Shear Stiffness of Closely Spaced Built-up Stainless Steel Columns

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Abstract

This paper aims at developing design recommendations for closely spaced built-up stainless steel columns, based on findings gained in recently performed research at the University of Belgrade. The research focuses on pin-ended built-up columns formed from two press-braked channel chords oriented back-to-back, addressing their flexural buckling capacity about the minor principal axis. The impact of overall and local chord slenderness, interconnection stiffness, geometric imperfections and material nonlinearity is thoroughly evaluated. In order to fully exploit their structural performance, two separate approaches for the design of built-up columns with welded or bolted interconnections are defined, including different formulas for shear stiffness.

Keywords

Stainless steel; Built-up member; Flexural buckling; Parametric study; Experiment; Design method; Shear stiffness.

1 Introduction

Using conventional cold-formed steel members with open cross-sections in built-up assemblies extends their application to large-scale structures: most frequently in light framing systems, wall bearing systems, trusses, latticed transmission towers and communication structures. If these structures are in a specific, aggressive or urban surrounding, different stainless steel alloys may be beneficially utilised owing to their excellent corrosion resistance, ease of maintenance, good toughness, high fire resistance, pleasing appearance and harmlessness to the environment. The built-up members with chords in contact or closely spaced and connected through packing plates by bolts or welds usually have a more efficient structural response under compression relative to alternative hot-rolled or welded single members at relatively the same cost. By forming symmetric built-up cross-section from individual channels or angle sections, the eccentricity between the centroid and shear centre is eliminated, which leads to higher overall member stability against torsional or torsional-flexural buckling. Besides, pronounced cross-sectional distortions, distribution of residual stresses and heat-affected zones in the vicinity of welds may be considerably minimized by the discontinuous welding process. Contrary to a solid column, the structural response of a built-up column is more complex considering the discreteness of its cross-section and absence of a solid web element. Effects of longitudinal shear, caused by the interaction between the contact areas of the individual chords, may affect the overall behaviour and reduce the flexural buckling resistance of the built-up member. The effects of shear on flexural deflection may significantly vary depending on the structural solution of interconnections and their number along the chords. In contrast to welded interconnections, the bolt-to-hole clearance in bolted interconnections usually results in a more substantial longitudinal slip between the chords and, consequently, causes additional flexibility of the built-up column. Thus, the longitudinal shear in built-up columns has to be evaluated and accounted for in the development of a design procedure.

Over the past two decades, significant attention has been paid to aspects of the potential use of stainless steel for construction purposes. Fundamental experimental work has focused on stainless steel structural elements with tubular and hollow cross-sections. The number of investigations on open stainless steel sections is far smaller and none of them address closely spaced built-up structural elements. The experimental and theoretical observations on carbon steel built-up columns serve as a basis for a better understanding of the behaviour of the equivalent columns made from stainless steel. In the middle of the last century, Bleich[1] developed a simplified analytical criterion based on an energy approach to determine the modified slenderness ratio of pin-ended battened columns. Zandonini[2] experimentally tested two series of compressed closely spaced built-up members consisting of two back-to-back channels with welded and snug-tight bolted interconnections. The end connections of all specimens were constructed by means of preloaded bolts. Astaneh et al.[3] performed an experiment on two back-to-back angles with welded, snug-tight and preloaded bolted interconnections. Using the experimental data of the aforementioned investigations[2], [3] as the basis, Zahn and Haaijer[4] recognized the impact of interconnection stiffness on the overall behaviour of closely spaced built-up columns and developed two different empirical formulations of the modified slenderness ratio for columns with snug-tight bolted interconnections and with welded or preloaded bolted interconnections. The developed empirical equations were introduced into the first edition of the AISC LRFD Specification[5]. The adopted design procedure involves modifying the general method for the design of axially compressed solid columns by replacing the modified (equivalent) overall slenderness ratio of a built-up member instead of the geometric one. Based on Bleich's work[1], Aslani and Goel[6] proposed a new analytical formula which includes a section separation ratio α to capture shear stiffness provided by interconnections, and verified it by own experimental data for welded back-to-back hot-rolled angle members. This analytical formula replaced Zahn and Haaijer's equation[4] for columns with welded or preloaded bolted interconnections in the second and third edition of the AISC LRFD Specification[5] and was also adopted in the Specification for Structural Steel Buildings ANSI/AISC 360-05[7]. Based on the updated experimental database, Sato and Uang[8] developed new simplified equations for the modified slenderness ratio by employing a K-shear factor which has different values depending on the shape of the built-up cross-section. The equations, valid for built-up columns with

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welded or preloaded bolted interconnections, are established in the design procedure of the previous[9] and latest version of the American Specification ANSI/AISC 360-16[10]. Sherman and Yura[11] emphasised that preventing longitudinal slip in the end interconnections has a beneficial impact on the overall behaviour of built-up members. They also proposed an equation for determining the shear transfer force in the end interconnections for which slip between individual chords must be prevented. The Specification ANSI/AISC 360-16[10] also accepted this observation and requires that the end connections of built-up columns must be constructed by means of welds or preloaded bolts. If the ends of the built-up column are connected by welds, the weld length should not be less than the maximum dimension of the built-up cross-section; if the ends of the built-up column are connected by bolts, their mutual longitudinal spacing should not be larger than four times the bolt diameter at a distance that is equal to 1.5 times the maximum dimension of the built-up cross-section. It should be pointed out that ANSI/AISC 360-16[10] requires that the slenderness ratio of each individual chord should not exceed 75% of the governing slenderness ratio of the built-up member.

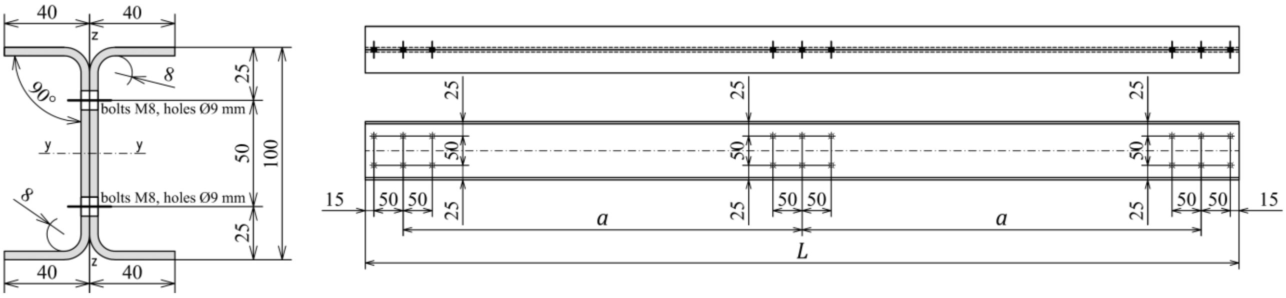
The part of Eurocode 3 for the design of stainless steel structural elements, EN 1993-1-4[12], does not provide explicit rules for determining the flexural-buckling resistance of stainless steel closely spaced built-up members. Clause 5.4.1[12] states that the design provisions for carbon steel columns given in EN 1993-1-1[13] may be applied to stainless steels columns. Eurocode 3 EN 1993-1-1[13] has a distinguishable analytical method for the design of compressed built-up members; Clause 6.4 offers a simplified design procedure that is primarily intended for uniform battened or laced built-up columns with pin-ended boundary conditions. Basically, the method replaces the discrete structure of a built-up column with an equivalent continuous column taking into account second order theory and smearing shear stiffness through properties of the bracing members. In order to restrict the influence of shear deformations or displacements between the connected chords, it is required that the number of the modules between the restraints of chords is not smaller than three. Besides, clause 6.4.4 provides the rules for closely spaced built-up members. Provided that the conditions given in Table 6.9[13], related to the maximum spacing between interconnections, are met, the closely spaced built-up member may be designed as a single member by ignoring shear deformations; otherwise, the provisions for battened members given in clause 6.4.3 should be applied. Contrary to the American Specification ANSI/AISC 360-16[10], the Eurocode 3 design approach[13] for closely spaced built-up columns does not address the influence of the interconnection shear stiffness on the column ultimate resistance. Additionally, there are no specific recommendations in terms of construction details for interconnections.

This paper aims to fill the gaps caused by the lack of research in the field of stainless steel built-up columns and at proposing new design criteria for this type of structural elements. The investigation focuses on pin-ended built-up columns formed from two press-braked channel chords oriented back-to-back in direct contact, addressing their flexural buckling capacity about the built-up axis, which corresponds to the minor principal axis. The paper presents FE (Finite Element) parametric studies based on a comprehensive experiment and FE simulation presented in detail in papers[14], [15], [16] and intended to extend the gathered experimental and numerical outcomes to a wider range of geometric variations affecting the compressive capacity of built-up columns including overall or chord failure modes. The investigation is concentrated on the most commonly used austenitic stainless steel grade EN 1.4301 (X5CrNi18-10). The FE results are used to develop two separate approaches for determining the flexural-buckling resistance of hinged supported built-up columns whose chords are directly connected by means of bolts (in EN 1993-1-8[17] denoted as shear bolt connection category A) or by welds. The design model is synchronised with rules given in parts of Eurocode 3: EN 1993-1-4[12], EN 1993-1-1[13] and based on Bleich's work[1].

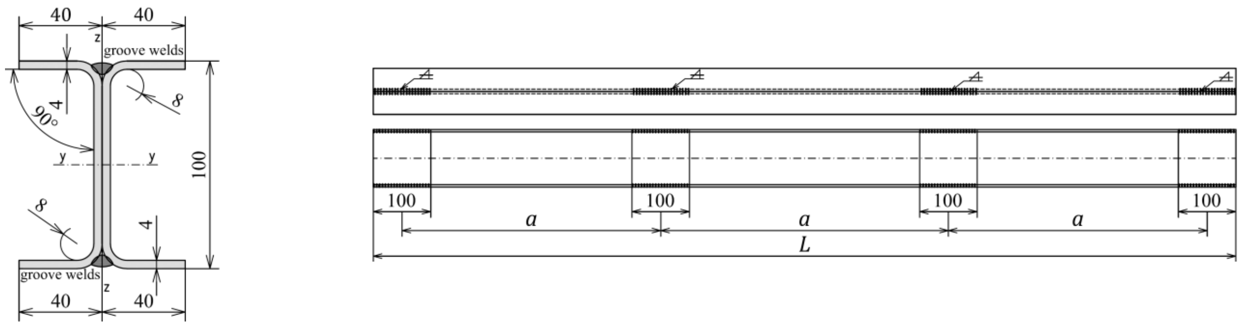
2 FE Parametric Studies

2.1 Description of influencing parameters

Extensive FE parametric studies are conducted with reference to a wide-ranging set of overall and local chord slenderness and interconnection type, to meet different performance levels of columns' structural behaviour and to establish a calculation model for the design buckling resistance $N_{b,Rd}$ of the compressed built-up member with hinged ends. A quasi-static analysis is made with the Abaqus software package[18]. The parametric studies cover the FE models of tested built-up columns that have been calibrated and validated against a flexural-buckling experiment[15], [16].



Cross-section, front view and side view of built-up columns with bolted interconnections



Cross-section, front view and side view of built-up columns with welded interconnections

Fig. 1 Nominal geometry and parameter designation of FE built-up columns

Table 1 Parameters and ranges considered in the main parametric study

Designation of the FE model	Parameters						
	Nominal length L (mm)	Overall slenderness ratio of built-up column λ = L/i	Type of connection	Number of modules between interconnections	Maximum distance between interconnections of chords a (mm)	Maximum chord slenderness ratio λch = a/imin	Ratio λch/ λ
U31b-2	500	30.7	bolt	2	185	15.3	0.50
U49b-3	800	49.2	bolt	3	225	18.7	0.38
U49b-2	800	49.2	bolt	2	335	27.8	0.57
U62b-3	1000	61.5	bolt	3	290	24.0	0.39
U62b-2	1000	61.5	bolt	2	435	36.1	0.59
U92b-3	1500	92.2	bolt	3	460	38.1	0.41
U92b-2	1500	92.2	bolt	2	685	56.8	0.62
U123b-3	2000	123.0	bolt	3	625	51.8	0.42
U123b-2	2000	123.0	bolt	2	935	77.5	0.63
U154b-4	2500	153.7	bolt	4	595	49.3	0.32
U154b-3	2500	153.7	bolt	3	790	65.5	0.43
U154b-2	2500	153.7	bolt	2	1185	98.2	0.64
U184b-5	3000	184.5	bolt	5	575	47.7	0.26
U184b-4	3000	184.5	bolt	4	720	59.7	0.32
U184b-3	3000	184.5	bolt	3	960	79.6	0.43
U184b-2	3000	184.5	bolt	2	1435	119.0	0.64
U215b-5	3500	215.2	bolt	5	675	56.0	0.26
U215b-4	3500	215.2	bolt	4	845	70.1	0.33
U215b-3	3500	215.2	bolt	3	1125	93.3	0.43
U215b-2	3500	215.2	bolt	2	1685	139.7	0.65
U246b-6	4000	246.0	bolt	6	645	53.5	0.22
U246b-5	4000	246.0	bolt	5	775	64.3	0.26
U246b-4	4000	246.0	bolt	4	970	80.4	0.33
U246b-3	4000	246.0	bolt	3	1290	107.0	0.43
U246b-2	4000	246.0	bolt	2	1935	160.4	0.65
U31w-2	500	30.7	weld	2	200	16.6	0.54
U49w-3	800	49.2	weld	3	240	19.9	0.40
U49w-2	800	49.2	weld	2	350	29.0	0.59
U62w-3	1000	61.5	weld	3	300	24.9	0.40
U62w-2	1000	61.5	weld	2	450	37.3	0.61
U92w-3	1500	92.2	weld	3	470	38.1	0.41
U92w-2	1500	92.2	weld	2	700	56.8	0.62
U123w-3	2000	123.0	weld	3	632.5	51.8	0.42

U123w-2	2000	123.0	weld	2	950	77.5	0.63
U154w-4	2500	153.7	weld	4	600	49.3	0.32
U154w-3	2500	153.7	weld	3	800	65.5	0.43
U154w-2	2500	153.7	weld	2	1200	98.2	0.64
U184w-5	3000	184.5	weld	5	580	47.7	0.26
U184w-4	3000	184.5	weld	4	725	59.7	0.32
U184w-3	3000	184.5	weld	3	970	79.6	0.43
U184w-2	3000	184.5	weld	2	1450	119.0	0.64
U215w-5	3500	215.2	weld	5	680	56.0	0.26
U215w-4	3500	215.2	weld	4	850	70.1	0.33
U215w-3	3500	215.2	weld	3	1135	93.3	0.43
U215w-2	3500	215.2	weld	2	1700	139.7	0.65
U246w-6	4000	246.0	weld	6	650	53.5	0.22
U246w-5	4000	246.0	weld	5	780	64.3	0.26
U246w-4	4000	246.0	weld	4	975	80.4	0.33
U246w-3	4000	246.0	weld	3	1300	107.0	0.43
U246w-2	4000	246.0	weld	2	1950	160.4	0.65

The CFSS (Cold-Formed Stainless Steel) built-up columns consisted of two press-braked channel chords placed back-to-back and directly and discontinuously interconnected by means of groove welds or bolts (see Fig. 1). The nominal dimensions of the one channel section were 100 x 40 x 4 mm with a nominal internal corner radius of 8 mm. The cross-section meets the compactness requirements necessary to prevent elastic local buckling and it is classified as class 3[14] according to EN 1993-1-4[12]. The geometric properties of the interconnections are uniform along the FE built-up column. The nominal length of welded interconnections is 100 mm. The bolted interconnections are designed with six M8 bolts class 8.8 in the arrangement shown in Fig. 1. The distance between end bolts in the longitudinal direction is 100 mm. The diameter of holes in the web of the cross-section is 9 mm and a 1 mm bolt-to-hole clearance is provided. The length of both interconnection types is selected to correspond to the maximum dimension of the built-up cross-section. Both ends of each FE model are flat and perpendicular to its longitudinal axis.

Parametric studies are divided into two parts: the main parametric study and the imperfection sensitivity study, in which the influence of various parameters on the column ultimate resistance is analysed. The main parametric study focuses on a wide range of overall and local chord slenderness ratios as shown in Table 1. The considered parameters, column's length L and spacing between interconnections a are also denoted on Fig. 1 for built-up members with bolted and welded interconnections, respectively. The analysed range of overall slenderness ratios from 31 to 246 may be used for different structural applications under static conditions of compressed built-up members. The spacing between interconnections is limited such that the slenderness of the individual chords does not exceed 65% of the overall built-up slenderness associated with the governing buckling mode about the minor principal axes, as in the experiment[15].The designation of the FE models in Table 1 is in accordance with the labelling system of tested specimens as explained in a previous paper[15]: the first letter indicates the shape of the chords' cross-section (U), the subsequent number indicates the overall slenderness of the column, and the final letter b or w indicates the interconnection type, i.e. weld or bolt. The number in the third position represents the number of modules between interconnections.

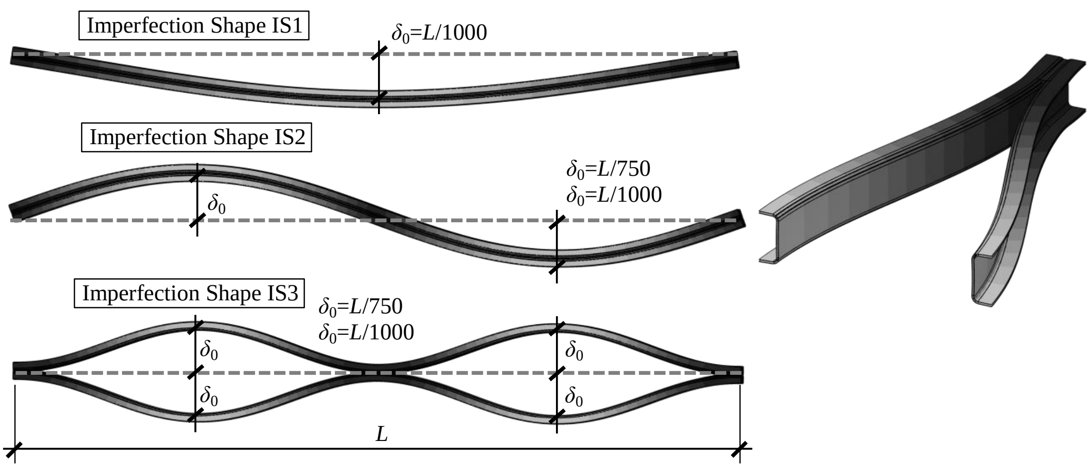


Fig. 2 Initial overall imperfections used in the sensitivity parametric study

The imperfection sensitivity study is performed in order to evaluate effects of the magnitude and shape of the initial overall imperfection pattern on potential buckling failure of individual chord members. The sensitivity study focuses on built-up columns of intermediate and high overall slenderness of 92, 184 and 246 and with interconnections at the ends and the mid-height of the column. The study encompasses the geometric imperfections of individual chords in the shape of a sine wave between interconnections in the plane perpendicular to their minor principal axis (imperfection shapes IS2 and IS3), considering variability in the amplitude of $\delta_0 = L/1000$ and of permissible fabrication tolerance of $\delta_0 = L/750$ specified in EN 1090-2[19], as depicted in Fig. 2. The structural behaviour of built-up columns, affected by imperfection shapes IS2 and IS3, is examined through a comparison with the behaviour of equivalent columns affected by the imperfection shape IS1, which is used as an input parameter in the main parametric study.

Moreover, in order to validate the FE model used for the parametric studies, it is important to incorporate the unique set of most important parameters affecting the structural behaviour of a built-up column and leading to good agreement between experimental and FE results[15],[16]: material nonlinearity, strain hardening effects, residual stresses and annealing effects in the vicinity of welded interconnections, and bolt slipping in bolted interconnections.

2.2 FE modelling

Comprehensive FE simulation of the flexural-buckling tests is depicted in detail in [16]. The mechanical properties obtained from the flat and corner longitudinal tensile coupon tests[14] are incorporated into the flat and corner parts of the press-braked chord section of FE models. In order to account for the reduction in strength properties in the vicinity of welds affected by the partial annealing of the material throughout the Heat Affected Zone (HAZ), the third material model is applied in the HAZ and welds[16], where a modified Ramberg–Osgood material model according to Arrayago et al.[20] is used to develop the stress–strain curve. Nominal values of key mechanical properties for annealed stainless steel EN 1.4301 (ASCE 304) are used according to Annex B, SE/ASCE 8-02[21]. Table 2 summarises the key material properties adopted for each of the three considered material models, including the yield strength f_y taken as the 0.2 % proof strength, the ultimate tensile strength f_u, the strain corresponding to the ultimate tensile strength ε_u and the strain hardening parameters n and m, in accordance with the two-stage Ramberg–Osgood material model [20]. Plasticity with isotropic hardening is used for all parts of the section with an initial modulus of elasticity of E = 200 GPa, and Poisson's ratio of v = 0.3. Nominal stress–strain curves are transformed to true stress–strain curves for input in the Abaqus plasticity model[18].

Table 2 Key material properties adopted in the FE models

Position	f_y (MPa)	f_u (MPa)	ε_u (%)	Strain hardening parameters	
				n	m
Flat parts	307	634	53	6.3	2.2
Corner parts	458	680	37	4.9	2.5
Welds and HAZ	207	571	64	8.3	2.0

The individual chords are modelled as S4R shell elements with reduced integration and with a size of 6 mm. The hexahedral solid elements C3D8R, 6 x 6 mm in size, are used to form the mesh of the welded interconnections. Contact conditions between the chords and the welds are defined by tie constraints at the joining surfaces. The attachment tool in the Abaqus software package[18] which involves attachment points is utilized to model the bolts in a nominal arrangement between chords. The bolts are modelled using the Cartesian mesh-independent connector type with a linear elastic stiffness of 50 000 N/mm, without consideration of rotational stiffness. The degrees of freedom of the bolt are coupled to the adjacent nodes by distributing coupling between the connector point and its corresponding surface on the chord's web. The corresponding nodes on the chords' webs within the radius of 5 mm around the reference point are kinematically constrained by means of two rigid bodies connected by a spring element. The surface-to-surface general contact interaction is selected as the modelling approach, to take into account the interactions between individual chords. The hard contact formulation of normal behaviour and the penalty friction formulation of tangential behaviour are used. The friction coefficient of 0.35 is assumed for all contact surfaces. The cross-section points at the column's ends are kinematically constrained to the central upper and lower reference points which are assigned hinged boundary conditions. Displacement control is used to apply the compressive load; a vertical displacement of U3 = 10 mm is applied to the upper reference point. The distribution of residual stresses in a fabricated austenitic stainless steel I-section, proposed by Gardner and Cruise[22], is adopted in the regions of welded interconnections. The residual stresses are incorporated into the models as initial model conditions through predefined fields. Maximum tensile residual stresses of 399 MPa are set in the vicinity of welds. The residual stresses are in self-equilibrium in the cross-section with maximum compressive residual stresses of 94 MPa. Discontinuous welding of individual chords caused variable cross-sections along the column. Thus, a stable equilibrium in the longitudinal direction is obtained in an initial analysis step prior to applying the compression load to the FE model. The residual stresses induced by the manufacturing process are not included in the FE models due to their minimal influence on the member compressive resistance[23].The FE analysis includes an eigenvalue buckling analysis and a nonlinear buckling analysis. The eigenvalue Linear Buckling Analysis (LBA) is employed in order to permit a realistic incremental nonlinear FE analysis. Superposition of initial imperfections in the form of the lowest overall buckling mode with an amplitude of $\delta_0 = L/1000$ (labelled in Fig. 2 as imperfection shape IS1) and the lowest local (cross-section) buckling mode with an amplitude of ω = 0.06 mm[14] obtained from LBA, is assigned to all FE models. A geometric and material nonlinear analysis is performed to obtain the ultimate loads and potential failure modes of cold-formed stainless steel built-up columns. The large displacements, pronounced non-linear material behaviour and complex contact conditions often lead to an inability to solve instability problems with standard implicit static numerical solvers. Hence, the FE study is performed as quasi-static using the dynamic explicit solver in the Abaqus software package[18] because it does not have the usual convergence issues.

3 Results and Discussion

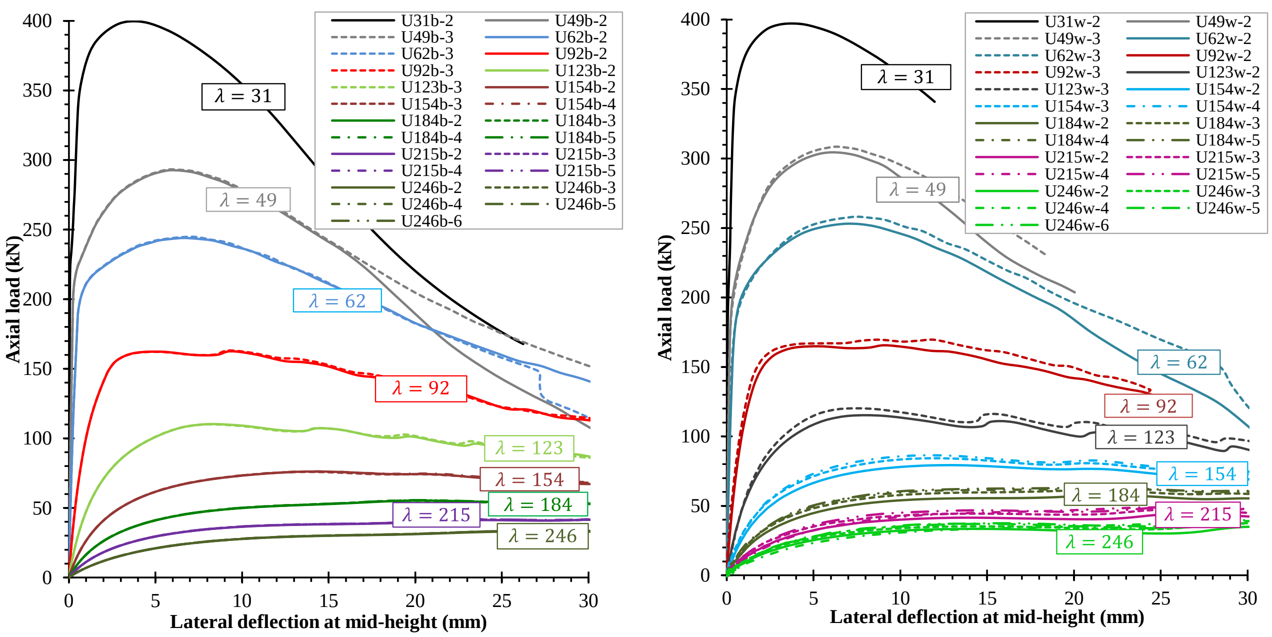
Key numerical results of the main parametric study presented in diagrammatic form as load–lateral deflection curves along the minor axis are shown in Fig. 3, both for FE built-up columns with bolted and welded interconnections. Besides, Fig. 4 compares the buckling capacities of equivalent FE built-up columns with different interconnection types, with the same overall slenderness ratio and the same number of modules between interconnections. As a result of the imperfection sensitivity study, Fig. 5 compares load–lateral deflection curves influenced by imperfection shapes IS2 and IS3 relative to basic curves influenced by imperfection shape IS1 and obtained in the main parametric study. The value of ultimate buckling load $N_{b,u}$ is also shown on the corresponding curve. A brief analysis of analysis outcomes includes the following:

1) The failure mode of each FE model is overall flexural buckling about the minor principal axis of the built-up section; the structural integrity of the built-up section is maintained in the ultimate limit state: the premature failure of the individual chord members is not identified;

2) The initial overall geometric imperfection modelled as a sine wave with an amplitude of $L/1000$ at the column's mid-height has a strong effect on the ultimate buckling resistance of built-up columns in the intermediate and high slenderness domain from λ = 123 to λ = 246. On the other hand, the residual stresses and reduction of enhanced strength properties of the material in the cross-section's corner regions in the vicinity of welds significantly affect column behaviour in the low slenderness domain up to λ = 92;

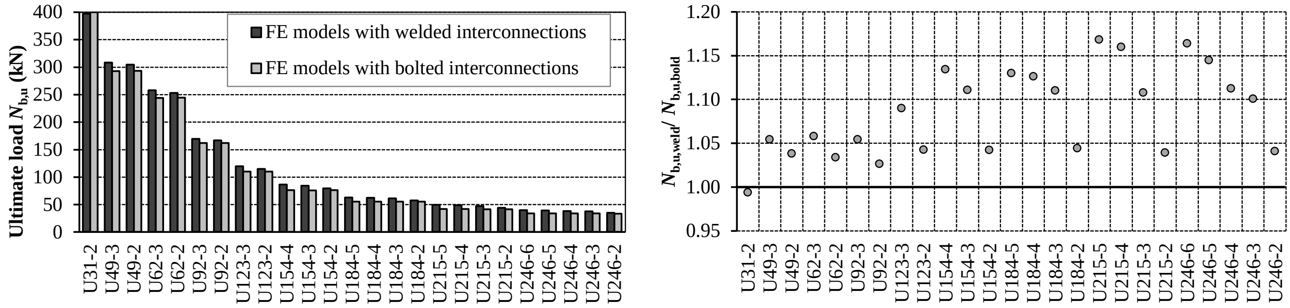
3) The FE columns with bolted interconnections of the same overall slenderness and with different chord slenderness ratios have almost identical buckling and post-buckling structural behaviour (see Fig. 3a). By increasing the number of interconnections, the ultimate buckling load of the FE column remains approximately unchanged; deviation is up to 3.6% for high slenderness λ = 215. This is due to the fact that the built-up column with bolted interconnections is less rigid and more susceptible to initial imperfections than the column with welded interconnections. It should be noted that in the experiment[15], an increase of column compressive capacity of 24.5% was recorded in the high slenderness domain (λ = 184) by changing the number of modules from two to three; however the measured geometric imperfections of tested specimens have considerably lower magnitudes and different distribution patterns compared with modelled FE geometric imperfections;

4) In contrast to the previous finding, in the case of FE columns with welded interconnections of the same overall slenderness, there is an increase of the ultimate load with an increasing number of modules between interconnections (see Fig. 3b). Even though increasing the number of interconnections from two to five increases the column strength by 16.5% in the high slenderness domain (λ = 215), the increase is limited up to 1.3% in the low slenderness domain (λ = 49) by the detrimental effects caused by the welding process. In addition, for the variation of the number of modules from two to three in the high slenderness domain (λ = 184), the rise of column buckling resistance is 10.2% in the experiment[15], whereas in the main parametric study it amounts to only 5.8%;



(a) Built-up columns with bolted interconnections (b) Built-up columns with welded interconnections

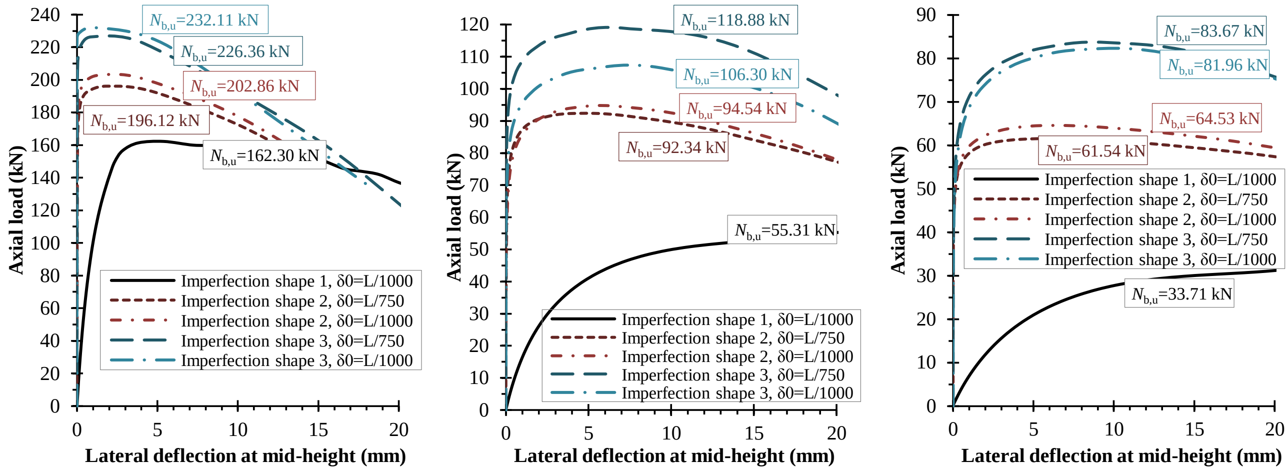
Fig. 3 Load–lateral deflection curves at mid height of FE models – main parametric study



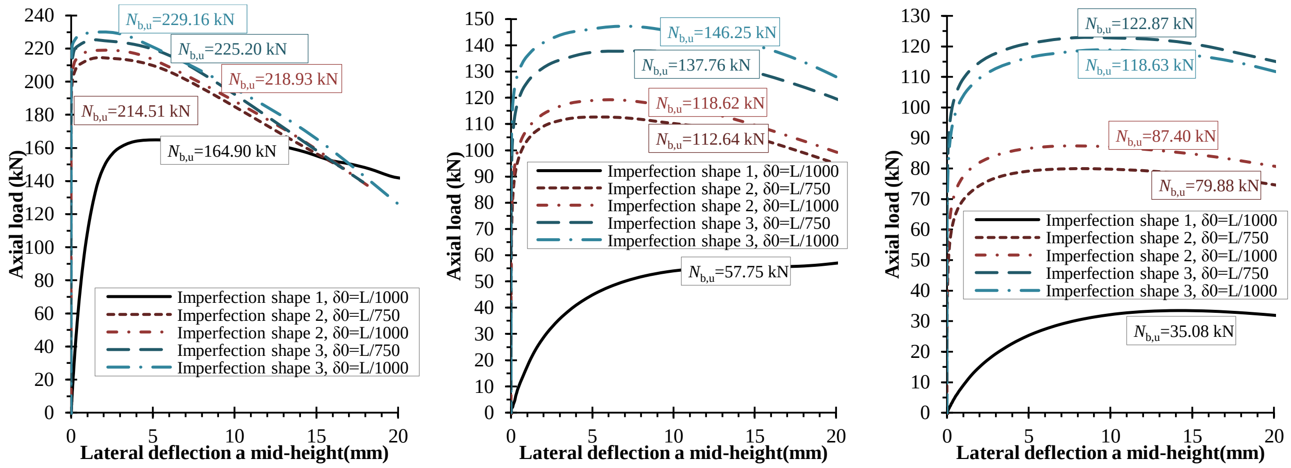
(a) Ultimate buckling loads of FE columns (b) Ratios between ultimate buckling loads of welded and bolted FE columns

Fig. 4 Load–lateral deflection curves at mid height of FE models – main parametric study

5) As indicated in Fig. 4, the FE built-up column with welded interconnections exhibits better structural response than that with bolted interconnections practically over the entire slenderness domain, except for λ = 31. This finding is strongly influenced by the higher shear stiffness of welded interconnections compared with bolted interconnections. The deteriorated structural response of the welded column with slenderness λ = 31 is associated with harmful impacts of residual stresses and partial annealing in the HAZ. In the case of columns with interconnections at their ends and mid-height, the ratio of welded column resistance-to-bolted column resistance ($N_{b,u,weld}/N_{b,u,bold}$) is almost constant and amounts to approximately 1.04 in the slenderness range λ = 123–246, and approximately 1.03 in the slenderness range λ = 49–92. Decreasing of the chord slenderness ratio in the overall slenderness range λ = 123–246 results in a gradual growth of compressive capacity; greater synergy of individual chord interaction within the welded built-up section leads to a more favourable buckling response. For the maximum number of modules, used in the range of high slenderness λ = 215–246, the buckling resistance of the columns with welded interconnections is approximately 16.8% higher relative to the equivalent columns with bolted interconnections. However, it can be noticed that the most slender columns (λ = 246) are less sensitive to the benefits of the higher stiffness of the welded interconnections; the aforementioned maximum rise of column strength is achieved herein by the higher number of interconnections between individual chords compared with columns with the slenderness λ = 215;



(a) Bolted built-up column U92b-2 (b) Bolted built-up column U184b-2 (c) Bolted built-up column U246b-2



(d) Welded built-up column U92w-2 (e) Welded built-up column U184w-2 (f) Welded built-up column U246w-2

Fig. 5 Load–lateral deflection curves at mid height of FE models – imperfection sensitivity study

6) The shape and amplitude of the initial overall geometric imperfections are crucial predictors of the critical failure mode, because their changes significantly affect the buckling response of a built-up column (see Fig. 5) As expected, the compressed built-up members are most sensitive to the sine wave (bow) shape of initial geometric imperfections with an amplitude of $L/1000$ at columns' mid-height (labelled as IS1). The distribution and magnitude of initial imperfections of individual chords, represented as a sine wave between interconnections (denoted as IS2 and IS3), do not contribute to the premature failure of individual chords. Furthermore, these imperfection shapes ensure higher initial stiffness and compressive capacity of built-up columns and may lead to an inelastic buckling response in the intermediate slenderness range. The effects of imperfection amplitude are most dominant in combination with imperfection shape IS3 both for welded and bolted built-up columns with slenderness λ = 184. As indicated in Fig. 5b, Fig. 5c and Fig. 5f, when considering imperfection shape IS3, the built-up column acts as a more stable system for a higher amplitude of $\delta_0 = L/750$ rather than for a lower amplitude of $\delta_0 = L/1000$. Additionally, it should be noticed that the columns with welded interconnections have a much greater effectiveness in the high slenderness domain both for λ = 184 and 246 than the equivalent bolted columns by considering the influence of imperfection shape IS3, while their ultimate strengths are almost equal in the intermediate slenderness domain λ = 92. Quantification of column strength growth by changing the shape and amplitude of the geometric imperfection, presented in Table 3, is provided through comparisons of ultimate buckling loads of built-up columns affected by imperfection shapes IS2 and IS3 and amplitudes $\delta_0 = L/1000$ and $\delta_0 = L/750$ with ultimate buckling resistances of built-up columns affected by imperfection shape IS1 and an amplitude of $\delta_0 = L/1000$. As indicated in Table 3, the increase of ultimate buckling loads varies significantly from 92% to 250% for imperfections shape IS3 and from 67% to 128% for IS2 in the high slenderness domain, while the increase of ultimate loads in the intermediate slenderness range is lower: from 37% to 43% for IS3 and from 21% to 33% for IS2.

Table 3 Quantification of the increase in ultimate buckling loads by changing geometric imperfections

Column	Amplitude	$N_{b,u}^{IS3,\delta_0}/N_{b,u}^{IS1,L/1000}$	$N_{b,u}^{IS2,\delta_0}/N_{b,u}^{IS1,L/1000}$
		Imperfection shape IS3	Imperfection shape IS2
U92b-2	δ_0 = L/1000	1.43	1.25
	δ_0 = L/750	1.39	1.21
U92w-2	δ_0 = L/1000	1.39	1.33
	δ_0 = L/750	1.37	1.30
U184b-2	δ_0 = L/1000	1.92	1.71
	δ_0 = L/750	2.15	1.67
U184w-2	δ_0 = L/1000	2.53	2.05
	δ_0 = L/750	2.39	1.95
U246b-2	δ_0 = L/1000	2.43	1.91
	δ_0 = L/750	2.48	1.83
U246w-2	δ_0 = L/1000	3.38	2.49
	δ_0 = L/750	3.50	2.28

4 Design Proposal

The development of the method leading to the establishment of design resistance expressions for CFSS closely spaced built-up members under compression based on the experiment[15] and the results of main parametric study obtained herein, are presented in Section 4.2. The proposed design procedure focuses on built-up columns formed from two press-braked channel chords oriented back-to-to back that are in direct contact. The basic material is austenitic alloy of stainless steel grade EN 1.4301.

4.1 Analytical criterions for the design of built-up columns

Using the energy method, Bleich (1952)[1] provided analytical solutions for elastic flexural buckling of simply supported latticed and battened built-up columns. The solutions are based on the condition that the strain energy due to deflection is equal to the work done by the external axial compression load, indicating the transition from the stable configuration to the unstable form of the elastic system. In the case of battened columns, the elastic strain energy consists of the energy due to overall bending of a built-up member, energy due to the local bending of individual chords and the energy due to the local bending of the bracing elements. Solving the energy condition[1] results in the critical buckling load of battened built-up columns $N_{\mathrm{cr,V}}$:

$$N_{\mathrm{cr,V}} = \frac{\pi^2 EI}{(kL)^2} = \frac{\pi^2 EI}{\left(1 + \frac{\pi^2 I_0}{24 I_{\mathrm{ch}}}\left(\frac{a}{L}\right)^2 + \frac{\pi^2 E I_0}{L^2}\frac{a h_0}{12 E I_{\mathrm{b}}}\right) L^2} \tag{1}$$

where k is the buckling length factor for battened built-up columns, given by Eq. (2):

$$k = \sqrt{1 + \frac{\pi^2 I_0}{24 I_{\mathrm{ch}}}\left(\frac{a}{L}\right)^2 + \frac{\pi^2 E I_0}{L^2}\frac{a h_0}{12 E I_{\mathrm{b}}}} \tag{2}$$

The buckling length factor k accounts for detrimental shear distortion effects caused by amplification of overall lateral deflections of the column and additional deflections of the column segments between battens. Equation (1) can also be written as:

$$N_{\mathrm{cr,V}} = \frac{1}{\frac{L^2}{\pi^2 EI} + \frac{a^2}{24 E I_{\mathrm{ch}}}\left[\frac{I_0}{I} + \frac{2 I_{\mathrm{ch}} h_0}{I_{\mathrm{b}} a}\frac{I_0}{I}\right]} \tag{3}$$

In foregoing stated equations, L is the column length, a is the distance between mid-points of interconnections, h_0 is the distance between the chord centroids, A_{ch} is the cross-sectional area of one chord, I_{ch} is the second moment of area of a single chord about the minor principal axis parallel to the axis of buckling, I_0 is the second moment of area of the built-up section about the buckling axis (neglecting the second moment of area of individual chords about their own minor principal axis), I_{b} is the in-plane second moment of area of one-batten members and I is the total second moment of area of a built-up member with respect to the principal axis perpendicular to the plane of buckling. By introducing the following notations for critical force N_{cr} and shear stiffness S_{v},

$$N_{\mathrm{cr}} = \frac{\pi^2 EI}{L^2} \tag{4}$$

$$S_{\mathrm{V}} = \frac{24 E I_{\mathrm{ch}}}{a^2\left[\frac{I_0}{I} + \frac{2 I_{\mathrm{ch}} h_0}{I_{\mathrm{b}} a}\frac{I_0}{I}\right]} \tag{5}$$

Eq. (3) can be reformulated as follows:

$$N_{\mathrm{cr,V}} = \frac{1}{\frac{1}{N_{\mathrm{cr}}} + \frac{1}{S_{\mathrm{V}}}} \tag{6}$$

In order to simplify Eq. (5), Bleich[1] neglected the influence of the second moment of area of individual chords I_{ch} with regard to the term $I_0 = 2A_{\mathrm{ch}}(h_0/2)^2$ when calculating the total second moment of area of a built-up column I, by approximating the ratio $1_0/1$ as equal to unity. This leads to

$$S_{\mathrm{V}} = \frac{24 E I_{\mathrm{ch}}}{a^2\left[1 + \frac{2 I_{\mathrm{ch}} h_0}{I_{\mathrm{b}} a}\right]} \tag{7}$$

However, the outcomes gained in the investigation of Aslani and Goel[6] show that Bleich's simplified approximation, given by Eq. (7), may result in errors in the prediction of buckling resistance, particularly in the case of battened columns with a relatively small distance between individual chords or closely spaced built-up columns. On the other hand, based on the experimental data of Zandonini's research[2], Zahn and Haaijer[4] demonstrated that built-up columns with snug-tight bolted interconnections are more susceptible to shear deformations. The Eurocode 3 design procedure takes into account these facts: Eq. (7) corresponds to the expression on the left-hand side of the conditional equation for shear stiffness of a battened column, defined in Clause 6.4.3 of EN 1993-1-1[13], which is given here as follows:

$$S_{\mathrm{V}} = \frac{24EI_{\mathrm{ch}}}{a^2\left[1+\frac{2I_{\mathrm{ch}}h_0}{I_{\mathrm{b}}a}\right]} \leq \frac{2\pi^2 EI_{\mathrm{ch}}}{a^2} \tag{8}$$

Expressions for shear stiffness S_{v} given by Eqs. (5), (7) and (8) take into account the flexural stiffness of the individual chords and battened members that is strongly associated with overall shear deformations.

The expression for critical load N_{cr} given by Eq. (4) takes into account the flexural stiffness of the built-up column with a stiff bracing system that is strongly associated with overall bending deformations. The total second moment of area of the built-up member I in Eq. (4) should be taken as:

$$I = 0.5{h_0}^2 A_{\mathrm{ch}} + 2I_{\mathrm{ch}} \tag{9}$$

It should be noted that Eq. (4) deviates from the expression for effective critical load $N_{\mathrm{cr,eff}}$ stated in clause 6.4.1 of EN 1993-1-1[13] given herein by Eq. (10), in terms of the second moment of area of the battened built-up column:

$$N_{\mathrm{cr,eff}} = \frac{\pi^2 EI_{\mathrm{eff}}}{L^2} \tag{10}$$

where:

$$I_{\mathrm{eff}} = 0.5{h_0}^2 A_{\mathrm{ch}} + 2\mu I_{\mathrm{ch}} \tag{11}$$

In Eq. (11), I_{eff} is the effective second moment of area of a battened built-up member and μ is the efficiency factor which is contained in the above stated formula representing the contribution of the chords' moments of inertia to the overall bending stiffness of the battened column. The efficiency factor μ ranges between zero and unity and depends on the overall slenderness of the built-up column.

4.2 Design method

The proposed procedure for the design of closely spaced built-up CFSS columns modifies the general method for the design of axially compressed stainless steel conventional (solid) columns stated in Clause 5.4.2, EN 1993-1-4[12]. The procedure introduces an empirical equation for the equivalent (modified) non-dimensional slenderness ratio of a built-up member $\bar{\lambda}_{\mathrm{eq}}$ instead of the geometric non-dimensional slenderness ratio of a solid member $\bar{\lambda}$, to reflect influences of shear deformations on the column strength. The analytic buckling curve is based on the Perry-Robertson equations and the linear expression for the imperfection parameter $\eta = \alpha(\bar{\lambda} - \bar{\lambda}_0)$. The influences of geometric imperfections, residual stresses and load eccentricity on the predicted flexural-buckling resistance is implicitly accounted for by employing an imperfection factor α associated with the appropriate buckling curve depending on the cross-section shape and manufacturing process. Two curves are specified in the existing EN 1993-1-4[12] for flexural buckling: for cold-formed sections ($\alpha = 0.49$, $\bar{\lambda}_0 = 0.4$) and for welded sections ($\alpha = 0.76$, $\bar{\lambda}_0 = 0.2$). However, by following research findings conducted over the last decade, the just-finished fourth edition of the Design Manual for Structural Stainless Steel[24] revises the buckling curves and adopts the conservative curve *D* for cold-formed channel sections made from austenitic stainless steel. Hence, considering basic material and type of chord section, the imperfection factor $\alpha = 0.76$ in conjunction with a non-dimensional limiting slenderness $\bar{\lambda}_0 = 0.2$ is employed herein both for welded and bolted CFSS built-up members. Several minor modifications of the design procedure stated in EN 1993-1-1[13] are made for the purpose of its applicability to a buckling check of closely spaced and directly interconnected CFSS built-up columns:

1) The expression for critical buckling load $N_{\mathrm{cr,V}}$ given by Eq. (6) is utilised;

2) The efficiency factor μ is set equal to unity when calculating the effective second moments of area I_{eff} in Eq. (11). Hence, Eqs. (4) and (9) are used in the calculation;

3) The second term within the denominator brackets is excluded from the expression for shear stiffness S_{v} in Eq. (8) because of the absence of the battens within the built-up cross-section with chords in contact. However, in order to satisfy the condition in Eq. (8) the expression on the right-hand side of this equation should be used:

$$S_{\mathrm{V}} = \frac{2\pi^2 EI_{\mathrm{ch}}}{a^2} \tag{12}$$

Eq. (12) is intended to predict the flexural-buckling resistance of CFSS closely spaced built-up columns with bolted interconnections;

4) Using key findings from Aslani and Goel[6], Bleich's exact solution given by Eq. (5) is employed in an attempt to introduce the beneficial impact of shear stiffness of welded interconnections in design procedure. However, the second term within the denominator brackets in Eq. (5) should be excluded:

$$S_{\mathrm{V}} = \frac{24EI_{\mathrm{ch}}}{a^2}\frac{I}{I_0} \tag{13}$$

Thus Eq. (13) is used to predict the flexural-buckling resistance of CFSS closely spaced built-up columns with welded interconnections.

The flowchart in Fig. 6 gives an overview of the proposed design method.

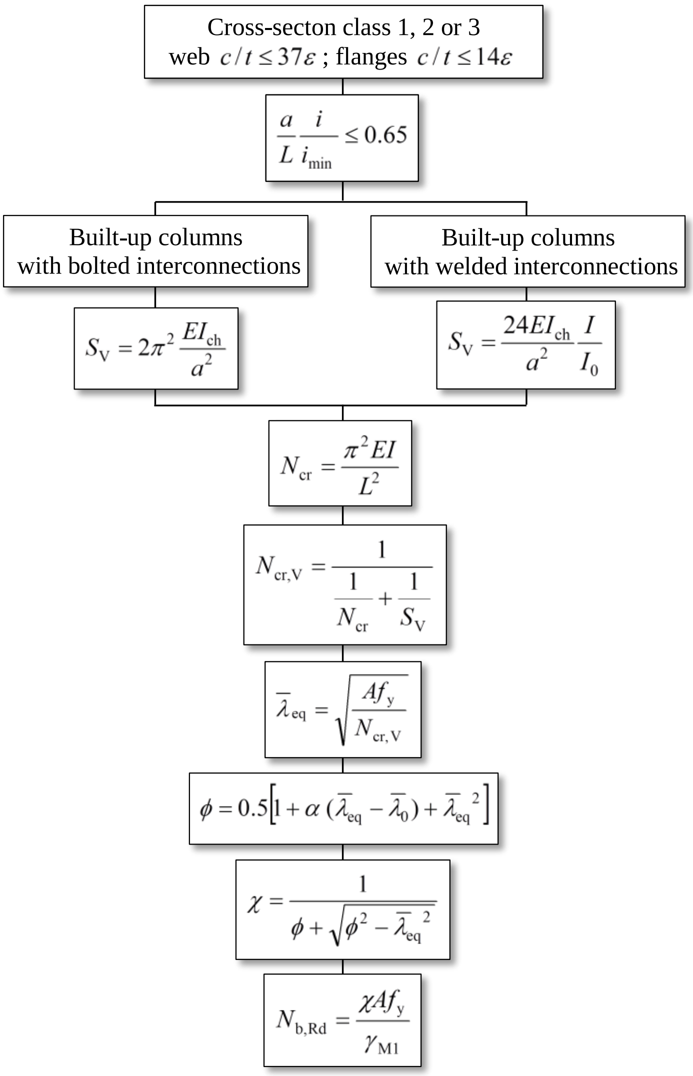


Fig. 6 Design method applicable for buckling design checks of CFSS closely spaced built-up columns

4.3 Rules

The procedure covers the following conditions:

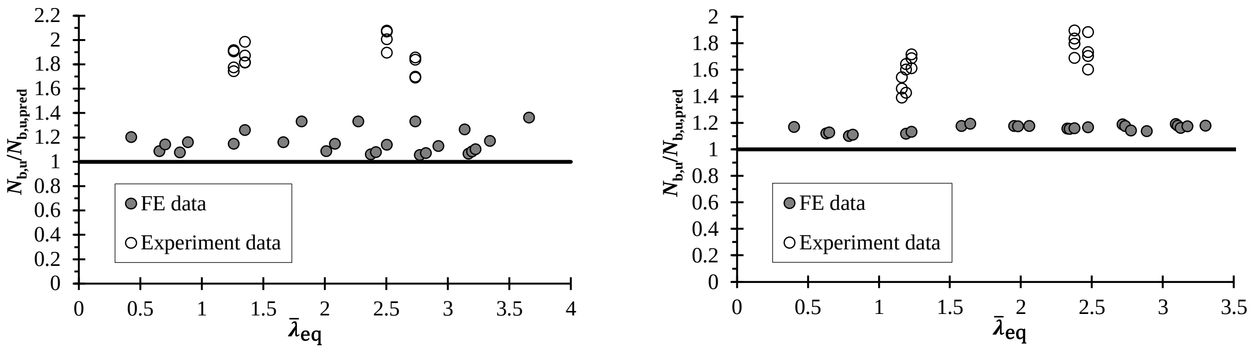
- the cross-section is cold-formed;
- the cross-section is semi-compact or compact, classified as Class 3, 2 or 1;
- the individual chords are interconnected by means of bolts or by welds;
- bolted interconnections should be designed as Category A: bearing type in accordance with EN 1993-1-8[17];
- the distance between end bolts in the longitudinal direction (in a line in the direction of load transfer) is equal to the maximum dimension of the built-up cross-section; the bolts are positioned on the chords' webs in an arrangement that meets requirements specified by EN 1993-1-8[17]: the internal spacing between centres of bolts in both

directionns is $3d_0$, in the case of end interconnections, the end distances from the centre of a bolt hole to the adjacent end of a chord's web is $2d_0$, where d_0 is the hole diameter for the bolt;

- the length-welded interconnection corresponds to the maximum dimension of the built-up cross-section; the welds are placed in the contact regions between both chords' flanges;
- the properties of interconnections are uniform along the column's length;
- the distances between mid-points of interconnections a are uniform along the column's length;
- the spacing between interconnections is limited such that the slenderness of the individual chords does not exceed 65% of the overall built-up slenderness about the axis of the built-up cross-section that corresponds to the minor principal axis; the chord slenderness ratio is based on the distance between interconnections a and a minimum radius of gyration of individual chords i_{min}.

4.4 Accuracy assessment of proposed design method

In order to assess the accuracy of the proposed design method, the comparative analysis is performed in which predicted buckling resistances of built-up columns are compared with generated experimental[15] and numerical buckling resistances. In design calculation, the yield strength f_y = 307 MPa, obtained in tensile coupon test of flat sheet material [14], and the partial safety factor γ_{M1} = 1.0 are used. The comparisons are graphically presented in Fig. 7 and a summary of the obtained results is reported in Table 4.



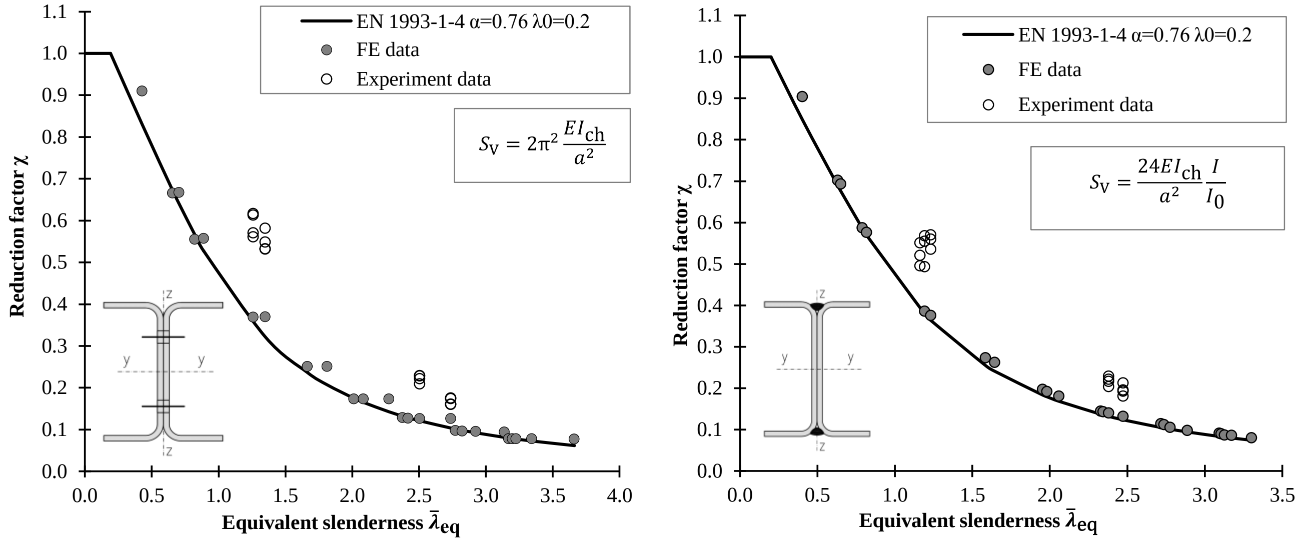
(a) Built-up columns with bolted interconnection (b) Built-up columns with welded interconnection

Fig. 7 Comparison between design resistance predictions and experimental and FE results

The significant distinctions between experimental and FE ultimate-to-predicted buckling load ratios are strongly associated with a discrepancy in the distribution and magnitude of initial geometric imperfections of tested specimens in experiment[15] and FE columns in the main parametric study, respectively. The mean experiment-to-predicted buckling load ratio $N_{b,u,exp}/N_{b,u,pred}$ is 1.87 and the Coefficient of Variation (CoV) is 6.3% for the columns with bolted interconnections, while the mean value of $N_{b,exp}/N_{b,u,pred}$ is 1.66 and CoV is 9.0% for the columns with welded interconnections. In the case of FE data, the mean numerical-to-predicted buckling load ratio $N_{b,u,FE}/N_{b,u,pred}$ is 1.16 and the CoV is 8.3% for the columns with bolted interconnections, while the mean value of $N_{b,u,FE}/N_{b,u,pred}$ is 1.16 and the CoV is 2.3% for the columns with welded interconnections. Considering both experimental and FE results, the mean value of the $N_{b,u}/N_{b,u,pred}$ ratio is 1.37 and the CoV is 30% for columns with bolted interconnections, while the mean value of the $N_{b,u}/N_{b,u,pred}$ ratio is 1.35 and the CoV is 20% for the columns with welded interconnections.

Table 4 Comparison between design resistance predictions and experimental and FE results

Dataset	Built-up columns with bolted interconnection			Built-up columns with welded interconnection		
	No. of experiments/ FE data	$N_{b,u}/N_{b,u,pred}$ Mean	$N_{b,u}/N_{b,u,pred}$ CoV	No. of experiments /FE data	$N_{b,u}/N_{b,u,pred}$ Mean	$N_{b,u}/N_{b,u,pred}$ CoV
Experiment data	16	1.87	0.063	17	1.66	0.090
FE data	25	1.16	0.083	25	1.16	0.023
Experiment +FE data	41	1.37	0.300	42	1.35	0.200



(a) Built-up columns with bolted interconnection (b) Built-up columns with welded interconnection

Fig. 8 Comparison between normalised experimental and FE results and Eurocode 3 column buckling curves

Graphical comparisons between the predicted design resistances presented by the buckling curve *D* and the normalised FE and experimental compressive capacities of CFSS built-up columns are also provided in Fig. 8. The FE and experimental ultimate loads are normalised by the squash load and plotted against the column equivalent slenderness ratio. The normalised FE and experimental results are based on the weighted average material yield strength[14], which eliminates the influence of the enhanced material strength in corner regions of press-braked section from the buckling curve. The comparisons show that the FE results of the main parametric study closely follow the buckling curve pattern, and confirm the applicability of the proposed design approach both for CFSS built-up compressed members with bolted and welded interconnections.

5 Reliability Analysis

In order to evaluate the reliability of the proposed design method and identify the value of the partial factor for member resistance γ_{M1}, the statistical analysis based on provisions stated in Annex D, EN1990[25] is performed. The points, representing pairs of corresponding experimental ($N_{b,u,exp}$) and FE ($N_{b,u,FE}$), and design data ($N_{b,u,pred}$), are plotted in Fig. 9. The diagram shown in Fig. 9 indicates the expected trend line of FE data regarding to line $\theta = \pi/4$ for stainless steel alloys. However, the experimental results show a scatter in comparison with numerical results. Thus, with the aim of obtaining an economical design resistance function, the generated results are split into two subsets with respect to FE and experimental results. Table 5 lists the key statistical parameters for comparisons between predicted design resistances and experimental and numerical data, respectively: the design (ultimate limit state) fractile factor $k_{d,n}$, the correction factor represented is the average experiment or FE resistance-to-design model resistance ratio based on a least squares best fit to the slope of all data *b*, the CoV of the experimental and FE data relative to the design model resistance V_δ, the combined CoV incorporating both model and basic variable uncertainties V_r and the partial factor for member resistance γ_{M1}. For yield strength, over-strength value of 1.3 and a CoV of 0.06 for austenitic stainless steel are used, as recommended by Afshan et al. [26].

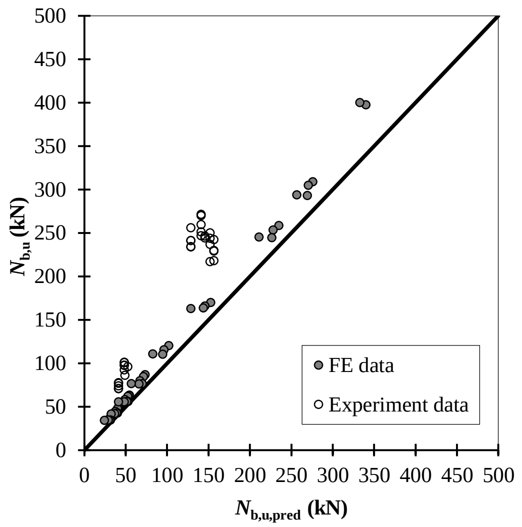


Fig. 9 Comparison of experimental and FE resistance with design resistance predictions

It can be seen from Table 5 that the obtained partial safety factors for the proposed design method, based on experimental and FE data, exceed the codified value of 1.1 in EN 1993-1-4[12]. Similar observations were found in a statistical analysis for flexural buckling resistance performed by Afshan et al.[26]. This indicates a need for further experiments in this structural field, in order to generate a larger database for more precise statistical analysis.

Table 5 Summary of reliability analysis of proposed design method based on experimental and FE results

Section type	Material	Dataset	No. of experiments / FE data	$k_{d,n}$	b	V_δ	V_r	γ_{M1}
Closely spaced built-up section	Austenitic stainless steel	Experiments	33	3.041	1.693	0.100	0.122	1.18
		FE data	50	3.048	1.141	0.060	0.093	1.13

6 Conclusions

A comprehensive investigation of the structural behaviour of CFSS closely spaced built-up members under pure compression, including literature review, experimental[14],[15], qualitative[16] and quantitative numerical studies, was carried out with the aim of acquiring new knowledge and a valuable database that enabled the development of an accurate and reliable design method. The following conclusions are drawn:

1. The structural response of a built-up column is simultaneously affected by a wide range of influencing parameters which determine the interaction level between individual chord members and developing shear forces in the interconnections. The type of interconnections, the number of interconnections and initial overall geometric imperfections have a crucial impact on a column's buckling resistance. However, the influence of the type and number of interconnections significantly vary depending on column slenderness and the distribution and magnitude of the imperfections. Based on results of the main parametric study in which the impact of overall and local chord slenderness and interconnection stiffness have been investigated, the initial overall geometric imperfection of a sine wave with an amplitude of $L/1000$ has a strong effect on the ultimate buckling resistance of a built-up column of intermediate and high slenderness. The combined weakening effect due to residual stresses and reduction of enhanced material strength properties in the vicinity of welds affects the column's behaviour in the low slenderness domain. The number of interconnections does not affect the compressive capacity of a built-up column with bolted interconnections: by decreasing the chord slenderness ratio, the ultimate buckling load remains approximately unchanged within the whole analysed slenderness range, with deviations up to 3.6%. This is caused by the flexibility of bolted interconnections and slipping effects in the hole-to-bolt clearance which contributes to higher shear deformations. On the other hand, decreasing the chord slenderness ratio results in a gradual growth of compressive capacity of the built-up column with welded interconnections for intermediate and high slenderness: in the high slenderness domain up to 16.5%, while in the low slenderness domain, the increase is limited on 1.3% by the detrimental welding effects. The built-up column with welded interconnections exhibits better structural response than those with bolted interconnections in almost the whole slenderness range: the ultimate buckling loads of welded built-up columns are 3–16.8% higher compared with columns with bolted interconnections;

2. Based on the imperfection sensitivity study, the shape of initial imperfections significantly determines the level of the column buckling resistance. The distribution of imperfections represented as a sine wave of individual chords between interconnections does not lead to the premature failure of individual chords of built-up columns with two modules between interconnections. Furthermore, such shapes of initial out-of-straightness ensure higher initial stiffness and compressive capacity of the built-up column: in comparison with the compressive capacity of built-up columns affected by a bow imperfection and an amplitude of $L/1000$, the increase of ultimate buckling loads varies significantly from 21% up to 250% over the analysed slenderness domain;

3. The FE results generated in the main parametric study and experimental data have been used to develop and validate a simple method for the design of pin-ended CFSS built-up columns whose chords are oriented back-to-back and directly connected by bolts or welds, by focusing on compact and semi-compact cross-sections. Aiming to fully exploit the shear capacity of the interconnections, the proposed design procedure involves two different formulas for shear stiffness, separately provided for built-up columns with bolted interconnections and built-up columns with welded interconnections. The flexural-buckling resistance is determined by considering the buckling curve D in conjunction with the non-dimensional limiting slenderness $\bar{\lambda}_0 = 0.2$. The proposed design method extends limits of the chord slenderness ratio-to-overall slenderness ratio up to 65% for both types of built-up columns;

4. The reliability analysis of the proposed design method performed on 33 experimental and 50 numerical results indicates a higher value of the partial safety factor in comparison with the codified value of 1.1 in EN 1993-1-4[12] and suggests that an increase in the number of reliable data for more precise statistical analysis is necessary.

List of Symbols

A	cross-sectional area of a built-up column
A_{ch}	cross-sectional area of one chord of a built-up column
a	distance between mid-points of interconnections (restraints of chords)
CFSS	cold-formed stainless steel
CoV	coefficient of variation
c	width or depth of a part of a cross section
d_0	hole diameter for the bolt
E	modulus of elasticity
FE	finite element
f_y	yield strength taken as the 0.2 % proof strength $f_{0.2}$
f_u	ultimate tensile strength
h_0	distance of centroids of chords of a built-up column
I	second moment of area of the built-up section, about the buckling axis $I = I_0 + 2I_{\mathrm{ch}}$
I_{ch}	second moment of area of single chord section about minor principal axis parallel to the buckling axis $I_{\mathrm{ch}} = A_{\mathrm{ch}} i_{\min}^2$
I_0	second moment of area of the built-up section about the buckling axis, neglecting the second moment of area of individual chords about their own minor principal axis $I_0 = 2A_{\mathrm{ch}}(h_0/2)^2$
I_{eff}	effective second moment of area of the built-up column
I_b	second moment of area of one batten about the buckling axis
i	radius of gyration of the built-up section about the buckling axis (minor principal axis)
i_{min}	minimum radius of gyration of single chord members
k	buckling length factor
L	length of built-up column
m	strain hardening parameter
N_{cr}	critical force of the built-up column
$N_{cr,eff}$	effective critical force of the built-up column
$N_{cr,V}$	critical buckling load of a built-up column
$N_{b,u}$	ultimate buckling load
$N_{b,u,bold}$	ultimate buckling load of built-up column with bolted interconnections
$N_{b,u,weld}$	ultimate buckling load of built-up column with welded interconnections
$N_{b,u,exp}$	experimental ultimate buckling load
$N_{b,u,FE}$	FE ultimate buckling load
$N_{b,Rd}$	design buckling resistance
n	strain hardening parameter
S_v	shear stiffness of a closely spaced built-up column
t	relevant thickness
α	imperfection factor
δ_0	overall imperfection amplitude
γ_{M1}	partial factor for the resistance of members

ε coefficient depending on f_y; $\varepsilon = \sqrt{\frac{235}{f_y}\frac{E}{210000}}$

ε_u strain corresponding to the ultimate tensile strength

η imperfection parameter

λ overall column slenderness ratio

λ_{ch} chord slenderness ratio

$\bar{\lambda}_0$ non-dimensional limiting slenderness ratio

$\bar{\lambda}$ non-dimensional slenderness ratio

$\bar{\lambda}_{eq}$ equivalent non-dimensional slenderness ratio

μ efficiency factor

ν Poisson's ratio

ϕ value for determining the reduction factor χ

χ reduction factor for the relevant buckling mode

ω local imperfection amplitude

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References

[1] F. Bleich, Buckling Strength of Metal Structures, McGraw-Hill Book Company, 1952.

[2] R. Zandonini, Stability of Compact Built-Up Struts: Experimental Investigation and Numerical Simulation, Costruzioni Metalliche (1985) 202-204.

[3] A. Astaneh, S.C. Gael, R. D. Hanson, Cyclic Out-of-Plane Buckling of Double-Angle Bracing, J. Struct. Eng., ASCE, 111(5) (1985) 1135-1153.

[4] C.J. Zahn, G. Haaijer, Effect of Connector Spacing on Double Angle Compressive Strength, Eng. J., AISC 25(3) (1988) 109–118.

[5] Load and Resistance Factor Design Specification for Structural Steel Buildings, American Institute of Steel Construction (AISC), INC., Chicago, IL., September 1, 1986.

[6] F. Aslani, S.C. Goel, An Analytical Criterion for Buckling Strength of Built-up Compression Members, Eng. J. AISC 28 (4) (1991) 159-168.

[7] Specification for Structural Steel Buildings. An American National Standard ANSI/AISC 360-05, American Institute of Steel Construction (AISC), INC, Chicago, IL., March 9, 2005.

[8] A. Sato, C.M. Uang, Modified Slenderness Ratio for Built-up Members, Eng. J., AISC (2007) 269–280.

[9] Specification for Structural Steel Buildings. An American National Standard ANSI/AISC 360-10, American Institute of Steel Construction (AISC), Chicago, IL., June 22, 2010.

[10] Specification for Structural Steel Buildings. An American National Standard ANSI/AISC 360-16, American Institute of Steel Construction (AISC), Chicago, IL., July 7, 2016.

[11] D.R. Sherman, J.A. Yura, Bolted double angle compression members, J. Construct. Steel Res., 46 (1–3) (1998) 470–471.

[12] Eurocode 3: Design of steel structures – part 1-4: General rules – supplementary rules for stainless steels, including amendment A1 (2015), EN 1993-1-4:2006+A1:2015, Brussels, Belgium, CEN 2015.

[13] Eurocode 3: Design of steel structures – Part 1-1: General rules and rules for buildings EN 1993-1-1, Brussels, Belgium, CEN 2005.

[14] J. Dobrić, D. Budjevac, Z. Marković, N, Gluhović, Material and cross-section behaviour of stainless steel compressed members, J. Construct. Steel Res., under review.

[15] J. Dobrić, Z. Marković, D. Budjevac, M. Spremić, N. Fric, Resistance of the cold-formed built-up stainless steel members – Part I: Experiment, J. Construct. Steel Res., under review.

[16] J. Dobrić, M. Pavlović, Z. Marković, D. Buđevac, M. Spremić, Resistance of cold-formed built-up stainless steel members – Part II: Numerical simulation, J. Construct. Steel Res., under review.

[17] Eurocode 3: Design of Steel Structures: Design of joints EN 1993-1-8, Brussels, Belgium, CEN 2005.

[18] ABAQUS User Manual. Version 6.12. Providence, RI, USA: DS SIMULIA Corp; 2012.

[19] Execution of steel structures and aluminium structures – Part 2: Technical requirements for steel structures EN 1090-2, Brussels, Belgium, CEN 2008.

[20] I. Arrayago, E. Real, L. Gardner, Description of stress–strain curves for stainless steel alloys, Mater. Des., 87 (2015) 540–552.

[21] Specification for the Design of Cold-Formed Stainless Steel Structural Members. ASCE Standard SEI/ASCE 8-02, American Society of Civil Engineers, Reston, VA, 2002.

[22] L. Gardner, R. Cruise, Modelling of residual stresses in structural stainless steel sections, J. Struct. Eng., ASCE 135 (1) (2009) 42–53.

[23] L. Gardner, D. A. Nethercot, Numerical modelling of stainless steel structural components a consistent approach, J. Struct. Eng., ASCE 130 (10) (2004) 1586–1601.

[24] Design Manual for Structural Stainless Steel First Edition, Fourth Edition, The Steel Construction Institute, 2017.

[25] Eurocode – basis of structural design EN 1990, Brussels, Belgium, CEN 2012.

[26] S. Afshan, P. Francis, N. R. Baddoo, L. Gardner, Reliability analysis of structural stainless steel design provisions, J. Construct. Steel Res., 114 (2015) 293–304.