AD 411: Design of web to flange welds in plate girders

The SCI Advisory Desk is frequently asked how to design the welds between the flange and web of a plate girder. The following note discusses the standard formula for the shear flow between web and flanges of a doubly symmetric beam which is used for weld design and gives the background to the formula in Eurocode 3 Part 5[1]. An example is also presented.

The design of a plate girder element is the responsibility of the building structural engineer just as is the design of a rolled section beam. The difference is that plate girder design involves choosing explicitly the width and depth of the beam and also the thicknesses and arrangement of the constituent plates, including the connection between them. The web to flange welds are not connections between elements so in the contractual arrangement usually adopted on projects, their design is not in the steelwork contractor's scope of work.

The relevant stresses in the beam which are carried by the web to flange welds are the shear stresses which act on planes parallel to the longitudinal axis of the element and are the result of the change in bending moment over an incremental length of the beam. Shear stresses which are equal and perpendicular to the longitudinal stresses are developed in the plane of the cross section and are termed "complementary" shear stresses. The sum of these stresses over the area of the cross section equals the applied shear force. The stresses are determined using the standard formula for calculating the shear stress distribution over the cross section which is found in strength of materials text books:

$$\tau = \frac{V_{Ed} A \bar{z}}{I_z b} \qquad \text{equation 1}$$

where:

- τ is the shear stress at a point in the cross section a distance z from the neutral axis of the section;
- V_{Ed} is the design shear force on the section;
- A is the area of the cross section further from the neutral axis than z;
- z is the distance from the neutral axis to the centroid of area A;
- I_z is the second moment of area of the whole cross section;
- b is the width of the section at the point considered.

Applying the formula to a rectangular cross section with the long dimension vertical carrying a vertical shear force produces a parabolic distribution of shear stress over the section which is a maximum at the neutral axis and zero at the top and bottom. When applied to an I section it produces the familiar distribution showing that most of the shear force is carried by the web of the beam.

When considering weld design, equation 1 can be written in terms of shear flow s between the flange and web by substituting $s = \tau b$ as shown:

$$s = \frac{V_{Ed} A_f \bar{z}}{I_z} \qquad \text{equation 2}$$

where A_f is the area of the flange. The shear flow is the shear force per unit length which is to be carried by the weld.

Part 5 of Eurocode 3 gives conservative and simplified formulae for sizing web to flange welds in clause 9.3.5(1) as follows:

$$s = \frac{V_{Ed}}{h_w} \text{ if } V_{Ed} \le \chi_w f_{yw} h_w t / \sqrt{3}\gamma_{M1} \qquad \text{equation 3}$$

where h_w is the depth of the web. For larger values of V_{Ed} the weld should be designed for

$$s = \eta f_{yw} t / \sqrt{3}\gamma_{M1}$$

Equation 3 is used if the shear force on the web is less than the shear buckling resistance of the web which is given by the expression on the RHS of the inequality. Clause 5.1(2), gives a value of slenderness for an unstiffened web where shear buckling does not arise:

$$\frac{h_w}{t} < \frac{72}{\eta}\varepsilon \text{ where } \varepsilon = \sqrt{\frac{235}{f_y}}$$

Tests have shown that the shear resistance of a stocky web exceeds the resistance predicted by the Von Mises yield criterion due to strain hardening. This effect is allowed for by including the factor η, the value of which is subject to national choice. According to the UK National Annex, η should be taken as equal to 1.0, ie the effect of strain hardening is ignored.

The simple formula for shear flow in equation 3 can be shown to be a conservative approximation if the second moment of area of the plate girder is based on the second moment of the flanges with respect to the neutral axis (ie neglecting the web and the second moments of the flanges about their own centre-line). The $A_f z$ term is the first moment of the flange about the neutral axis of the beam. Substituting these values in equation 2 gives:

$$s = \frac{V_{Ed} A_f (h_w + t_f)/2}{A_f (h_w + t_f)^2/2} = \frac{V_{Ed}}{(h_w + t_f)} \approx \frac{V_{Ed}}{h_w} \qquad \text{equation 4}$$

Neglecting the thickness of the flange in calculating the shear flow is clearly conservative.

Example

A 10m span plate girder 600 mm deep by 300 mm wide with 30 mm thick flanges and a 10 mm thick web (steel grade S355) carries a central point load of 800 kN. The top flange of the beam is fully restrained. Size the web to flange welds.

$I_z = 1/12(600^3 \times 300 - 540^3 \times 290) = 1.60 \times 10^9$ mm^4.
$W_p = 30 \times 300 \times 285 \times 2 + 270 \times 10 \times (270/2) \times 2$
$= 5.86 \times 10^6$ mm^3.

$M_R = 345 \times 5.86 \times 106/109 = 2.02$ MNm.
$M_{Ed} = 800 \times 10/4 = 2.0$ MNm
ie the beam is sized for bending.

Eurocode 3 Part 5:
Web slenderness: $h_w/t = 540/10 = 54$. The limiting slenderness is $72\varepsilon = 58.6$ so the web is not slender and shear buckling does not arise ie $\chi_w = 1.0$.

The limiting value of design shear force:
$V_{Ed} = 1.0 \times 355 \times 40 \times 10 / \sqrt{3} \times 1.0 = 1106 > 400$kN

The simple formula can be used:
$s = 400 / 540 = 0.74$ kN/mm. For two welds, this is 0.37 kN/mm per weld. The 6mm leg fillet weld length required (longitudinal resistance, 1.01 kN/mm) over 200 mm = $(200 \times 0.37)/1.01 = 73$ mm. Adding twice the leg length for stops and starts gives 85 mm: use 90 mm. Provide an intermittent 6mm fillet weld on both sides of the web, 90 mm hit and 110 mm miss. The average shear resistance per mm is $(90–12)/200 \times 1.01 = 0.39 > 0.37$ kN/mm – OK.

Apply the standard formula:
$s = 400 \times 9000 \times 285 / 1.6 \times 109 = 0.64$ kN/mm. For two welds this is 0.32 kN/mm per weld. The 6mm fillet weld leg length required over 200 mm = $200 \times 0.32/1.01 = 63$ mm. Adding twice the leg length for stops and starts gives 75 mm: use 80 mm. Provide an intermittent 6 mm fillet weld on both sides of the web 80 mm hit and 120 mm miss. The average shear resistance per mm is $(80 – 2)/200 \times 1.01 = 0.34 > 0.32$ kN/mm – OK.

The simple formula in Eurocode 3 is more conservative.

The size of the smallest continuous fillet weld which is just sufficient to transfer the web to flange shear flow may be impractically small (a 3.0 mm leg fillet weld has a longitudinal shear resistance of 0.51 kN/mm). A larger intermittent fillet weld can be used, as in this example, but is not suitable for elements where corrosion is an issue because the web to flange joint is unsealed where there is no weld. In practice, a steelwork contractor may choose to provide a continuous fillet weld to avoid having to set out all the stops and starts. The works may also have a standard weld procedure for the relevant plate thicknesses with a pre-determined size of fillet weld which is larger than the calculated value.

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1. BS EN 1993-1-5:2006 Eurocode 3 – Design of steel structures – Part 1-5 design of plated elements