Local buckling of plates made of high strength steel

Tapani Halme[a], Lauri Huusko[b,a], Gary Marquis[a], Timo Björk[a]
[a] Lappeenranta University of Technology,
Faculty of Technology Engineering, Lappeenranta, Finland
[b] Reijlers Oy, Hyvinkää, Finland

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Introduction

Slender plated structural elements which are subject to compressive stresses often fail due to local buckling. This failure mode is taken into account in the design of plated structures either by defining the effective width of the plate which supports the compressive forces or using the reduced stress method to estimate the ultimate load. Both these methods are presented in European steel standard EC 3's part EN 1993-1-5:2004 for plated steel structures [1]. The definition of an effective width is based on using the reduction factor ρ which takes into account the material properties, the boundary conditions, the loading and the dimensions of the structure. However, a number of tests have shown that the design rules in EN 1993-1-5 lead to overly optimistic results for the calculated ultimate loads of plated structures. In EC 3 the validity of equations for computing local buckling resistance is restricted to steel grades for which the yield strength, $f_y \leq 700$ MPa. Based on both the literature survey and the recent experimental buckling tests using an ultra high strength bainitic steel (UHSS), it is proposed that the current design equations can be extended up to at least a nominal yield strength of 1000 MPa. Additionally, a new simplified equation for the reduction factor ρ is proposed.

1 plate buckling analysis according to EC3

The effective area of the compression zone for flat compression elements is calculated using equation

$$A_{c,eff} = \rho A_c, \tag{1}$$

where A_c is the gross cross-sectional area and ρ is the reduction factor. The reduction factor for internal compression elements, e.g. web of an I -beam, may be taken as

$$\rho = 1, \text{ for } \bar{\lambda}_p \leq 0{,}673, \tag{2a}$$

$$\rho = \frac{\bar{\lambda}_p - 0.055(3+\psi)}{\bar{\lambda}_p^{\,2}}, \text{ for } \bar{\lambda}_p > 0{,}673. \tag{2b}$$

where ψ is the stress ratio which depends on the stress distribution and $\bar{\lambda}$ is the plate modified slenderness. For plates with uniform compression stress, the stress ratio $\psi = 1$ and the reduction factor for class 4 plate elements, i.e., Eq. 2b, simplifies to

$$\rho = \frac{\bar{\lambda}_p - 0.22}{\bar{\lambda}_p^{\,2}}, \text{ for } \bar{\lambda}_p > 0{,}673. \tag{3}$$

The reduction factor for compression elements with free edges, e.g., flanges of an I -beam, may be taken as

$$\rho = 1, \text{ for } \bar{\lambda}_p \leq 0{,}748, \tag{4a}$$

$$\rho = \frac{\bar{\lambda}_p - 0{,}188}{\bar{\lambda}_p^{\,2}}, \text{ for } \bar{\lambda}_p > 0{,}748. \tag{4b}$$

The modified plate slenderness is defined as

$$\bar{\lambda}_p = \sqrt{\frac{f_y}{\sigma_{cr}}} = \frac{\bar{b}/t}{28{,}4\,\varepsilon\sqrt{K_\sigma}}, \quad \varepsilon = \sqrt{\frac{235}{f_y\,[\text{N/mm}^2]}}, \tag{5}$$

where $\bar{b}$ is the appropriate width of the element depending on the boundary conditions, t is the thickness of the plate, f_y is the yield stress, σ_{cr} is the elastic critical stress and K_σ is the buckling factor based on stress ratio and the boundary conditions. The parameter ε, which takes into account the yield stress of the material, is currently valid in EC 3 only up to value 700 MPa, whereas currently produced high strength structural steels have yield stresses up to 1200 MPa.

2 literature review

The literature review concentrates on local buckling tests made for high strength steels, i.e., yield strength $f_y > 700$ MPa. Some reference studies using lower strength steels were also included. The cross-sections of the compression members in these experimental studies were either cross-shaped or box shaped, i.e., either outstanding plate elements or internal plate elements, respectively. All the tests were performed as uniaxial compression tests.

Nishino et al. [2] tested eight box beams made of ASTM A7 with yield strength $f_y = 270$ MPa and A514 with yield strength $f_y = 750$ MPa. The beams were made of four plates welded together. In the conclusion the investigators noted that a) the effect of residual stresses was more pronounced for the lower yield strength steel and b) the higher strength steel had better local buckling strength with otherwise identical test pieces. The results also show that the design rules of EC 3 give non-conservative buckling capacities for higher plate slenderness for both materials.

A series of buckling tests was performed by McDermott in 1969 for steel grade A514 [3]. Twelve beams were manufactured for local plastic buckling tests, i.e., compact cross-sections. The yield strength was analysed by coupon test as 752 MPa. The test pieces were cross-shaped, i.e., outstanding plate elements in compression. The tests confirmed the validity of EC 3 for low plate slenderness.

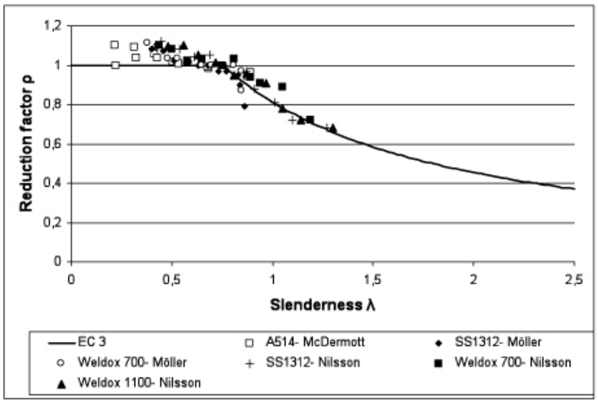

Fig. 1. Outstanding elements

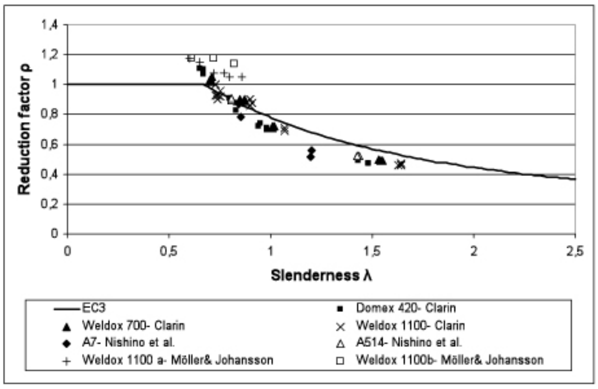

Fig. 2. Internal elements

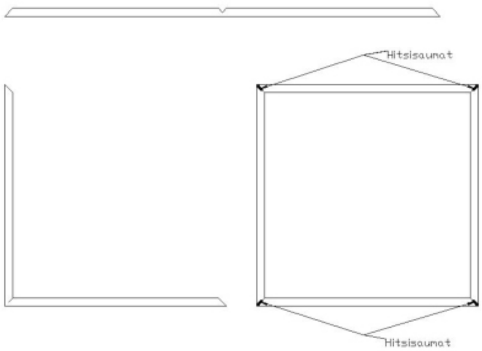

Fig. 3. Test piece manufacturing arrangement

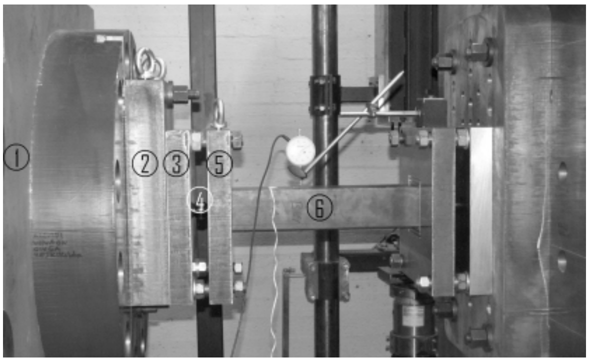

Fig. 4. Test arrangement 1) load frame 2) attachment plate 3) outer end plate 4) steel ball 5) inner end plate 6) test piece

Numerous experimental studies have been performed at Luleå University of Technology. Möller [4] performed a series of tests which investigated the effect of yield strength, residual stresses and initial deviations on local buckling of flanges using cross-shaped test pieces. The major conclusion of this study was that the yield strength is the major parameter affecting the local buckling capacity. Altogether 22 cross-shaped test pieces were used in the tests. Two materials compared were SS1312 and Weldox 700 with yield strengths f_y = 235 MPa and f_y = 700 MPa, respectively. A follow-up investigation was performed by Nilsson [5] in which 30 cross-shaped test pieces were tested. Specimens were fabricated from material SS1312, Weldox 700 and Weldox 1100 with yield strengths f_y = 235 MPa, f_y = 700 MPa and f_y = 1100 MPa, respectively. The results showed that residual stresses and geometric deviations had small effect on buckling capacity. The tests also confirmed the validity of the design rules of EC 3 for outstanding elements. Clarin [6] tested 48 box beams, i.e., local buckling of internal elements was investigated. The materials were Domex 420 (f_y = 420 MPa), Weldox 700 (fy = 700 MPa) and Weldox 1100 (f_y = 1100 MPa). A companion study was also made by Möller and Johansson [7] using box beam test specimens fabricated from two alternate Weldox 1100 -steels, with yield strengths f_y = 1130 MPa and f_y = 1349 MPa, respectively. These results showed that the reduction factor in EC 3 overestimates the buckling capacity when slenderness is lower than 0.9 whereas with higher slenderness the design rules are clearly non-conservative.

All of the previously described test results are presented graphically in Figs. 1 and 2 for outstanding and internal elements, respectively. The results clearly show that the Eurocode 3 design line gives non-conservative results for the internal elements when slenderness is higher than 0.8 – 0.9 whereas the test results for the outstanding elements are consistent with the code.

3 test results of uhs – bainitic steels

For comparison purposes the local buckling of square beam sections fabricated from ultra high strength bainitic steels with nominal yield strengths of 900 and 960 MPa was investigated at Lappeenranta University of Technology [8]. The purpose was to validate the design rules of EC3 for local buckling of slender plate elements made of steels with yield strength higher than 700 MPa. Test sections were manufactured by first milling two identical plates as shown in the upper portion of Fig. 3. These plates were then bent to form two "L" shaped sections with sharp corners. Finally, the two sections were placed in a jig and the corners were laser welded. The compression test arrangement is shown in Fig. 4.

Test pieces from plates with measured yield strength 1121 MPa *Table 1*

Test piece	B75	B84	B93	B102	B111	B120
Length l [mm]	374.50	418.00	463.00	511.00	555.00	602.00
Average width *b* [mm]	74.96	85.25	94.09	102.44	112.11	120.92
Average thickness *t* [mm]	2.98	3.03	2.98	2.97	2.99	3.06
Slenderness	0.969	1.081	1.216	1.326	1.442	1.521
Cross-section area [mm²]	998.29	1144.40	1225.85	1322.78	1448.12	1591.06
Ultimate capacity [kN]	884.1	821.0	769.5	771.1	779.2	847.2

Test pieces from plates with measured yield strength 1100 MPa *Table 2*

Test piece	B125	B140	B155	B170	B185	B200
Length l [mm]	374.00	421.00	467.00	511.00	556.00	601.00
Average width *b* [mm]	125.70	140.60	155.29	170.38	185.26	200.19
Average thickness *t* [mm]	4.98	5.00	5.02	4.99	5.01	5.00
Slenderness	0.961	1.072	1.179	1.302	1.408	1.527
Cross-section are [mm²]	2801.54	3108.54	3416.92	3695.58	4013.71	4299.12
Ultimate capacity [kN]	2465.4	2393.6	2367.9	2317.1	2340.0	2303.0

The inner end plates (5 in Fig. 4) were grooved to a depth of 2 mm corresponding to the profile geometry of the beams, i.e., welding was not used between the specimen and the inner end plates. The compression load was applied fully through the steel ball (item 4 in Fig. 4) so as to avoid secondary bending of the specimen.

In total, 12 test pieces were fabricated from two materials, two wall thicknesses and different beam widths and lengths. Geometric and measured load capacities for the test pieces are given in Tables 1 and 2.

Results are presented graphically in Fig. 5. The results confirm several earlier research studies that Eurocode gives non-conservative reduction factors, i.e., the calculated ultimate buckling loads are higher than obtained from the test results. In this figure, it has been assumed that the equation for ε, Eq. 5, is valid for the steel grades used in this investigation.

4 New reduction factor for internal elements

The test results clearly show the need for a more refined equation for the reduction factor for internal plate elements under uniaxial loading than what is given in Eq. 3. Thus a new formulation for the reduction factor is proposed in this paper. In searching a new formulation, the following criteria were set as objectives:

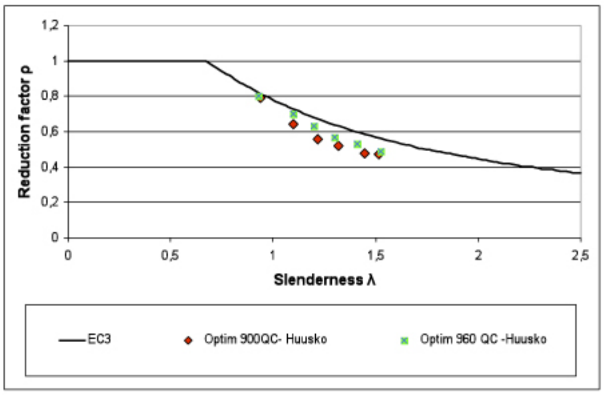

Fig. 5. Test results from UHSS bainitic steel

- Resulting consistency with test results
- Resulting slightly conservative ultimate buckling loads
- Simple expression

The following equation was chosen to replace the current reduction factor, see [9]:

$$\rho = 1,\ \text{for}\ \bar{\lambda}_p \leq 0{,}673, \qquad (6)$$

$$\rho = \frac{0.75}{\bar{\lambda}_p + 0.1},\ \text{for}\ \bar{\lambda}_p > 0{,}673.$$

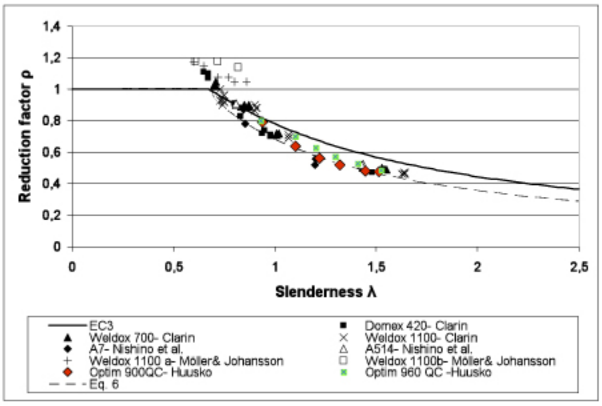

Fig. 6. New reduction factor compared to new and existing test results

Comparison between the test results and the new formulation for the local buckling reduction factor is presented in Fig. 6. In this figure, it has been assumed that Eq. 5 is valid for all steel grades reported. The resulting curve is slightly conservative when plate slenderness is between 0.673 – 0.9, but is in excellent agreement with test results on higher plate slenderness.

5 Summary and conclusions

Steel manufacturers have continued to develop and market steels with increasingly higher yield strengths. These steel grades are usually termed extra or ultra high strength steels. In many design guidance documents for steel structures, the validity of design equations does not extend to the strength levels of ultra high strength steels. In the European standard for steel structures, Eurocode 3, the validity of equations for computing local buckling resistance is restricted to steel grades for which the yield strength, $f_y \leq 700$ MPa. This paper includes a survey of published experimental local buckling data for slender high strength steel profiles. New experimental data generated at Lappeenranta University of Technology for slender plates subjected to pure compression loading is also presented. Based on both the literature survey and the recent experimental work, the current design equations can be extended at least to the nominal yield strength range of 1000 MPa. Additionally, a modification for the current effective width equation for local buckling of class 4 cross-sections in Eurocode is proposed based on the literature survey and new test results.

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For further information please contact:

Rautaruukki Corporation, Suolakivenkatu 1, FI-00810 Helsinki, Finland. Tel. +358 20 5911.
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