Shear buckling resistance of non-uniform thickness bridge girder webs

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Summary

In very large steel bridge girders, the web often must be composed of more than one plate strip. As an alternative to longitudinal stiffeners of a slender web of uniform thickness, the bottom web plate strip may be designed as a vertical extension of the bottom flange – thicker than the upper web strip. A thicker bottom web strip enhances both the shear buckling resistance of the web and the bending moment resistance of the cross-section. The magnitude of these beneficial effects are addressed in this article. Moreover, the effect on the shear-bending resistance interaction is investigated. Non-linear finite element simulation is conducted and comparison to the predictions of the EC 3 is made.

Keywords: Bridge girders, shear buckling, shear-bending interaction, non-linear finite element simulation.

1. Introduction

In Swedish steel and composite bridges the post-critical behaviour of the web plates has been accounted for since some 20 years, and for other kinds of steel structures for 40 years. This means that the flanges can be fully utilized in bending, although this is not the case with the slender web plates connecting them. In other European countries this is not allowed, and this means that the web often have longitudinal stiffeners, which increase the critical stress for the web in bending, but also the total costs. Another aspect is that the web must be able to carry the vertical shear force, and this can lead to demands for either thicker webs or longitudinal stiffeners to increase the buckling coefficient, k_τ . Since the shear capacity increases much faster than the thickness of the web, it is however often advantageous to increase the web a few mm's, instead of introducing longitudinal stiffeners.

For very high girders, however, the web must often be composed of more than one plate strip. This gives an excellent opportunity to increase the thickness of the web part in compression, which will increase the shear capacity, and even more so the bending capacity of the steel cross section. If for example the lower 25% of a cross section at support is composed of a thicker plate, the bending stresses have decreased to 50% at the intersection to the thinner web plate. Furthermore, most of the extra material placed in the web can be excluded from the bottom flange. Although the material put in the flange has a somehow better cantilever arm in bending, the shear lag in a wide bottom flange to some extent can compensate for this.

2. Shear buckling of uniform thickness slender web plates

2.1 Resistance prediction of the EC3

The shear buckling resistance of an un-stiffened slender uniform thickness web is governed by the EC3 slenderness parameter, [1],

$$\bar{\lambda}_w = \sqrt{\frac{\tau_y}{\tau_{cr}}} = \sqrt{\frac{f_y}{\sqrt{3}\tau_{cr}}} = 0,76 \sqrt{\frac{f_y}{\tau_{cr}}} \quad (1)$$

For a long web this turns into

$$\bar{\lambda}_{w\infty} = 0,35 \frac{h_w}{t} \sqrt{\frac{f_y}{E}} \quad (2)$$

For a rectangular web by which the length to width ratio is a/h_w it holds

$$\bar{\lambda}_w = \frac{0,81}{\sqrt{5,34 + 4(h_w/a)^2}} \frac{h_w}{t} \sqrt{\frac{f_y}{E}} \quad (3)$$

In any case, the buckling reduction factor for a slender web with rigid end post is given by

$$\chi_w = \frac{1,37}{0,7 + \bar{\lambda}_w} \quad (4)$$

and the resistance is

$$V_R = \frac{\chi_w f_y h_w t}{\sqrt{3}}$$

Considering a numerical example by which $h_w = 4$ m, $a = 8$ m, $t = 20$ mm & steel S355 – which would correspond to a very slender large bridge girder web with c/c cross-beams 8 m – yields $\bar{\lambda}_w = 2,65$, $\chi_w = 0,40$ and the shear resistance $V_R = 6600$ kN.

2.2 Resistance prediction of nonlinear finite element simulations

The shear buckling resistance is simulated by means of non-linear finite element analysis taking into account material plasticity as well as initial geometrical imperfections. The model is shown in figure 1 below and shares the same geometrical parameters as defined above.

Residual stresses are not included explicitly but are assumed covered by the magnitude of the geometrical imperfections. The geometrical initial imperfections are taken affine to the first eigenmode in accordance with figure 1 below. The amplitude of the initial buckles is taken as $h_w/200 = 20$ mm.

Fig. 1: Finite element model with four-noded shells (left). Assumed initial deformations affine with first eigenmode (right).

The resistance from the nonlinear simulation as defined above is about 6900 kN. The state of deformation and plastic strains is shown in figure 2 below. In the figure, the contour limit for plastic strains is taken low so as reveal the extent of the plastic regions. The strains in the figure are membrane strains – hence plate bending strains are excluded.

Fig. 2: Resistance 6900 kN from nonlinear analysis.

Obviously the magnitude of initial imperfections is somewhat arbitrary although quite reasonable. The resulting resistance of 6900 kN deviates by some 5 % to the prediction 6600 kN by the EC 3 above. This is clearly reasonable enough.

The objective of this simulation was to demonstrate the ability to simulate shear buckling and to predict the resistance with adequate precision.

3. Shear buckling of non-uniform thickness slender web plates

Analysis is conducted for bottom portions of the web being 50 mm thick – which might be seen as an extension of the bottom flange. The portions investigated are 500 mm, 1000 mm & 1500 mm. For each configuration, a critical load analysis is conducted which allows for a code evaluation of the shear resistance. In addition to that, complete nonlinear analysis is also conducted so as to compare the resistances.

For the case of the bottom 500 mm of the web being 50 mm thick, the critical load is 4250 kN in accordance with figure 3 below. The resistance as determined by nonlinear analysis is 7650 kN. Using the critical load for subsequent code evaluation it is obtained

$$\bar{\lambda}_w = \sqrt{\frac{V_y}{V_{cr}}} = \sqrt{\frac{0,355 \cdot (3500 \cdot 20 + 500 \cdot 50)}{\sqrt{3} \cdot 4250}} = 2,14$$

$$\chi_w = \frac{1,37}{0,7 + 2,14} = 0,48$$

$$V_R = \frac{0,48 \cdot (3500 \cdot 20 + 500 \cdot 50) \cdot 0,355}{\sqrt{3}} = 9300 \text{ kN}$$

This resistance is obviously an over-estimation. Using a fictitious web area for the last step in analysis – corresponding to 4000 x 20 web – results in a resistance

$$V_R = \frac{0,48 \cdot 4000 \cdot 20 \cdot 0,355}{\sqrt{3}} = 7880 \text{ kN}$$

which is differs only 3 % from the resistance predicted by the nonlinear analysis. Increasing the bottom part of the web to 1000 mm yields a critical shear load of 5200 kN as shown if figure 4 below. Using the same analysis route as above – i.e. accounting for the enhanced critical load but considering a web area of 4000 x 20 – yields $\bar{\lambda}_w = 1,94$, $\chi_w = 0,52$ and a resistance of $V_R = 8520 \text{ kN}$. As seen in figure 4 below, the resistance from nonlinear analysis is 8540 kN and so the difference is less than 1 %.

Increasing the bottom part further to 1500 mm, the critical load is magnified to a 6740 kN as shown in figure 5. This yields $\bar{\lambda}_w = 1,70$, $\chi_w = 0,57$ and a resistance of $V_R = 9360 \text{ kN}$. As seen in figure 5 below, the resistance from nonlinear analysis is 9760 kN and so the difference is about 4 %.

Fig. 3: Bottom 500 mm of web with thickness 50 mm. Critical shear load 4250 kN (left). Resistance 7650 kN as determined by nonlinear analysis.

Fig. 4: Bottom 1000 mm of web with thickness 50 mm. Critical shear load 5200 kN (left). Resistance 8540 kN as determined by nonlinear analysis.

Fig. 5: Bottom 1500 mm of web with thickness 50 mm. Critical shear load 6740 kN (left). Resistance 9760 kN as determined by nonlinear analysis.

In all the analysis above, there is no rotational constraint of the upper flange. In a composite bridge structure, the upper flange is attached to the concrete slab via welded studs which offer a rotational constraint of the entire upper flange. Obviously the beneficial effect on resistance is an interesting matter.

For that purpose the analysis is performed by a rotational constraint of the upper flange. The resistances for i) uniform 20 mm flange ii) 500 mm iii) 1000 mm & iv) 1500 mm height of thickness 50 mm increased from i) 6900 to 7000 kN, ii) 7650 to 7750 kN, iii) 8540 kN to 8640 kN, & iv) 9760 to 10020 kN. These are moderate increases – however this is not because there is no such beneficial effect but rather due to that this effect is already accounted for by the torsionally stiff of the 1000 x 50 flange.

Reducing for that purpose the flanges to the moderate 400 x 20 results in resistances of i) 6000 kN, ii) 6600 kN, iii) 7400 kN & iv) 8070 kN, respectively. Hence, it is seen that the influence of the rotational constraints provided by a stiff flange or a composite flange may enhance the shear resistance by an amount of 15-25 %. This is certainly a factor to take into account in bridge design.

Moreover, the interaction between shear force and bending moment is important in bridge design. Considering the uniform thickness web 4000 x 20 mm with 1000 x 50 flanges, the nominal bending capacity of such a cross-section is $M = 355 \cdot 10^{-9} \cdot (1000 \cdot 50 \cdot 4000 + 4000^2 \cdot 20/6) = 89,9 \text{ MNm}$.

In figure 6 and figure 7 below, analysis is made for combinations of simultaneous bending and shear for the cross-section with uniform 20 mm thickness. The same geometrical imperfections as described above have been applied. For zero shear force, the bending resistance is about 95 % of the nominal elastic resistance due normal stress buckling of the compressed zone of the web. The figures report the bending resistance for 0, 25, 50 & 75 % of the shear resistance without bending. In addition, analysis is made also for 95 % of the shear resistance.

Fig. 6: Uniform thickness web 4000 x 20 mm. Bending moment resistance 84,4MNm with no shear force (left). Bending moment resistance 83,9 MNm at simultaneous shear force 0,25 x 6900 kN (right).

In figure 8 the corresponding analysis results are found for 0 & 75 %, respectively for the cross section with the bottom 1000 mm being 50 mm thick. Analysis is made also for 25, 50, & 95 % but not reported explicitly in figures.

The analysis results are collected into the non-dimensional interaction diagrams as given in figure 9. They are both normated to the resistance of the cross-section with uniform 20 mm web thickness. This procedure is chosen so as to clearly demonstrate the gains of the thicker bottom web strip. There is some 16 % more in the latter cross-section whereas the resistance gain is 25-30%. Moreover, there is virtually no interaction at all between shear & bending i.e. the full resistancies may be utilized simultaneously.

Fig. 7: Uniform thickness web 4000 x 20 mm. Bending moment resistance 81,2 MNm at simultaneous shear force 0,50 x 6900 kN (left). Bending moment resistance 76,3 MNm at simultaneous shear force 0,75 x 6900 kN (right).

Fig. 8: Bottom 1000 mm thickness 50 mm. Bending moment resistance 108,2 MNm with no shear force (left). Bending moment resistance 106,0 MNm at simultaneous shear force 0,75 x 8540 kN (right).

Fig. 9: Interaction curves for bending & shear. Uniform thickness web 4000 x 20 red curve. Bottom 1000 mm web thickness 50 mm blue curve. Flanges 1000 x 50 mm. The ordinate is $(V / 6900)$ and the abscissa is $(M / 89,3)$ in which 6900 & 89,3 are the shear resistance [kN] & bending resistance [MNm] for the uniform web thickness cross-section.

4. Discussions & conclusions

For very large girders, the web needs to be manufactured from two plate strips. While there is no remedy to avoid the longitudinal butt weld between the web plates, the bottom strip of the web may be design as an extension of the bottom flange. The thicker web bottom strip increases both the shear & bending resistance of the cross-section by which longitudinal trapezoidal stiffeners may be eliminated. From the studies above it may be concluded that the considered design exhibits a clear potential in bridge design. Adding some 15 % material without adding any welding improves the resistance by 25-30 %. Moreover, it enhances the interaction properties for shear & bending such that the sensitivity for simultaneous actions is reduced.

References

- [1] *Eurocode 3 – Design of Steel structures-Part 1-5: Plated structural elements*. EN 1993-1-5.