FLEXURAL BUCKLING AND BENDING INTERACTION OF STAINLESS STEEL MEMBERS

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Abstract: *There are several methods to cover the interaction of compression and bending for slender steel members. Usually an interaction formula is given and more recently also the general method can be used in terms of more advanced global analysis. In respect to stainless steel, the structural design standards have been developed largely based on the carbon steel standards. The current formula for interaction of axial force and bending moment given by EN 1993-1-4 was derived on limited results available and there is no limitation for the general method in the standard despite the non-linear stress-strain behaviour of stainless steels. Therefore the main aim of the paper is to show and compare numerical results for stainless steel beam-columns with existing Eurocode, the general method and some formulae proposed by other researchers taken from experimental and parametric studies. Their possible applicability will be shown as well as the way for further development. The conclusions may be used for other non-linear materials too, such as aluminium alloys, to some extent.*

Keywords: Stainless steel, Beam-column, General method, Non-linear stress-strain diagram.

1. Current methods of beam-column behavior description

The whole paper is limited to the interaction between major axis bending and axial compression of member subjected to major axis flexural buckling. So, the members are laterally and torsionally restrained along their length. Together with the Eurocode for stainless steels EN 1993-1-4 (2006), the current carbon steel code EN 1993-1-1 (2005) considering both methods (Boissonnade, Jaspart, Muzeau & Villett, 2004; Greiner & Lindner, 2006) for members subjected to both bending and compression is compared.

The main problem of stainless steel code interaction factor is the neglect of bending moment distribution along the member and material hardening. Also the boundary values of interaction factor may be limiting for members with dominant moment. So, the design approach given by EN 1993-1-4 (2006) is over conservative.

Therefore several research groups made experiments and numerical studies to develop more accurate interaction formula. All groups were focused on the accurate determination of the interaction factor:

- Salmi and Talja made experiments (Salmi & Talja, 1995; Talja, 1997) for hollow and open sections beam-columns for austenitic and duplex steel and a slight modification of ENV 1993-1-1 (1992) was suggested.
- An extensive experimental and numerical study was published by Gardner, Rossi, Young & Zhao (2015) for section interaction. It shows the effect of significant strain hardening of stainless steel for section resistance.
- The most general proposal was published by Lopes, Real & Silva (2009) that is also derived from ENV 1993-1-1 (2005).

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- Other numerical research by Greiner & Kettler (2008) was published for stocky sections only. Additionally the EN 1999-1-1 (2007) design rules for aluminium alloys are compared. It was believed that these rules developed for non-linear stress-strain diagram may show reasonable results as well.

2. Shell FE model parametric study

Firstly, a FE (finite element) model of open I sections and rectangular / square hollow sections (RHS / SHS) made in software Abaqus (used GMNIA, geometrically and materially non-linear analysis with initial imperfections) was validated. For validation were used the experiments from Salmi & Talja in Finland and the prediction of the FE model was reasonably accurate (Jandera & Syamsuddin, 2014), therefore it was accepted for subsequent parametric study.

The parametric study was made using three profiles representing different section slenderness and two section types. Three stainless steel grades (austenitic, ferritic and duplex) were considered and the material was defined by one stage Ramberg-Osgood stress-strain diagram.

The comparison between the design formulae for all selected methods and numerically established interaction factors is shown in Tab. 1.

- The EN 1993-1-4 (2006) provision showed good but most conservative results.
- The EN 1993-1-1 (2005) methods showed lower scatter but most of the results were on the unsafe side. A similar conclusion was found for ENV 1993-1-1 (1992) approach.
- Salmi & Talja (1995) expression indicated slightly better results that were on safe side. However, the results are still much conservative, the method may be perhaps the most suitable.
- The procedure used in EN 1999-1-1 (2009) for aluminium alloy structures and method published by Greiner & Kettler (2008) showed good value in average, but were very scattered for triangular and bi-triangular moment distribution.
- Lopes, Real & Silva (2009) proposal was found very unsafe, nevertheless the method with little modification (1) to (3) gives much better agreement (Fig. 1), but it may be still unsafe especially for non-uniform moment distribution. All the symbols used in the formulas are according to EN 1993-1-4.

$$k_y = 1.2 - \frac{1.2\,\mu_y N_{Ed}}{N_{b,Rd,y}} \tag{1}$$

$$\mu_y = \left(0.97\,\beta_{M,y} - 2.11\right)\bar{\lambda}_y + 0.44\,\beta_{M,y} + 0.09, \text{if } \bar{\lambda}_y \leq 0.3 \text{ then } \mu_y \leq 1.0 \text{ else } \mu_y \leq 0.9 \tag{2}$$

$$\beta_{M.y} = 1.8 - 0.7\psi \tag{3}$$

Tab. 1: Comparison between the design formulae and the numerical results

	EN 1993-1-4	EN 1993-1-1 Method 2	EN 1993-1-1 Method 1	ENV 1993-1-1	Salmi-Talja	Lopes-Real-Silva	Greiner-Kettler	Aluminium
	all cases considered							
$k_y/k_{y.FEM}$	1.235	0.969	0.925	0.997	1.129	0.828	1.053	0.966
Standard dev.	1.012	0.472	0.315	0.215	0.282	0.140	0.937	0.524
	uniform moment only							
$k_y/k_{y.FEM}$	1.015	0.885	0.878	1.001	1.162	0.834	0.872	0.845
Standard dev.	0.180	0.171	0.158	0.218	0.286	0.131	0.193	0.153

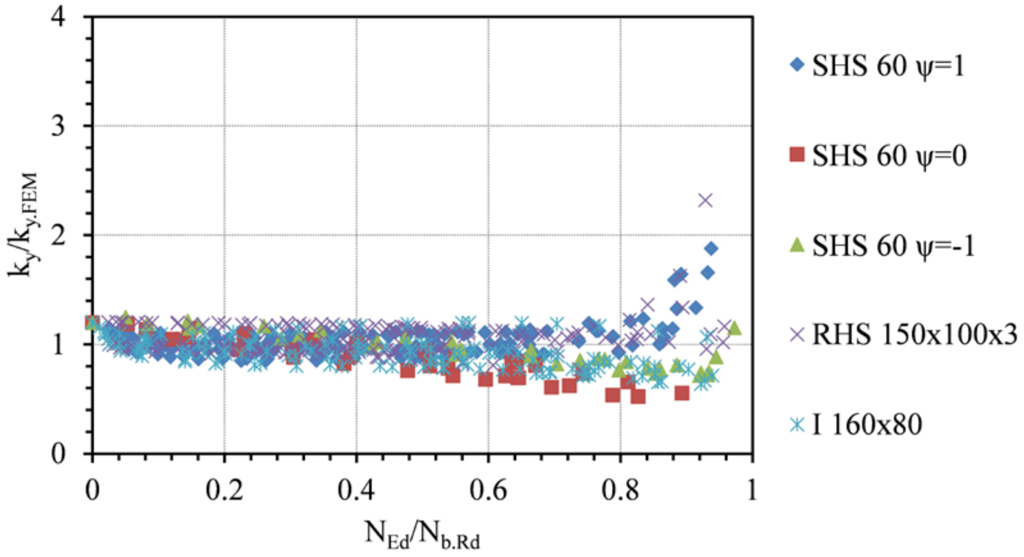

Fig. 1: Comparison for the modified method published originally by Lopes, Real & Silva

For further possible modifications of the EN 1993-1-4 (2006), more numerical results would be needed. Even more important would be to carry out more beam-column tests mostly for the case of small bending moments.

3. The general method of EN 1993-1-1

The general method is an alternative method of EN 1993-1-1 (2005) based on geometrically non-linear analysis of structure with imperfection (GNIA). The comparison was made for four members modelled in software Abaqus again. In formula the initial equivalent geometric imperfection amplitudes were calculated according to the buckling curve and for the section verification, the moment including the second order effect was used.

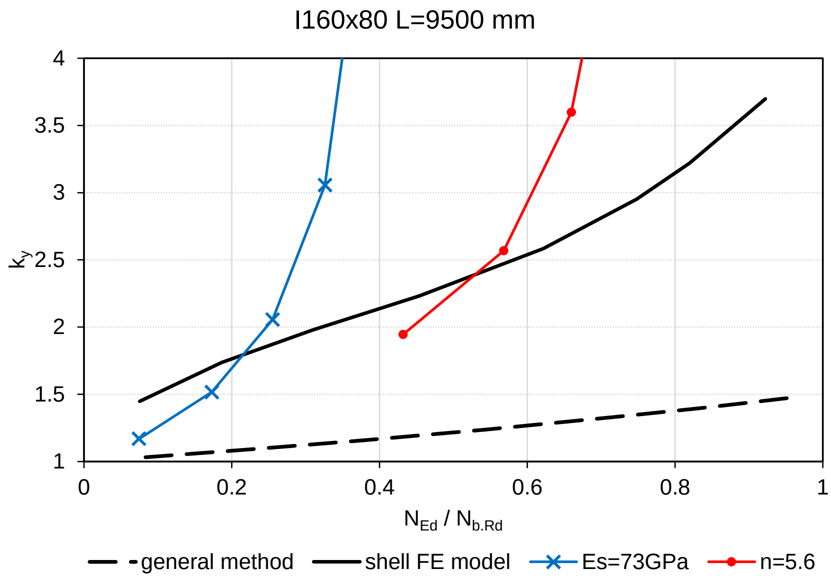

Fig. 2: Comparison of FE shell model of I 160x80 and the general method considering various Elasticity moduli

The comparison showed that the general method underestimates cases where the global buckling is almost negligible. Neglecting of the material non-linearity caused lower increase in the bending moment and is therefore generally unsafe. So, the stainless steel member is losing stiffness progressively with the load increase. For column, this is compensated by the equivalent geometric imperfection. However, it is not able to cover the effect of decreased stiffness caused e.g. by bending moment load. Therefore, the stiffness in the following beam models was considered as follows (the comparison is shown in Fig. 2):

- The secant elasticity modulus for stress level equal to the yield strength was tested.
- Non-linear stress-strain diagram was represented by the Ramberg-Osgood diagram but with hardening exponent $n = 5{,}6$.

Both modifications gave very conservative results. The secant modulus clearly underestimates the initial stage of the loading as well as the fibers and the sections which are loaded lower than the critical section. The beam model considering the non-linear diagram also underestimates the resistance. This is because the effect of the non-linearity on column buckling is once present in the buckling curve respectively in the equivalent geometric imperfections. The general method is recently not suitable for material described by the non-linear stress-strain diagram.

4. Conclusion

The main objective of this paper is to study relevant interaction factors for stainless steel beam-columns. The comparison showed that the current design code for stainless steel is over conservative, other published proposals are slightly more accurate but some of them are on the unsafe side and all of them still do not fit the real behavior of beam-columns accurately.

General method was compared too. It was shown that using of the initial Elasticity modulus underestimates the resistance of slender members. This may be compensated by reduction of the Elasticity modulus or by using the non-linear stress strain diagram (GMNIA) that is the most suitable method to be used if the equivalent geometric imperfections would be reduced.

References

EN 1993-1-4 (2006) Eurocode 3: Design of steel structures - Part 1-4: General rules -Supplementary rules for stainless steels, CEN, Brussel.

EN 1993-1-1 (2005) Eurocode 3: Design of steel structures - Part 1-1: General rules and rules for buildings, CEN, Brussels.

Boissonnade, N., Jaspart, J.P., Muzeau, J.P., Villett, M. (2004) New interaction formula for beam-columns in Eurocode 3: The French-Belgian approach. Journal of Constructional Steel Research. vol. 60, pp. 421–431.

Greiner, R., Lindner, J. (2006) Interaction formulae for members subjected to bending and axial compression in EUROCODE 3 - the Method 2 approach. Journal of Constructional Steel Research. vol. 62, pp. 757–770.

Salmi A., Talja, P. (1995) Design of stainless steel RHS beams, columns and beam-columns. Research Note 1619, VTT Building Technology, Finland.

Talja, A. (1997) Test report on welded I and CHS beams, columns and beam-columns. Report to ECSC. VTT Building Technology, Finland.

ENV 1993-1-1 (1992) Eurocode 3: Design of steel structures Part 1.1 General rules and rules for buildings, CEN, Brussels.

Gardner, L., Rossi, B., Young, B., Zhao, O. (2015) Behaviour of structural stainless steel cross-sections under combined loading – Part I: Experimental study. Engineering Structures. vol. 89, pp. 236-246.

Gardner, L., Rossi, B., Young, B., Zhao, O. (2015) Behaviour of structural stainless steel cross-sections under combined loading – Part II: Numerical modelling and design approach. Engineering Structures. vol. 89, pp. 247-259.

Lopes, N., Vila Real, P., da Silva, L.S. (2009) Stainless steel beam-columns interaction curves with and without lateral torsional buckling. 7th EUROMECH Solid Mechanics Conference, Lisbon, 17 p.

Greiner, R., Kettler, M. (2008) Interaction of bending and axial compression of stainless steel members. Journal of Constructional Steel Research, vol. 64, pp. 1217-1224.

EN 1999-1-1 (2007) Eurocode 9: Design of Aluminium structures, Part 1-1: General structural rules. CEN, Brussels.

Jandera, M., Syamsuddin, D. (2014) Interaction Formula for Stainless Steel Beam-Columns, Eurosteel 2014, Naples, pp. 145-146.