 1993-2 by Hendy and Murphy has extensive coverage of BS EN 1993-1-5. *Structural Design of Steelwork to EN 1993 and EN 1994* by Martin and Purkiss contains examples of fabricated section design including the design of intermediate transverse stiffeners and end posts. Readers of the second resource should note that the larger elastic section modulus (not the smaller) is calculated in example 5.4, with a consequent problem in the calculated resistance.

Conclusions from Part 2

This article has attempted to introduce the design rules that apply to any fabricated beam section with a Class 4 web – the fact that the web and flanges are of different grades is not significant. The hard work was completed with the calculation of the cross section moment resistance, covered in Part 1. On-line tools cannot help here, as the calculation of M_{cr} – where on-line tools are invaluable – is based on the gross section properties, not the effective section properties needed for the cross sectional resistance.