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Applicable equivalent bow imperfections in GMNIA for Eurocode buckling curves – in case of box sections

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Abstract

The flexural buckling resistance of compressed columns is typically determined using the flexural buckling curves of EN1993-1-1. With advances in computing capabilities, it has become possible to obtain the flexural buckling resistance as a result of a geometrically and materially nonlinear analysis (GMNIA) using imperfections. In design practice, equivalent geometric imperfections are usually used, but their values are calibrated against GNIA analysis. In the present research work, welded box-section columns are investigated and a proposal for relative slenderness and yield strength dependent equivalent geometric imperfection magnitudes is developed. The required equivalent bow imperfection magnitude is determined to achieve the same resistance level as the calculation according to Eurocode-based buckling curves. GMNIA analysis are conducted on simply supported columns subjected to concentrated force. The steel grade varied between S235 and S960. Several different section geometries are studied, the relative slenderness ratio is varied between 0.3 and 2.2. Design curve is fitted to the results and recommendation is given for applicable equivalent geometric imperfection magnitudes for steel welded box-section columns.

Keywords

GMNIA analysis, equivalent geometric imperfections, flexural buckling, stability, high strength steel

1 Introduction

Imperfections are of great importance in the analysis of compressed columns. In design practice, there are several ways to take imperfections into account. In the last decades, the flexural buckling curves of EN1993-1-1 [1] have been used to determine the buckling resistance of compressed columns. In this case, the imperfections (initial geometric imperfection and residual stress) are indirectly taken into account by means of the buckling curves. Nowadays, the role of manual calculation is increasingly being replaced by advanced numerical models, and the load carrying capacity is obtained as a result of geometrical and material nonlinear analysis using imperfections (GMNIA). In such cases, imperfections are directly considered in the model. Since imperfections, especially residual stresses are diverse and show a large variance, in practice equivalent geometric imperfections are usually used taking into account both the geometric imperfection and effect of residual stresses. Currently, the proposal to use equivalent bow imperfections in the EN 1993-1-1 refers to GNIA and not GMNIA analysis and therefore needs improvement for the application of GMNIA analysis.

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2 Literature review and research aim

Currently if engineers want to design a structure using GMNI analysis, the applicable equivalent imperfection is defined by Annex C of EN 1993-1-5 [2] and Table 5.1. of EN1993-1-1 [1]. However, the applicability of these imperfections does not result the same level of resistance as the resistance based on the flexural buckling curves of the EN1993-1-1 standard [1]. Walport at al. [3] examined this issue in the recent years, and provided a generally applicable imperfection magnitude, defined by eq. (1), where α is the imperfection factor for the relevant buckling curve as prescribed in EN1993-1-1 [1].

$$\frac{e_0}{L} = \frac{\alpha}{150} \tag{1}$$

The proposed equivalent imperfection is determined based on benchmark ultimate loads that are obtained from FEM using beam elements with geometric imperfections of amplitude L/1000 and residual stresses. The proposed equivalent imperfection can be used for several steel grades and stainless steel and applicable for different cross-section types. However, it was observed that there is quite large scatter in the accurate applicable equivalent imperfections depending on slenderness, steel grade and cross-section geometry. The proposed equivalent imperfection is a general (average) value that can be used safely for all the examined cases. In the current research we are intended to obtain the accurate applicable equivalent imperfection magnitude for only welded square box-section columns made of carbon steel based on the resistance of EN1993-1-1 [1] using the flexural buckling curves. It is analysed how the cross-section geometry and steel grade affect the applicable equivalent geometric imperfection that has to be applied in GMNI analysis to achieve the same resistance as can be calculated based on the flexural buckling curves of EN1993-1-1 [1]. Based on the current research findings design equation is proposed to obtain the applicable equivalent imperfection magnitude in function of the relative slenderness ratio, steel grade and cross-section geometry for steel square welded box section columns.

3 Numerical model development

3.1 Geometric model, boundary conditions

Numerical model is developed using the finite element software Ansys 19.2 [4]. The model is a full shell model using four node thin shell elements (Shell 181). The applied finite element mesh size is verified by checking the appropriateness of the mathematical and geometrical finitization. The finite element mesh used divides each side to at least 16 parts in the transverse direction, in longitudinal direction the size of the finite elements is double of their transversal size. The numerical model used in the present paper represents a simply supported box-section column subjected to concentrated force at the two ends. The developed model is shown in Fig. 1 together with the applied finite element mesh, boundary and loading conditions.

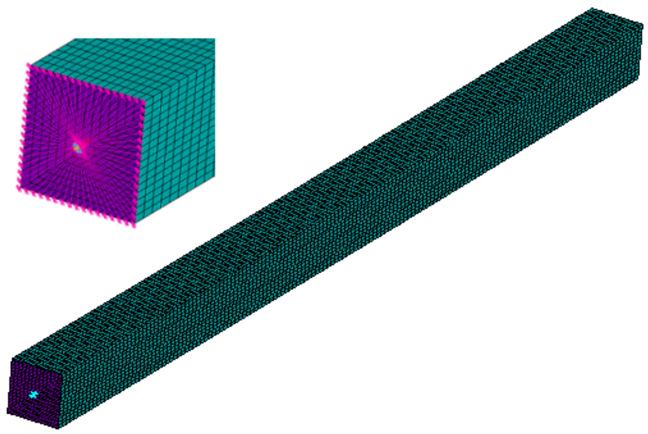

Figure 1 Finite element model (mesh, boundary and loading conditions)

3.2 Applied material model

For the nonlinear calculations, a multilinear material model is used according to Gardner at al. [5]; shown in Fig. 2. This multilinear material model can be defined from three basic parameters - elastic modulus (E), yield strength (f_y), and tensile strength (f_u).

Nowadays, the application of high strength steels (HSS) is growing in civil engineering practice due to their numerous advantages. For this reason, high strength steels have been used in addition to testing of normal strength steel (NSS) columns. In the analysis six different steel grades are used, namely: S235, S355, S460, S500, S700, S960. Originally, it is known that high-strength steels behave differently than columns made of normal strength steel; the linear behaviour of the stress-strain relationship ends before the yield strength is reached and even there is no yield plateau [6]. However, numerous recent studies show stress-strain curve of HSS steel grades can be similar to usual steel grades [7]-[10] depending on its manufacturing technique up to S700. Therefore, in the present study the material model shown in Fig. 2. is applied for all the examined steel grades.

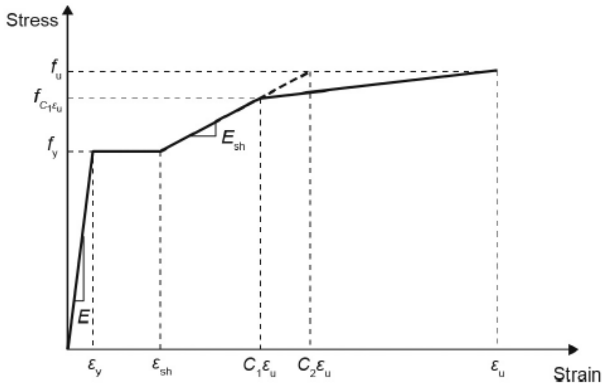

Figure 2 Multilinear material model according to Gardner et al. [3]

3.3 Applied equivalent geometrical imperfection

In the numerical model both the global and local imperfection shapes are modelled using manually defined imperfection shapes. The global imperfection shape is a half-sine wave shape. The shape of the local imperfection is a continuous sine wave on each side of the specimen. The number of half-waves is equal to the rounded down ratio L/B, where L is the length of the sample and B is the width of the cross-section. The amplitude of the global imperfection (e) is examined in present research, the global imperfection amplitude is commonly defined as the length divided by a certain constant ($L/k_{globimp}$). In the current study the $e/L = k_{globimp}$, named as imperfection amplifier is calculated for each case and the determined values are evaluated. The amplitude of the local imperfection is set to B/10000, where B is the width of the cross-section. However, the amplitude of the local imperfection did not affect considerably the results of the study, since only cross-sections belonging to cross-section class 1-3 are investigated.

3.4 Validation of numerical model

The finite element model was verified with test data. The verification process is introduced by Somodi and Kövesdi in [11].

4 Equivalent bow imperfections for each buckling curves

4.1 Effect of cross-section dimensions on the bow imperfection

Seven different welded box-section columns are studied with the steel grade of S355. The following variants are investigated: (i) the basic case 250x250x8 cross-section, (ii) the b/t ratio does not change, but b and t parameters double or halve, (iii) only b or only t parameter changed, (iv) both parameters and B/t ratio changed.

In addition, special attention is paid to testing columns having cross-section classes 1, 2 and 3. The studied cross-sections are summarised in Table 1. For each type of cross-section, 10 slenderness ratios (the relative slenderness varied between 0.3 and 2.2) are investigated by applying different span lengths and for each one the required equivalent bow imperfection is determined to obtain the same load carrying capacity value as by using the corresponding buckling curves of EN1993-1-1 [1].

Figure 3 presents the results of the study where the vertical axis shows the required equivalent bow imperfection magnitude divided by the length of the element – imperfection amplifier ($e/L = k_{globimp}$), and the horizontal axis represents the relative slenderness ratio of the analysed girders.

Table 1 Investigated cross-sections

yield strength	coefficient	width	height	thickness	B/t ratio	cross-section class
fy	ε	B	H	t	B/t	
[Mpa]	[-]	[mm]	[mm]	[mm]	[-]	
355	0.8136	125	125	4	31.25	2
355	0.8136	250	250	8	31.25	2
355	0.8136	500	500	16	31.25	2
355	0.8136	120	120	6	20.00	1
355	0.8136	250	250	12	20.83	1
355	0.8136	150	150	8	18.75	1
355	0.8136	280	280	8	35.00	3

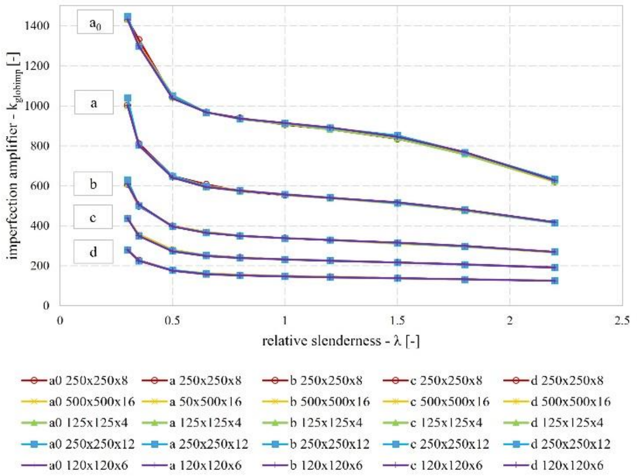

Figure 3 Imperfection amplifiers required to recover the load carrying capacity corresponding to the standard buckling curves in the function of the relative slenderness - different cross-sections

It can be seen, that for the buckling curves of a_0, a, b, c and d, the results fall approximately on a single curve regardless of the geometry of the cross-section. It is found that all the results are within 2% of the average of the different section results, therefore it can be concluded that for all square cross-sections the same imperfection amplifier $k_{globimp}$ should be applied regardless of the thickness, width and b/t ratio. The diagram shows that different imperfection amplifier curves belong to the different buckling curves. However, it is observed that the difference between the curves can be described by the imperfection factor α parameter that is defined in EN1993-1-1 standard [1]. Fig. 4 shows all the imperfection amplifier curves multiplied by the imperfection factor α for a single section size. It is found that all the previous curves are close to each other and can be described by a single function that depends only the relative slenderness.

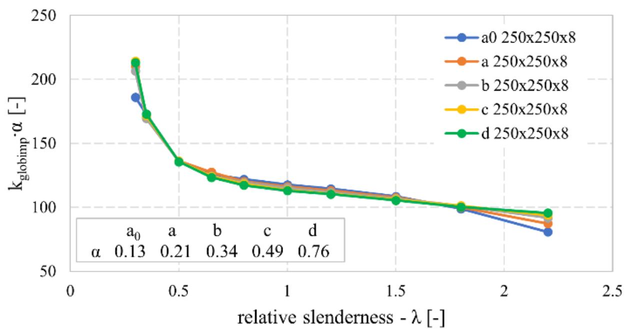

Figure 4 Imperfection amplifier curves for a single section size multiplied by the imperfection factor α

It is found that Eq. (2) is a good approximation for the numerically obtained imperfection amplifier $k_{globimp}$ values.

$$k_{globimp.S355} = (\lambda^A - B \cdot \lambda + C)/\alpha \tag{2}$$

where α=-3.64, B=18.16 and C=132 and λ is relative slenderness.

4.2 Effect of steel grade on the equivalent bow imperfection

Since the modification of the cross-section has no effect on the required equivalent bow imperfection, only 1 to 1 cross-section is tested for different steel grades. Six steel grades are studied: S235, S355, S460, S500, S700, S960, the details are given in Table 2. The geometric dimensions of the sections are chosen, all sections belong to cross-section class 3 or lower.

Table 2 Investigated cross-sections for different steel grades

yield strength	coefficient	width	height	thickness	b/t ratio	cross-section class
fy	ε	B	H	t	B/t	
[Mpa]	[-]	[mm]	[mm]	[mm]	[-]	
235	1.0000	250	250	8.	31.25	1
460	0.7148	250	250	8.	31.25	3
500	0.6856	240	240	8.	30.00	3
700	0.5794	200	200	8.	25.00	3
960	0.4948	175	175	8.	21.88	3

The results of the numerical simulations are shown in Fig. 5.

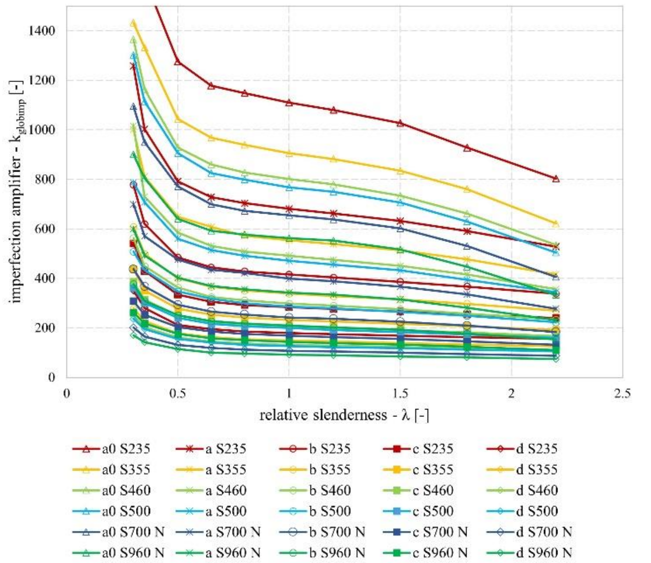

Figure 5 Imperfection amplifiers required to recover the load carrying capacity values corresponding to the standard buckling curves in the function of the relative slenderness - different steel grades

Table 3 Applicable constants to the imperfection amplifier formula for different steel grades.

	S235	S355	S460	S500	S700	S960
A	-3.90	-3.64	-3.57	-3.44	-3.32	-3.18
B	20.2	18.2	17.7	18.2	15.6	13.6
C	159	132	119	116	97.7	83.4

For different steel grades, the same type of parallel curves is obtained for each buckling curve. The curve corresponding to the required equivalent bow imperfection is drawn highest for grade S235 and lowest for grade S960. In other words, for higher material grades, a higher equivalent geometric imperfection is required to achieve the same load carrying capacity. It is found that Eq. (2) can be accurately applied for all the examined steel grades but with slightly different A, B and C constants. The applicable constants are summarized in Table 3.

Fig. 6 shows the applied imperfection amplifier functions based on Eq. (2) multiplied by the imperfection factor α for each examined steel grades (lines) and all the calculated numerical results (points), also multiplied by the imperfection factor α.

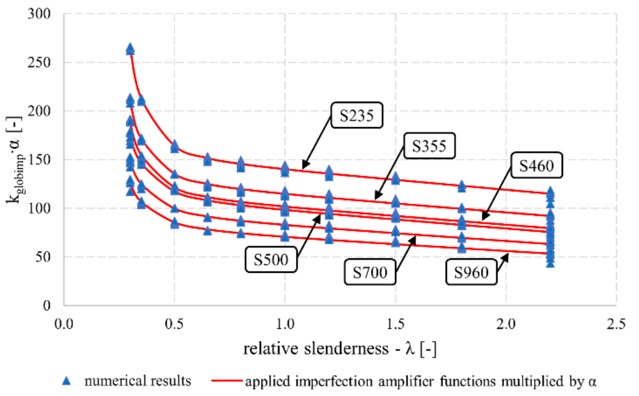

Figure 6 Imperfection amplifier curves multiplied by the imperfection factor α for different steel grades

Results show the curves presented in Fig. 6 have similar shape, and their difference can be described by the coefficient $\varepsilon=\sqrt{235/f_y}$. Figure 7 shows all the data points and curves that presented on Fig. 6, but the vertical axis now represents the imperfection amplifier $k_{globimp}$ multiplied by the imperfection factor α and divided by the coefficient ε. The results show that the imperfection amplifier $k_{globimp}$ depends linearly on the coefficient ε with a good approximation.

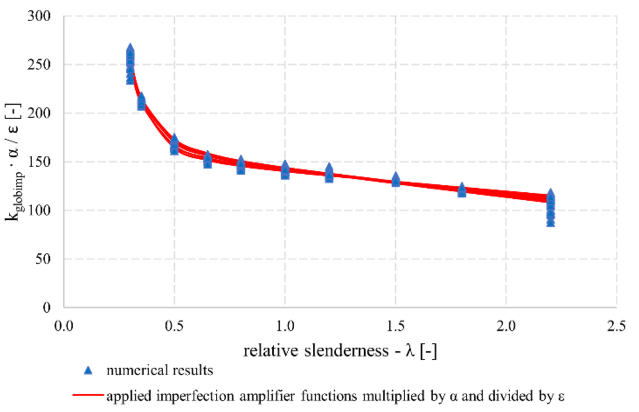

Figure 7 Imperfection amplifier curves multiplied by the imperfection factor α and divided by the coefficient ε for different steel grades

4.3 Development of general imperfection function

For practical usability, approximate function is sought which, knowing the relative slenderness and steel grade, can be used to obtain generally the required equivalent geometric imperfection magnitudes. Based on the previous findings a general function, Eq. (3) is developed to provide the necessary equivalent imperfection amplitudes in function of the slenderness, yield strength and imperfection factor α of the desired buckling curve.

$$k_{globimp} = \frac{\varepsilon}{\alpha}(\lambda^{-3.79} - 26.1 \cdot \lambda + 168) \quad (3)$$

where λ is relative slenderness
α is imperfection factor,
ε is $\sqrt{235/f_y}$

Together with the numerically calculated results (blue points) and the unique approximate functions for each steel grades (Eq. (2) – red lines), the general approximate function (Eq. (3) – black dashed lines) are plotted in function of the relative slenderness in Fig. 8.

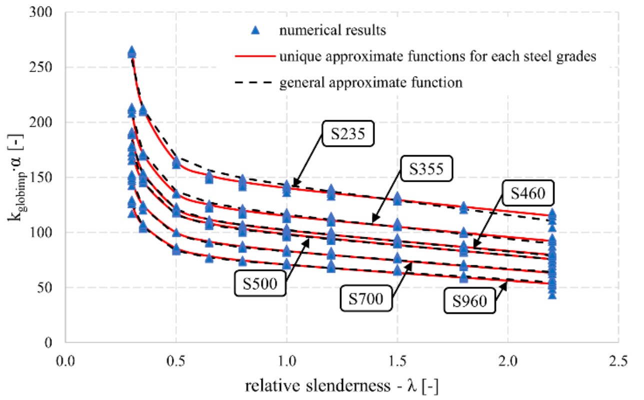

Figure 8 General and unique approximate functions for each steel grades multiplied by the imperfection factor α

4.4 Validation and error of the general approximate function

The GMNI analyses are performed using the imperfection amplifier obtained by the general approximation function for the material grades presented earlier. It is investigated to what extent the load carrying capacity obtained from the numerical calculation differs from the result of the manual calculation according to EN1993-1-1 [1].

The percentage of deviation is calculated for each case, which is always less than 2%. The total root mean square (RMS) of the deviations is 0.48%.

The approximation function gives the most accurate results for the a and b buckling curves, in which case the deviation was always less than 1%. The largest errors typically occur at a slenderness of 2.2. Excluding the corresponding results, curve c also shows only deviations below 1%, and all but S235 and S355 material grades show deviations of less than 1%. The errors by buckling curves are shown in Figures 9-10.

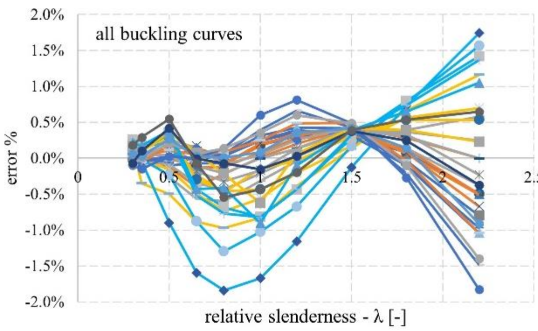

Figure 9 Errors in relation to the load carrying capacity obtained by manual calculation according to EN1993-1-1 [1] for different buckling curves -I

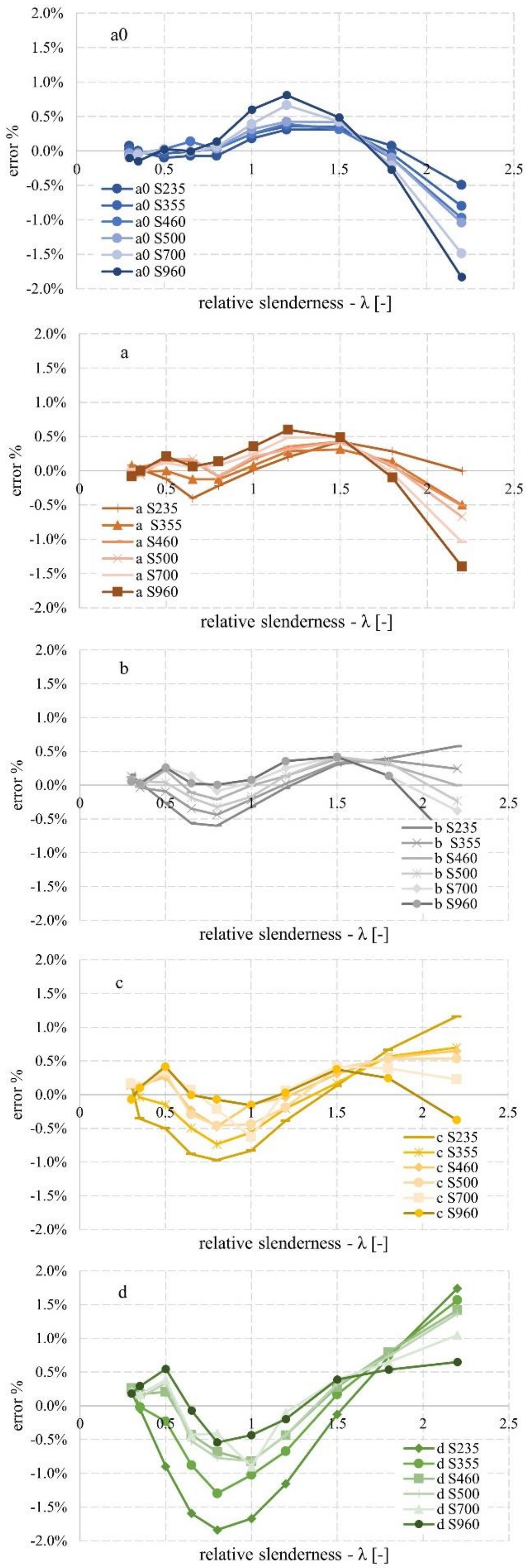

Figure 10 Errors in relation to the load carrying capacity obtained by manual calculation according to EN1993-1-1 [1] for different buckling curves – II

5 Conclusions

In this study, a numerical analysis is performed to determine the required equivalent geometric imperfection that GMNIA used to give the same flexural buckling resistance as the manual calculation using the buckling curves prescribed by EN1993-1-1 [1]. Only welded square box-section columns subjected to concentrated compressive forces are tested. Several steel grades are examined ranging from S235 up to S960.

Based on the numerical study the following conclusions are drawn:

- The dimensions and proportions of the box-section (width, thickness, B/t ratio) do not affect the required equivalent geometric imperfection.

- The magnitude of the required equivalent geometrical imperfection is determined for the five buckling curves, based on the test results of 10 columns of different slenderness per 6 steel grades, a well-fitted analytical function is given in dependence of relative slenderness and steel grade.

- Taking into account the imperfection amplifier obtained by using the general approximation function, the load carrying capacity obtained from the GMNIA results differs in all cases by less than 2% from the load carrying capacity obtained by the manual calculation according to EN1993-1-1 [1].

Acknowledgements

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References

[1] EN 1993-1-1:2005, Eurocode 3: Design of steel structures - Part 1.1: General rules and rules for buildings.

[2] EN 1993-1-5:2005, Eurocode 3: Design of steel structures - Part 1.5: Plated structural elements.

[3] Walport, F.; Gardner, L.; Nethercot, D.A. (2020) *Equivalent bow imperfections for use in design by second order inelastic analysis*, Structures **26**, 670-685.

[4] ANSYS® v19.2, Canonsburg, Pennsylvania, USA.

[5] Gardner, L.; Yun, X.; Fieber, A.; Macorini, L. (2019) *Steel Design by Advanced Analysis: Material Modeling and Strain Limits*, Engineering **5**, 243-249.

[6] Somodi, B., Kövesdi, B.; Hornyák, T. (2021) *Partial factor for local buckling of welded box sections*, Structures **30**, 440-454.

[7] Li, T.; Li, G.; Chan, S.; Wang, Y. (2016) *Behavior of Q690 high-strength steel columns: Part 1: Experimental investigation*, Journal of Constructional Steel Research **123**, 18–30.

[8] Qiang, X.; Bijlaard, F.S.K.; Kolstein, H. (2013) *Post-fire performance of very high strength steel S960*, Journal of Constructional Steel Research **80**, 235–242.

[9] Wang, J.; Afshan, S.; Schillo, N.; Theofanous, M.; Feldmann, M.; Gardner, L. (2017) *Material properties and compressive local buckling response of high strength steel square and rectangular hollow sections*, Engineering Structures **130**, 297–315.

[10] Meng, X.; Gardner, L. (2020) *Testing of hot-finished high strength steel SHS and RHS under combined compression and bending*, Thin-Walled Structures **148**, 106262

[11] Somodi, B., Kövesdi, B. (2017) *Flexural buckling resistance of welded HSS box section members*, Thin-Walled Structures **119**, 266-281.