Investigation of the Eurocode 3 Method for Computing the Buckling Resistance of Plate Girders with Web Stiffeners

Christian O. Sorensen¹, Philip Eddy Eriksen²

¹Civil Engineering and Architectural Section, Institute of Technology, The Norwegian University of Life Sciences, Ås, Norway.
²Blink Hus Arkitekter AS, Fayes gate 7, P. O. Box 2300, 3103 Tønsberg, Norway.

Abstract: Tall steel beams, like plate-girders, are susceptible to local web-buckling when loaded by a point load. To prevent this, transverse stiffeners are often installed in the area below the point load to transfer the load into the girder web. The method described in Eurocode 3 NS-EN 1993-1-5 on how to compute the local web buckling resistance of a girder is investigated by executing both FEM-analyses and practical laboratory experiments with point-loaded girders without and with transverse web-stiffeners of various configurations, and comparing these results with the capacities obtained from the code-method, which implies that the point-load capacity decreases when the distance between stiffeners becomes small, i.e. approaches 20 mm. This seems unreasonable, particularly as only one single stiffener at each side of the web is commonly utilized in engineering design. The conclusion of this investigation is that the code-method is not valid for stiffener-distances below a limit.

Keywords: Steel girder; web buckling; stiffeners; point-load capacity.

I. INTRODUCTION

A plategirder is a type of beam constructed from plates of steel that are either bolted or welded together. The purpose is to obtain a beam that is larger than anything that can be built by a steel mill or factory. This type of girder is usually used to make certain types of bridges, and the girders themselves are very often in the shape of an I-beam. The size and shape of the girder allows builders to construct bridges that are much longer and heavier duty than bridges constructed with other designs. Web-stiffeners are commonly required to prevent local web-buckling at the points of application of concentrated loads. One single transverse stiffener are often sufficient at each side of the web, but a pair of stiffeners are required in other cases when the point-load is larger. The validity, for smaller stiffener spacing, of a method described in EUR NS-EN 1993-1-5:2006 (Design of Steel Structures – Plated Structural Elements) was investigated [1, 2].

The purpose of the stiffeners is to transfer the point-load through the stiffeners and into the girder-web via the vertical welds between stiffeners and web, Figure 1 and 2. This takes place through compressive stress between the underside of the girder flange and the top edge contact area of the stiffeners. This pressure is then transferred as shear stress through the stiffener to and through the stiffener/web-weldment, Figure 1.

The structural steel design code, NS-EN 1993-1-5 Section 6 – Transverse Loading Capacity, describes a computational method for stiffener configurations as shown in Figure 1. Previous preliminary computations at the University of Life Sciences has indicated that the method yields too conservative results, and more so when stiffeners are closely spaced, as the capacity than
The plate girders were continuously supported laterally at the top flange. Web buckling underneat the point of load application occurred, both in the practical lab-tests and in the FEM-models, with no internal stiffeners, Figure 3, as expected. The stiffener configurations illustrated in Figures 4 and 5 effectively prevented web buckling below the point load, and eventual failure took place as shear buckling of the web, Figure 7, in the areas adjacent to the stiffened part of the web. The stiffeners were placed too far apart in the configuration visualized in Figure 6, to prevent the web from buckling directly beneath the load, i.e. “ordinary” Euler web-buckling took place.

**II. CAPACITY VALUES OBTAINED FROM EUR NS-EN 1993-1-5:2006**

According to EUR NS-EN 1993-1-5:2006 the point load capacity with respect to web failure may be computed from

\[ F_{Rd} = \frac{f_{yw} L_{eff} t_w}{\gamma_M} \]

where

- \( f_{yw} \) is the yield strength of the web
- \( t_w \) is the web thickness
- \( \gamma_M = \) material factor
- \( L_{eff} \) is the effective length for resistance to transverse forces which should be determined from \( L_{eff} = \chi_F \ell_y \)

where \( \ell_y \) is the effective loaded length appropriate to the length of the stiff bearing, \( s \), Figure 8. The reduction factor, \( \chi_F \), should be obtained from

\[ \chi_F = \frac{0.5}{\lambda_F} \leq 1.0 \]

Where

\[ \lambda_F = \sqrt{\frac{L_{eff} f_{yw}}{F_{cr}}} \]

Effective loaded length

\[ \ell_y = s_y + 2t_f (1 - \frac{m_y}{m_f}) \]

where
m₁ = \frac{f_{yf} b_f}{f_{yw} t_w} \quad \text{and} \quad m₂ = 0.02 \left( \frac{h_w}{t_f} \right)^2 \quad \text{if } \lambda_F > 0.5
\quad \text{and} \quad m₂ = 0 \quad \text{if } \lambda_F \leq 0.5

f_{yf} = \text{yield strength of the flange}

b_f = \text{flange width}

h_w = \text{web height, i.e. distance between flanges}

F_{cr} = 0.9 F_E

\text{where}

k_F = 6 + 2 \left( \frac{h_w}{a} \right)^2

“a” being the distance between the transverse stiffeners, Figure 8.

\text{Table 1. Results computed from EUR NS-EN 1993-1-5:2006}

<table>
<thead>
<tr>
<th>TYPE</th>
<th>STIFFENER CONFIGURATION</th>
<th>a-value (mm)</th>
<th>Max.load F_{Rd} (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>No interior stiffeners, only end stiffeners</td>
<td>670</td>
<td>22.0</td>
</tr>
<tr>
<td>II</td>
<td>One interior stiffener ea. s. of web centered on point load in addition to end stiffeners</td>
<td>0</td>
<td>25.5</td>
</tr>
<tr>
<td>III</td>
<td>Two stiffeners ea. side of web, centered on point load in addition to end stiffeners</td>
<td>50</td>
<td>33.8</td>
</tr>
<tr>
<td>IV</td>
<td>Two stiffeners ea. side of web, centered on point load in addition to end stiffeners</td>
<td>182</td>
<td>42.8</td>
</tr>
</tbody>
</table>

The capacity of 25.5 kN, obtained with a single stiffener ea. s. of the web, ought to be considerably higher than the capacity of 22 kN with no interior stiffeners.

Table 1. Further, it does not seem reasonable that the capacity is greater with stiffeners @ 182 mm than when spaced @ 50 mm.

### III. LABORATORY WORK

The types of weld-fabricated beam, support, and loading configuration as in Figure 9 were utilized, with and without transverse stiffeners inserted by welding under the point load (stiffeners are not shown in Figure 9).

\text{Fig 9. Simply supported, end-fork stabilized plate-girder}

All steel: S355

Modulus of Elasticity: 210000 N/mm².

The distances between stiffeners were as given in Table 1. Refer to Figures 10 through 13 for photographs of the girders. The photo of a beam without interior stiffeners in Fig. 10 was taken subsequent to initial web buckling failure, and illustrates the test setup as well.

\text{Fig 10. Beam with no internal stiffeners, Configuration I, after web buckling}

\text{Fig 11. Stiffener configuration II}
Table 2. Results from laboratory tests

<table>
<thead>
<tr>
<th>TYPE</th>
<th>STIFFENER CONFIGURATION</th>
<th>Max.load $F_{Bd}$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>No interior stiffeners, only end stiffeners</td>
<td>38</td>
</tr>
<tr>
<td>II</td>
<td>One interior stiffener ea. s. of web centered on point load in addition to end stiffeners</td>
<td>78</td>
</tr>
<tr>
<td>III</td>
<td>Two stiffeners ea. side of web, centered on point load in addition to end stiffeners, $a = 50$ mm</td>
<td>81</td>
</tr>
<tr>
<td>IV</td>
<td>Two stiffeners ea. side of web, centered on point load in addition to end stiffeners, $a = 182$ mm</td>
<td>58</td>
</tr>
</tbody>
</table>

The test results obtained from the laboratory experiments, listed in Table 2, are in agreement with common engineering experience. One stiffener ea. s. of web beneath the point load, doubles the capacity, while two stiffeners, spaced 50 mm, ea. s. of web, yields approximately the same capacity. With stiffeners 182 mm apart, the capacity decreases to about 70% of the capacity of the beam with one stiffener ea. s. of the web or with two stiffeners @ 50 mm ea.s. of the web.

IV. FEM – ANALYSES

The same type of fine mesh was utilized in all areas of the model, Figure 14. Imperfections in shape, residual stress and eccentricities were not accounted for in this FEM-analysis. Accordingly, it was expected that the critical buckling loads would be considerably higher than the maximum loads obtained for the real beams in the laboratory experiments. The FEM-model support conditions are visualized in Figure 15. The modeling was intended to simulate the support conditions in the practical laboratory experiments. The loading block is fixed against lateral movements, but free to move vertically, which was achieved with roller supports around the edges of the block. The girder-ends are fork-supported, i.e. free to rotate laterally, and vertically, but fixed against linear horizontal movement.

Non-linear analyzes yielded critical buckling loads which occurred at initial buckling of the web when no internal stiffeners were installed and also in the case when internal stiffeners were spaced far apart, 182 mm, and corresponds to the Euler-column initial buckling load, i.e. a perfectly ideal structure. When one single stiffener pair was inserted, and also when two stiffener pairs, spaced at 50 mm, were installed, shear buckling, Fig. 7, occurred, Figures 17 and 18.
A larger spacing, 182 mm, results in a substantially reduced capacity, although still considerably higher than without interior stiffeners.

VI. CONCLUSION

The lab-test results and the FEM-analysis results are in agreement with respect to the load capacities relative to each other, and the results seems logical. The code-results are not in line with the above, and the results are rather unreasonable, based on common engineering knowledge and practice.

It may safely be concluded that the code-method is in need of revision. Limits on stiffener spacing are required, and the method should include the case with only one stiffener pair placed beneath the point load, as this is commonly used in practice.

ACKNOWLEDGEMENT

The contribution of The University of Life Sciences is acknowledged, particularly the assistance from Professor Anders Nøygård with the FEM-analysis and mechanical workshop manager Bjørn Brenna with the fabrication of numerous plate girders. Øyvind Kleven, Hjellnes Consult AS, Oslo contributed with advice at the upstart of the project. The data-assistance by Sigrun Sørensen, P.E., The Norwegian Public Roads Administration, during the finalization of this article, is appreciated.

REFERENCES

the Location of Point-Load in an I-Beam), The University of Life Sciences, Norway, 2013.

AUTHOR BIOGRAPHY


Eriksen, Philip Eddy Structural Engineer, Blink Hus Architects AS, Norway M.S., Structural Engineering, University of Life Sciences, Norway, 2014