Građevinski materijali i konstrukcije
Building Materials and Structures
journal homepage: www.dimk.rs

doi: 10.5937/GRMK2104235D UDK: 624.014.2.04
Original scientific paper 669.14.018.8

Stability design criteria for closely spaced built-up stainless steel columns

Jelena Dobrić*[1)], Nina Gluhović[1)], Zlatko Marković[1)], Dragan Buđevac[1)]

[1)] *University of Belgrade, Faculty of Civil Engineering, Bulevar kralja Aleksandra 73, 11000 Belgrade, Serbia*

Article history
Received: 16 October 2021
Received in revised form: 27 November 2021
Accepted: 30 November 2021
Available online: 30 December 2021

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

ABSTRACT
This paper aims to develop design recommendations for closely spaced built-up stainless steel columns, based on findings gained in 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 and addresses their flexural buckling capacity about the minor axis. The impact of overall and local chord slenderness, interconnection stiffness, geometric imperfections and material nonlinearity is 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 that include different formulas for shear stiffness.

Highlights:
1. FE parametric studies including overall and chord slenderness and interconnection type.
2. Assessment of sensitivity to geometric imperfections.
3. Development of a design method for closely spaced built-up columns.
4. Reliability analysis of proposed design method.

1 Introduction

The use of cold-formed steel members with open cross-sections in built-up assemblies extends their application to light framing systems, wall bearing systems, trusses, latticed transmission towers and communication structures. If these structures are in a specific, aggressive or urban area, different stainless steel alloys may be utilized owing to their excellent corrosion resistance, ease of maintenance, good toughness, high fire resistance, pleasing appearance and general environmental benefits. 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 compared to hot-rolled or welded single members at similar cost.

The built-up columns made of symmetrically placed individual channels or angle sections are more stable in torsional or torsional-flexural buckling than their individual, integral members. Furthermore, cross-sectional distortions, residual stresses and heat-affected zones in the vicinity of welds may be considerably minimized by the discontinuous welding process. The structural response of built-up column is more complex than that of a comparable solid column considering the reduced shear rigidity of built-up section with discrete interconnections. 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 bending deflection may significantly vary depending on the interconnectivity along the chords. In contrast to welded interconnections, the hole clearance in bolted interconnections can result in a more substantial longitudinal slip between the chords and, consequently, leads to 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 suitable design procedure.

Over the past two decades, significant attention has been paid to aspects of the potential use of stainless steel in construction. Experimental work has focused on stainless steel structural elements of tubular and hollow cross-section. The number of investigations on open stainless steel sections is much 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. Bleich (1952) [1] developed a simplified analytical criterion based on an energy approach to determine the modified slenderness ratio of pin-ended battened columns. Zandonini [2] 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.

* Corresponding author:
E-mail address: jelena@imk.grf.bg.ac.rs

Astaneh et al. [3] performed tests on two back-to-back angles with welded, snug-tight and preloaded bolted interconnections. Using the data from these investigations [2], [3] as the basis, Zahn and Haaijer [4] recognized the effect 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 fully effective slenderness.

Based on Bleich's work [1], Aslani and Goel [6] proposed a new analytical formula which includes a section separation ratio α to determine shear stiffness provided by interconnections, and verified it by their 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 AISC LRFD Specification [5] and was also adopted in the Specification for Structural Steel Buildings ANSI/AISC 360-05 [7]. Based on the new test database, Sato and Uang [8] developed 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. These equations, valid for built-up columns with either 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] showed that preventing longitudinal slip in the end interconnections has a beneficial effect on the overall behaviour of built-up members. They also proposed an equation for determining the shear transfer force in the end interconnections to prevent slip between individual chords. As per section E6 of ANSI/AISC 360-16 [10] 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 longitudinal spacing should not be larger than four times the bolt diameter over 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 of individual chords should not exceed 75% of the governing slenderness ratio of the built-up member.

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 states that the design provisions for carbon steel columns given in EN 1993-1-1 [13] may be applied to stainless steels columns. EN 1993-1-1 [13] has different analytical method for the design of compressed built-up members in comparison with ANSI/AISC 360-16 [10]. 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. Essentially, 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.

Clause 6.4.4 also 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 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 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 propose new design criteria for these types of structural elements. The investigation focuses on pin-ended built-up columns formed from two press-braked channel chords oriented back-to-back to form a non-slender I-section, addressing their flexural buckling capacity about the built-up 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 snug-tight bolts (in EN 1993-1-8 [17] denoted as shear bolt connection category A) or by welds. The design model is compatible with rules given in EN 1993-1-4 [12], EN 1993-1-1 [13] and is based on Bleich's work [1].

2 FE parametric studies

2.1 Description of influencing parameters

Extensive FEPSs (Finite Element Parametric Studies) are conducted with reference to a wide-ranging set of overall and local chord slenderness and interconnection type in order to meet different performance levels of structural behaviour and to establish a calculation model for the design buckling resistance $N_{b,Rd}$ of the compressed built-up columns 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 flexural-buckling tests [15], [16].

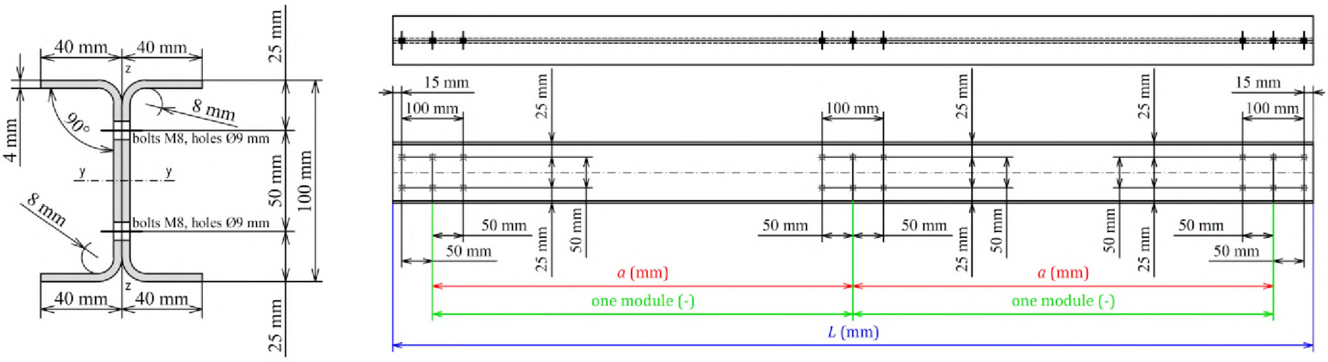
The CFSS (Cold-Formed Stainless Steel) built-up columns consist of two press-braked channel chords placed back-to-back and directly and discontinuously interconnected by means of ether groove welds or bolts (see Figure 1). The nominal dimensions of the channel section are 100 x 40 x 4 mm with an internal corner radius of 8 mm. The cross-section is classified as class 3 [14] according to EN 1993-1-4 [12]. The nominal length of welded interconnections is 100 mm. The bolted interconnections are designed with six M8 bolts grade 8.8 in the arrangement shown in Figure 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 hole clearance is provided.

Table 1. Parameters and ranges considered in the main FEPS for columns with bolted interconnections

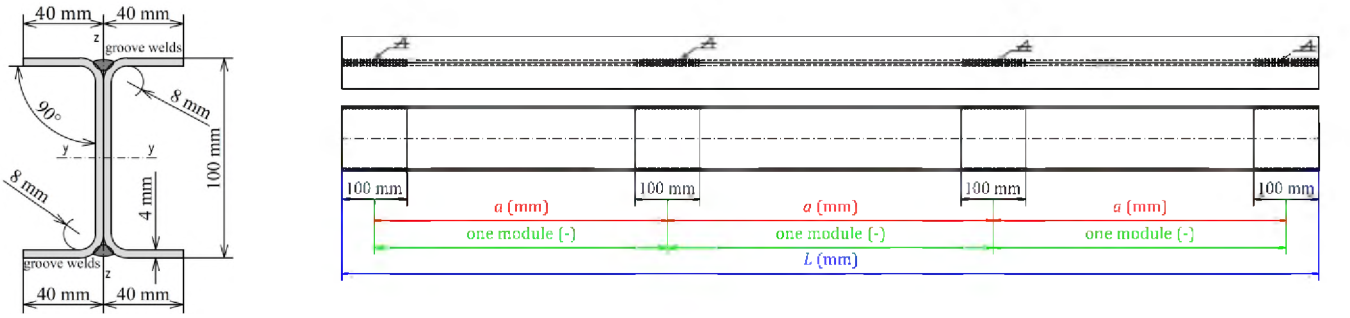
Designation of the FE model	Parameters					Ratio λ_{ch}/λ
	Nominal length L (mm)	Slenderness ratio of built-up column $\lambda = L/i$	Number of modules between interconnections	Maximum distance between interconnections of chords a (mm)	Maximum chord slenderness ratio $\lambda_{ch} = a/i_{min}$	
U31b-2	500	30.7	2	185	15.3	0.50
U49b-3	800	49.2	3	225	18.7	0.38
U49b-2	800	49.2	2	335	27.8	0.57
U62b-3	1000	61.5	3	290	24.0	0.39
U62b-2	1000	61.5	2	435	36.1	0.59
U92b-3	1500	92.2	3	460	38.1	0.41
U92b-2	1500	92.2	2	685	56.8	0.62
U123b-3	2000	123.0	3	625	51.8	0.42
U123b-2	2000	123.0	2	935	77.5	0.63
U154b-4	2500	153.7	4	595	49.3	0.32
U154b-3	2500	153.7	3	790	65.5	0.43
U154b-2	2500	153.7	2	1185	98.2	0.64
U184b-5	3000	184.5	5	575	47.7	0.26
U184b-4	3000	184.5	4	720	59.7	0.32
U184b-3	3000	184.5	3	960	79.6	0.43
U184b-2	3000	184.5	2	1435	119.0	0.64
U215b-5	3500	215.2	5	675	56.0	0.26
U215b-4	3500	215.2	4	845	70.1	0.33
U215b-3	3500	215.2	3	1125	93.3	0.43
U215b-2	3500	215.2	2	1685	139.7	0.65
U246b-6	4000	246.0	6	645	53.5	0.22
U246b-5	4000	246.0	5	775	64.3	0.26
U246b-4	4000	246.0	4	970	80.4	0.33
U246b-3	4000	246.0	3	1290	107.0	0.43
U246b-2	4000	246.0	2	1935	160.4	0.65

Table 2. Parameters and ranges considered in the main FEPS for columns with welded interconnections

Designation of the FE model	Parameters					Ratio λ_{ch}/λ
	Nominal length L (mm)	Slenderness ratio of built-up column $\lambda = L/i$	Number of modules between interconnections	Maximum distance between interconnections of chords a (mm)	Maximum chord slenderness ratio $\lambda_{ch} = a/i_{min}$	
U31w-2	500	30.7	2	200	16.6	0.54
U49w-3	800	49.2	3	240	19.9	0.40
U49w-2	800	49.2	2	350	29.0	0.59
U62w-3	1000	61.5	3	300	24.9	0.40
U62w-2	1000	61.5	2	450	37.3	0.61
U92w-3	1500	92.2	3	470	38.1	0.41
U92w-2	1500	92.2	2	700	56.8	0.62
U123w-3	2000	123.0	3	632.5	51.8	0.42
U123w-2	2000	123.0	2	950	77.5	0.63
U154w-4	2500	153.7	4	600	49.3	0.32
U154w-3	2500	153.7	3	800	65.5	0.43
U154w-2	2500	153.7	2	1200	98.2	0.64
U184w-5	3000	184.5	5	580	47.7	0.26
U184w-4	3000	184.5	4	725	59.7	0.32
U184w-3	3000	184.5	3	970	79.6	0.43
U184w-2	3000	184.5	2	1450	119.0	0.64
U215w-5	3500	215.2	5	680	56.0	0.26
U215w-4	3500	215.2	4	850	70.1	0.33
U215w-3	3500	215.2	3	1135	93.3	0.43
U215w-2	3500	215.2	2	1700	139.7	0.65
U246w-6	4000	246.0	6	650	53.5	0.22
U246w-5	4000	246.0	5	780	64.3	0.26
U246w-4	4000	246.0	4	975	80.4	0.33
U246w-3	4000	246.0	3	1300	107.0	0.43
U246w-2	4000	246.0	2	1950	160.4	0.65



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



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

Figure 1. Nominal geometry and parameter designation of built-up columns in FEPSs

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.

FEPSs were divided into two parts: the main parametric study and the imperfection sensitivity study, in which the influences of various parameters on the column compressive resistance were analysed. The main FEPS focuses on a wide range of overall and local chord slenderness ratios as listed in Table 1 and Table 2 for columns with bolted and welded interconnections, respectively. The considered parameters were column length L and spacing between interconnections a. In Table 1 and Table 2, λ is the overall slenderness ratio of the entire section about the built-up member axis (equal to column length-to-radius of gyration of the built-up section about the buckling axis—minor principal axis), whereas λ_{ch} is chord slenderness ratio equal to spacing between interconnections-to-minimum radius of gyration of an individual chord. The chord slenderness ratios of built-up columns were varied by changing the number of modules between interconnections, where one module represents one regular spacing between two adjacent interconnections (see Figure 1). The analysed range of overall slenderness ratios from 31 to 246 (the corresponding range of non-dimensional slenderness ratio $\bar{\lambda}$ is 0.38 to 3.07) 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. This is strongly associated with the findings of the experimental research of Dobrić et al. [15], where, for interconnection spacings thus adopted, the governing buckling mode of all built-up specimens was governed by the overall flexural buckling about the minor principal axis of the built-up section. The designations of the FE models in Table 1 and Table 2 are 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 weld or bolt interconnection. The number in the third position represents the number of modules between interconnections.

The imperfection sensitivity study was performed to thoroughly assess possibilities for potential buckling failures of individual chord members affected by the shape and magnitude of their initial out-of-straightness imperfections. 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 two variabilities in the amplitude of δ_0 = L/1000 and a permissible fabrication tolerance of δ_0 = L/750 specified in EN 1090-2 [19], as shown in Figure 2. It was assumed that these imperfections can lead to premature failure of individual chords before the built-up column as a whole becomes unstable. Therefore, this study focused only on built-up columns with interconnections at the ends and at mid-height, for which the chord slenderness ratio-to-overall slenderness ratio is approximately 65%. A range of intermediate and high overall slenderness of 92, 184 and 246 was considered. 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, shown in Figure 3, which is used as an input parameter in the main FEPS.

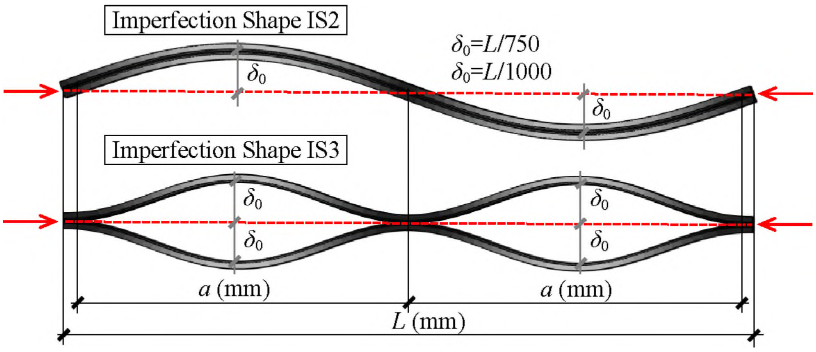


Figure 2. Overall geometric imperfections of built-up columns with two modules used in the imperfection sensitivity study

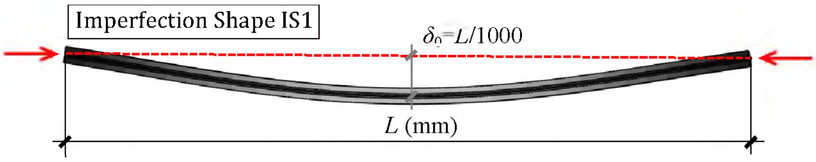


Figure 3. Overall geometric imperfections used in the main FEPS

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 that leads to good agreement between tests [15] and FE results [16]. These are: material nonlinearity, strain hardening effects, residual stresses and annealing effects in the vicinity of welded interconnections, and bolt slipping in bolted interconnections.

Comprehensive FE simulations of the flexural-buckling tests are presented in detail in [16]. The mechanical properties obtained from the flat and corner longitudinal tensile coupon tests [14] were 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 was applied in the HAZ and welds [16], where a modified Ramberg–Osgood material model according to Arrayago et al. [20] was used to develop the stress–strain curve. Nominal values of key mechanical properties for annealed stainless steel EN 1.4301 (ASCE 304) were used according to Annex B of SE/ASCE 8-02 [21]. Table 3 summarises the key material properties adopted for each of the three considered material models. The yield strength f_y is 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, are in accordance with the two-stage Ramberg–Osgood material model [20].

Plasticity with isotropic hardening was 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 were transformed to true stress–strain curves for input in the Abaqus plasticity model [18].

The individual chords were 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, were used to form the mesh of the welded interconnections. Contact conditions between the chords and the welds were defined by tie constraints at the joining surfaces. The attachment tool in the Abaqus software package [18] which involves attachment points was utilized to model the bolts in a nominal arrangement between chords. The bolts were modelled using the Cartesian mesh-independent connector type with a linear elastic stiffness of 50 kN/mm. This value was calibrated against test data obtained on specimens with bolted interconnections [16]. The rotational stiffness of connector was not considered. The degrees of freedom of the bolt were coupled to the adjacent nodes by distributing the coupling system 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 were kinematically constrained by means of two rigid bodies connected by a spring element. The surface-to-surface general contact interaction was selected in the modelling approach in order to take into account the interactions between individual chords. The hard contact formulation of normal behaviour and the penalty friction formulation of tangential behaviour were used. A friction coefficient of 0.35 was assumed for all contact surfaces.

The cross-section points at the column's ends were kinematically constrained to the central upper and lower reference points which were assigned hinged boundary conditions. Displacement control was used to apply the compressive load; a vertical displacement of 10 mm was applied to the upper reference point.

Table 3. Key material properties adopted in the FE models

Position	f_y (N/mm^2)	f_u (N/mm^2)	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 distribution of residual stresses in a fabricated austenitic stainless steel I-section, proposed by Gardner and Cruise [22], was adopted in the regions of welded interconnections. The residual stresses were incorporated into the models as initial model conditions through predefined fields. Maximum tensile residual stresses of $1.3 \cdot f_y$ = 399 N/mm^2 were set in the vicinity of welds, where f_y = 307 N/mm^2 is experimentally obtained yield strength of the basic flat sheet material [14]. The residual stresses are in self-equilibrium in the cross-section with maximum compressive residual stresses of 94 N/mm^2. Discontinuous welding of individual chords caused variable cross-sections along the column. Thus, a stable equilibrium in the longitudinal direction was obtained in an initial analysis step prior to applying the compression load to the FE model. The residual stresses induced by the manufacturing process were not included in the FE models due to their minimal influence on the member compressive resistance [23].The FE analysis included an eigenvalue Linear Buckling Analysis (LBA) and a nonlinear buckling analysis. The eigenvalue LBA was employed in order to permit numerical Geometrically and Materially Nonlinear Analysis with Imperfections (GMNIA). Superposition of initial imperfections in the shape of the lowest overall buckling mode with an amplitude of $\delta_0 = L/1000$ (labelled in Figure 3 as imperfection shape IS1) and the lowest local (cross-section) buckling mode with an amplitude of ω_0 is assigned to all FE models. The amplitudes of local geometric imperfection ω_0 were determined by means of the modified Dawson and Walker model [24], as given by Eq.(1), where t is the thickness of the plate, fy is the yield strength of basic flat sheet material and $\sigma_{cr,min}$ is the minimum critical buckling stress of all the plate elements of the cross-section:

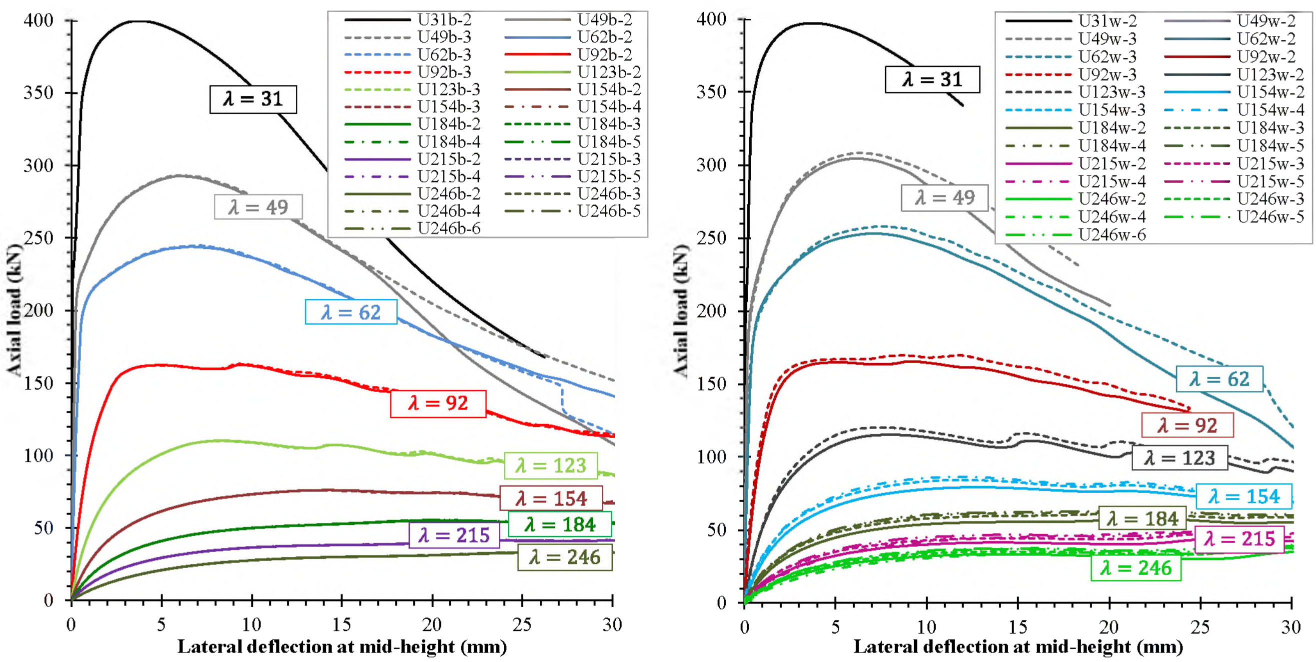
$$\varpi_0 = 0.036t\left(f_y/\sigma_{\text{cr,min}}\right) \tag{1}$$

This expression has been found to have good agreement between the tests and FE results for stainless steel channel stub columns [14] and slender built-up columns with chord in contacts [16]. The adopted approach in imperfection modelling is based on scientific investigation [23] covering structural behaviour of cold-formed stainless steel columns.

A GMNIA was performed to obtain the ultimate loads and potential failure modes of CFSS 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 FEPSs were performed as quasi-static using the dynamic explicit solver in the Abaqus software package [18], thus, successfully overcoming the usual convergence issues.

3 Discussion of results

Key numerical results of the main FEPS presented in diagrammatic form as load–lateral deflection curves in the buckling plane are shown in Figure 4, both for built-up columns with bolted and welded interconnections. Figure 5 compares the buckling capacities of equivalent 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, the load–lateral deflection curves generated for columns affected by imperfection shapes IS1, IS2 and IS3 are presented and compared in Figure 6. The value of ultimate buckling load $N_{b,u}$ is also shown on the corresponding curve. Moreover, Figure 7 compares the different buckling responses of the selected built-up column U184w-2 caused by variations in the shape of overall geometric imperfections, but without change the amplitude value of $L/1000$. A brief analysis of the results of FEPSs is presented as follows:



(a) Built-up columns with bolted interconnections

(b) Built-up columns with welded interconnections

Figure 4. Load–lateral deflection curves at mid-height of FE models – main FEPS

1) The failure mode of each FE model is overall flexural buckling about the minor axis of the built-up section without any local-overall interactions. The structural integrity of the built-up section is maintained in the ultimate limit state: the premature failure of the individual chord members does not occur;

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

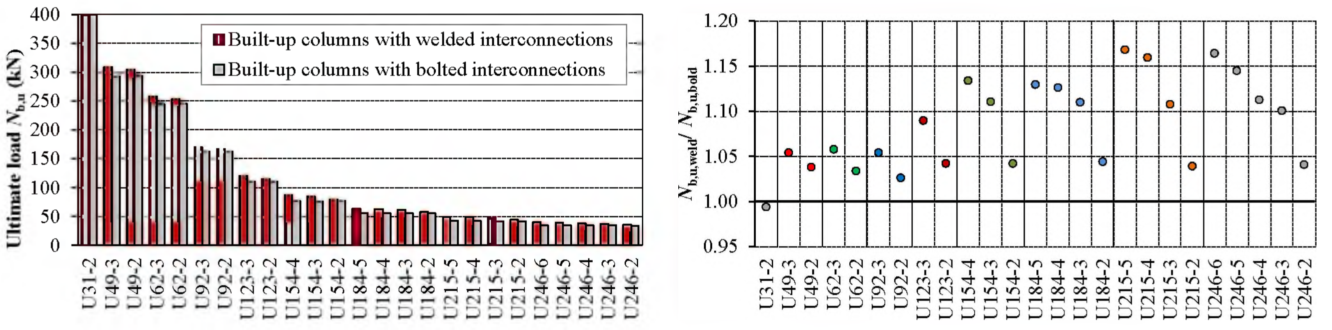
3) The FE models with bolted interconnections of the same overall slenderness and with different chord slenderness ratios have almost identical buckling and post-buckling structural behaviour (see coincidences of load-lateral deflection curves in Figure 4a). By increasing the number of interconnections, the buckling loads of the columns remain unchanged with a small deviation 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 tests [15], an increase of column compressive capacity of 24% 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 geometric imperfections of the FE models;

4) In contrast to the previous finding, the FE models with welded interconnections of the same overall slenderness showed an increase of the compressive resistance with an increasing number of modules between interconnections (see Figure 4b). Increasing the number of interconnections from two to five increases the column resistance by 16% in the high slenderness domain (λ = 215). However, this increase is limited to 1.3% for the low slenderness (λ = 49) due to the effects of the welding process. In addition, for the variation of the number of modules from two to three in the high slenderness range (λ = 184), the increase of column buckling resistance was 10% in the tests [15], whereas in the main FEPS is only 6%;

5) As indicated in Figure 5, the FE models with welded interconnections exhibit a better structural response than FE models with bolted interconnections, over almost the entire slenderness range, except for low slenderness λ = 31. This finding is strongly influenced by the higher shear stiffness of welded interconnections compared with bolted interconnections. The lowest structural response of the welded column with slenderness λ = 31 is associated with the effects of residual stresses and partial annealing in the HAZ. In the case of columns with interconnections at their ends and at 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 to 246, and approximately 1.03 in the slenderness range λ = 49 to 92. Decreasing of the chord slenderness ratio in the overall slenderness range λ = 123 to 246 resulted in a gradual growth of compressive resistance as the improved composite action of chords within the welded built-up section leads to a more favourable buckling response. For the maximum number of modules, used in the high slenderness range, the buckling resistance of the columns with welded interconnections is approximately 17% higher relative to the equivalent columns with bolted interconnections. As shown in Figure 5b, the FE model U246-6 and FE model U 215-5 have approximately same values of the ratio $N_{b,u,weld}/N_{b,u,bold}$. Hence, in comparison with columns with slenderness λ = 215, the slenderest columns (λ = 246) are less sensitive to the benefits of the higher stiffness of the welded interconnections. This leads to the conclusion that the beneficial effects of higher number of interconnections between chords of built-up columns on their ultimate resistances increase with increasing the overall column slenderness.

However, it should be noted that the ratio $N_{b,u,weld}/N_{b,u,bold}$ ≈ 1,10 is approximately same for equivalent FE models with three modules between interconnections in the intermediate and high slenderness range λ = 123 to 246;

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 Figure 6). As expected, the compressed built-up members are most sensitive to the sine wave shape of initial geometric imperfections with an amplitude of $L/1000$ at 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 resistance of built-up columns and may lead to an inelastic buckling response in the intermediate slenderness range. It can be seen from Fig. 3 that the direction lines of applied compression loads deviate from mid-length of individual chords between adjacent inter-



(a) Ultimate buckling loads of FE models

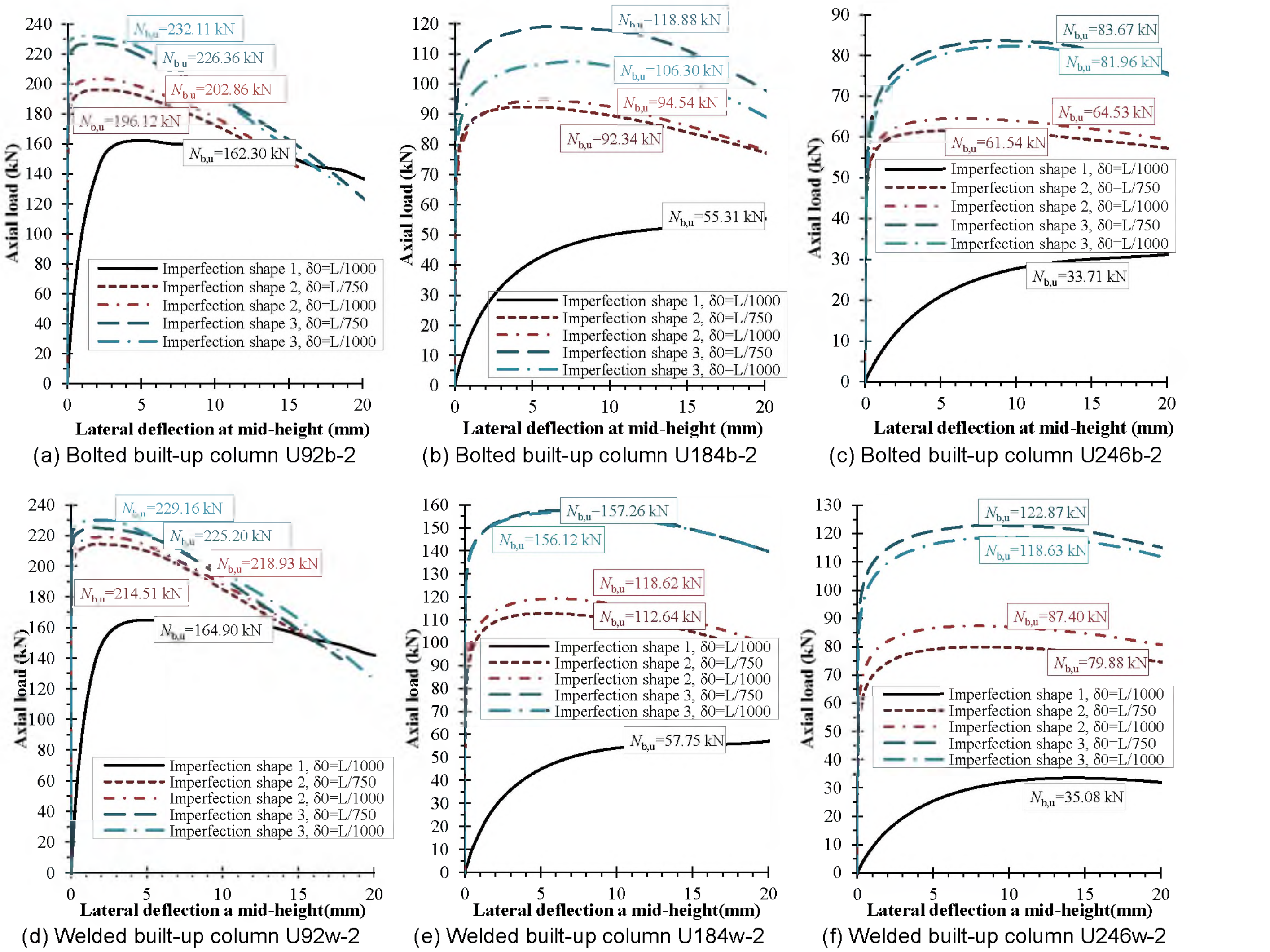
(b) Ratios between ultimate buckling loads of welded and bolted built-up columns

Figure 5. Comparison of ultimate buckling loads of FE models – main FEPS

connections. However, they pass through the built-up section at the column's mid-height both for IS2 and IS3, which has an impact on the overall buckling behaviour of the columns. Besides, in the case of IS3, the individual chords are specifically geometrically positioned within the built-up section, which affects the overall flexural stiffness of the built-up columns. Moreover, the analysed shapes of imperfections IS2 and IS3 do not represent the critical, lowest buckling modes of built-up columns that were computed by linearized eigenvalue analyses.

There are specific simultaneous effects of imperfection amplitude and imperfection shape IS3 both on welded and bolted built-up columns with high slenderness. It can be seen in Figure 6b, Figure 6c, Figure 6e and Figure 6f that 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 $\delta_0 = L/1000$ when considering imperfection shape IS3. Additionally, for the same shape IS3, the columns with welded interconnections have a much greater effectiveness in the high slenderness range both for λ = 184 and 246 than the equivalent bolted columns, while their compressive resistances are almost equal in the intermediate slenderness range (λ = 92).

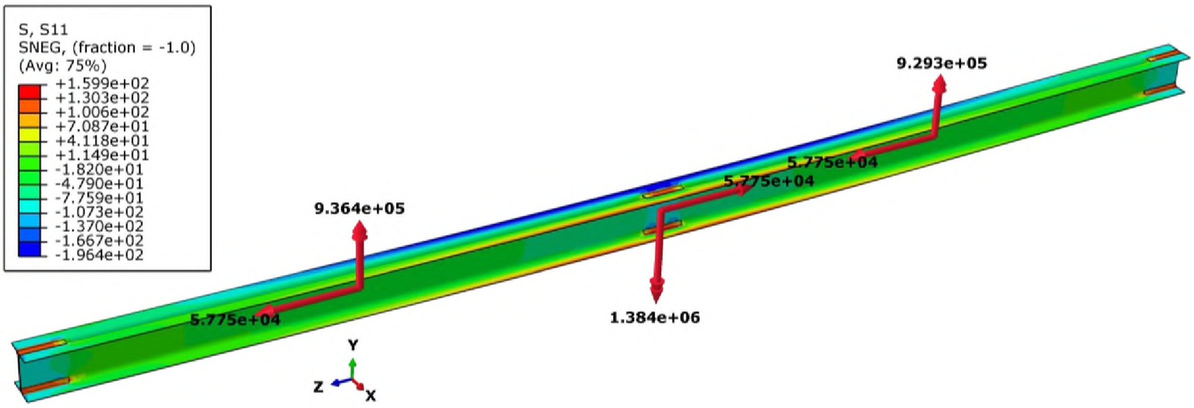
The influence of the variation in initial out-of-straightness on the ultimate response of the built-up columns is also highlighted through the variation in distribution of longitudinal stresses and internal forces generated at the failure state of the selected FE model U184w-2, as shown in Figure 7. The internal forces and moments are calculated for the column cross-sections at mid-height and at mid-distances between interconnections. It can be clearly seen from Figure 7 that the considered imperfection shapes do not change the critical failure mode of the built-up column which occurs by overall buckling about the minor axis. The location of the critical cross-section under the maximum bending moment is near the mid-height of column affected by either IS1 or IS3. However, the critical cross-section of column affected by the asymmetric curvature of imperfection shape IS2 is located approximately at mid-distance between the interconnections. For bending about the minor axis, the longitudinal stresses vary linearly through the both flanges with the maximum compressive stresses of $\sigma_{11,max}$ = 191 N/mm^2 to $\sigma_{11,max}$ = 200 N/mm^2 occurring at the edge fibres on the one side of the buckled column. Contrary to the column affected by either IS2 or IS3 for which the entire critical cross-section is under compressive stresses (see Figure 7b and Figure 7c), the tensile longitudinal stresses occurring on the convex side of the deflected column influenced by IS1 can be seen in Figure 7a.



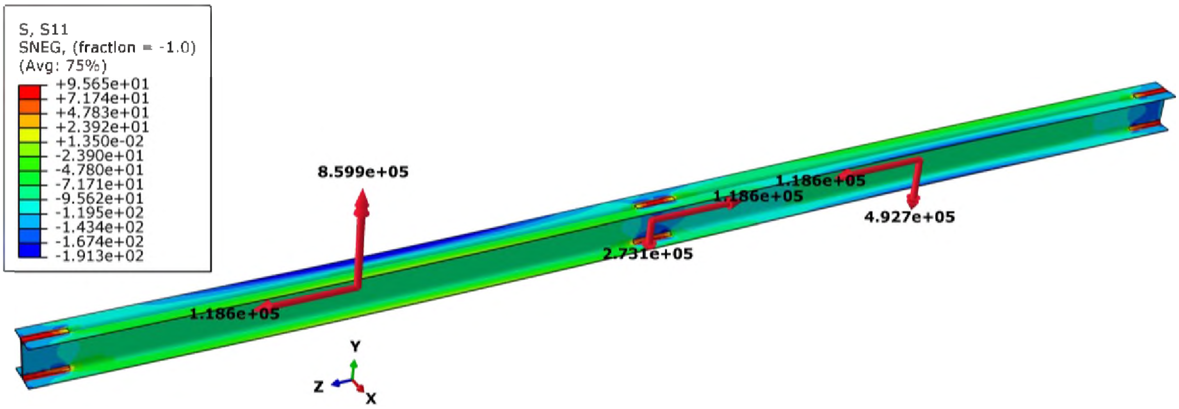
(a) Bolted built-up column U92b-2 (b) Bolted built-up column U184b-2 (c) Bolted built-up column U246b-2

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

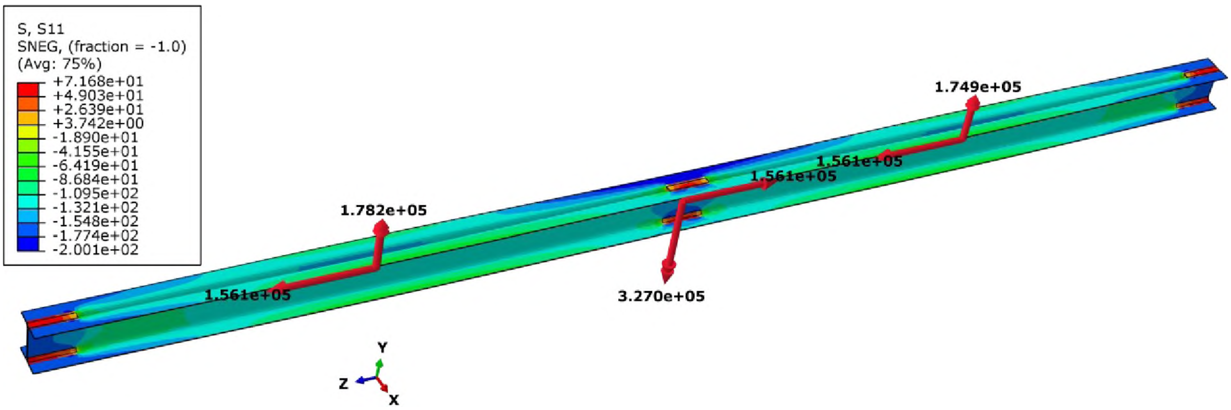
Figure 6. Load–lateral deflection curves at mid height of FE models – imperfection sensitivity study



(a) Buckling response affected by IS1 and δ_0 = $L/1000$



(b) Buckling response affected by IS2 and δ_0 = $L/1000$



(c) Buckling response affected by IS3 and δ_0 = $L/1000$

Figure 7. The axial stresses and internal forces of FE model U184w-2 at the ultimate load level

Quantification of the increase of column resistance by changing the shape and amplitude of the geometric imperfection, presented in Table 4, is provided through comparisons of ultimate buckling loads of built-up columns affected by IS2 and IS3 and amplitudes δ_0= $L/1000$ and δ_0 = $L/750$ with those of built-up columns affected by imperfection shape IS1 and an amplitude of δ_0 = $L/1000$. As indicated in Table 4, 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 range, 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 4. Quantification of the increase of buckling loads by changing geometric imperfections.

Column	Amplitude	$N_{\mathrm{b,u}}^{\mathrm{IS3},\delta_0}/N_{\mathrm{b,u}}^{\mathrm{IS1},L/1000}$ Imperfection shape IS3	$N_{\mathrm{b,u}}^{\mathrm{IS2},\delta_0}/N_{\mathrm{b,u}}^{\mathrm{IS1},L/1000}$ 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	2.70	2.05
	δ_0 = L/750	2.72	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 column buckling tests [15] and the results of main FEPS 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 [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 $N_{\mathrm{cr,V}}$:

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

where k is the buckling length factor for battened built-up columns, given by Eq. (3):

$$k = \sqrt{1 + \frac{\pi^2 I_0}{24 I_{\mathrm{ch}}}\left(\frac{a}{L}\right)^2 + \frac{\pi^2 E I_0}{L^2}\frac{a h_0}{12 E I_{\mathrm{b}}}} \tag{3}$$

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 (2) can also be written as:

$$N_{\mathrm{cr,V}} = \frac{1}{\frac{L^2}{\pi^2 EI} + \frac{a^2}{24 E I_{\mathrm{ch}}}\left[\frac{I_0}{I} + \frac{2 I_{\mathrm{ch}} h_0}{I_{\mathrm{b}} a}\frac{I_0}{I}\right]} \tag{4}$$

In foregoing 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. The following notations for critical force N_{cr} and shear stiffness S_{v} may be introduced:

$$N_{\mathrm{cr}} = \frac{\pi^2 EI}{L^2} \tag{5}$$

$$S_{\mathrm{V}} = \frac{24 E I_{\mathrm{ch}}}{a^2\left[\frac{I_0}{I} + \frac{2 I_{\mathrm{ch}} h_0}{I_{\mathrm{b}} a}\frac{I_0}{I}\right]} \tag{6}$$

Therefore, Eq. (4) can be reformulated as follows:

$$N_{\mathrm{cr,V}} = \frac{1}{\frac{1}{N_{\mathrm{cr}}} + \frac{1}{S_{\mathrm{V}}}} \tag{7}$$

In order to simplify Eq.(6), 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 I_0/I as equal to unity. This leads to

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

However, the outcomes gained in the investigation of Aslani and Goel [6] show that Bleich's simplified approximation, given by Eq.(8), may result in significant 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. It was shown that the I_0/I ratio decreases as the distance between centroids of chords becomes smaller. On the other hand, based on the test data of Zandonini [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 aspects: Eq.(8) 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 as follows:

$$S_{\mathrm{V}} = \frac{24EI_{\mathrm{ch}}}{a^2\left[1+\dfrac{2I_{\mathrm{ch}}h_0}{I_{\mathrm{b}}a}\right]} \leq \frac{2\pi^2 EI_{\mathrm{ch}}}{a^2} \tag{9}$$

Expressions for shear stiffness S_v given by Eqs (6), (8) and (9) 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.(5) 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.(5) is taken as:

$$I = 0.5{h_0}^2 A_{\mathrm{ch}} + 2I_{\mathrm{ch}} \tag{10}$$

It should be noted that Eq. (5) deviates from the expression for effective critical load $N_{cr,eff}$ stated in clause 6.4.1 of EN 1993-1-1 [13] given by Eq.(11), in terms of the second moment of area of the battened built-up column, as follows:

$$N_{\mathrm{cr,eff}} = \frac{\pi^2 EI_{\mathrm{eff}}}{L^2} \tag{11}$$

where:

$$I_{\mathrm{eff}} = 0.5{h_0}^2 A_{\mathrm{ch}} + 2\mu I_{\mathrm{ch}} \tag{12}$$

In Eq.(12), 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 0 and 1.0 and depends on the overall slenderness of the built-up column.

4.2 Proposed 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 of 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 $\eta = \alpha(\bar{\lambda}_{\mathrm{eq}} - \bar{\lambda}_0)$. 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 EN 1993-1-4 [12] for flexural buckling: for cold-formed sections (α = 0.49, $\bar{\lambda}_0$ = 0.4) and for welded sections (α = 0.76, $\bar{\lambda}_0$ = 0.2). However, by based on research findings conducted over the last decade, the fourth edition of the Design Manual for Structural Stainless Steel [25] has revised the buckling curves and adopted the more 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 α = 0.76 in conjunction with a non-dimensional limiting slenderness $\bar{\lambda}_0$ = 0.2 is used in this method 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 $N_{cr,V}$ given by Eq.(7) is utilized;

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. (5) and (10) are used in the calculation method;

3) The second term within the denominator brackets is excluded from the expression for shear stiffness S_v in Eq. (9) 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. (9) the expression on the right-hand side of this equation should be used. This gives:

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

Eq.(13) is intended to predict the flexural-buckling resistance of CFSS built-up columns with bolted interconnections;

4) Using key findings from Aslani and Goel [6], Bleich's exact solution given by Eq. (6) 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. (6) should be excluded, which leads to:

$$S_{\mathrm{V}} = \frac{24EI_{\mathrm{ch}}}{a^2}\frac{I}{I_0} \tag{14}$$

Thus Eq. (14) is used to predict the flexural-buckling resistance of CFSS closely spaced built-up columns with welded interconnections. The flowchart in Figure 8 gives an overview of the proposed design method.

4.3 Range of application

The procedure covers the following conditions:

– the cross-section is cold-formed from austenitic stainless steel;

– the cross-section is classified as class 3;

– 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 length of the bolted interconnection is defined by the distance between end bolts in the longitudinal direction (in a line in the direction of load transfer) that 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 bolt holes in both directions is $5d_0$, the end distances from the centre of a bolt hole to the adjacent end of a chord's web is $2d_0$ in the case of end interconnections, where d_0 is the diameter of bolt hole;

Cross-secton class
web $c/t \leq 37\varepsilon$; flanges $c/t \leq 14\varepsilon$

$$\frac{a}{L}\frac{i}{i_{\min}} \leq 0.65$$

Built-up columns with bolted interconnections	Built-up columns with welded interconnections
$S_{\mathrm{V}} = \dfrac{2\pi^2 EI_{\mathrm{ch}}}{a^2}$	$S_{\mathrm{V}} = \dfrac{24EI_{\mathrm{ch}}}{a^2}\dfrac{I}{I_0}$

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

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

$$\bar{\lambda}_{\mathrm{eq}} = \sqrt{\frac{Af_{\mathrm{y}}}{N_{\mathrm{cr,V}}}}$$

$$\phi = 0.5\left[1 + \alpha\left(\bar{\lambda}_{\mathrm{eq}} - \bar{\lambda}_0\right) + \bar{\lambda}_{\mathrm{eq}}^{\;2}\right]$$

$$\chi = \frac{1}{\phi + \sqrt{\phi^2 - \bar{\lambda}_{\mathrm{eq}}^{\;2}}}$$

$$N_{\mathrm{b,Rd}} = \frac{\chi A f_{\mathrm{y}}}{\gamma_{\mathrm{M1}}}$$

Figure 8. Design method applicable for buckling design checks of CFSS closely spaced built-up columns

– the length of 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;

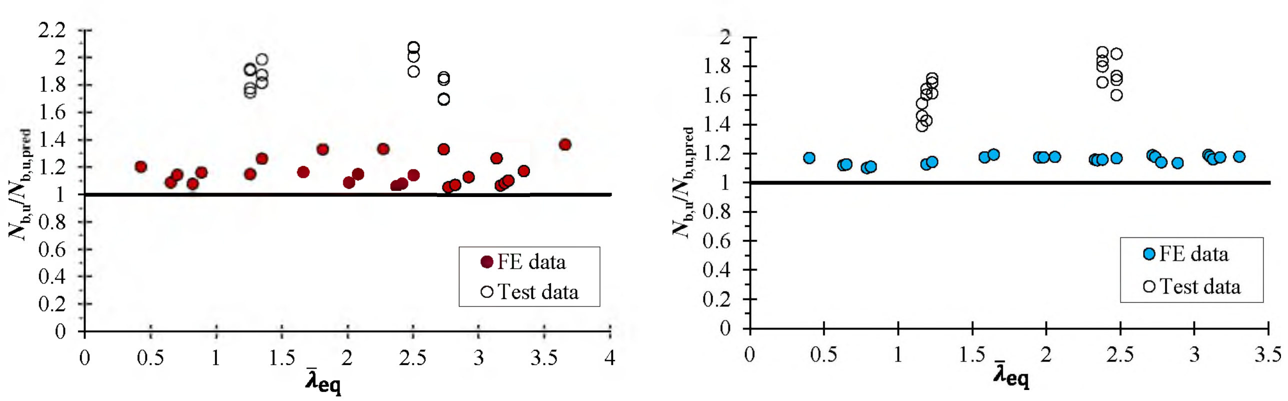
– the spacing between interconnections is limited such that the slenderness of the individual chords does not exceed 65% of the overall built-up slenderness about the axis of the built-up cross-section that corresponds to the minor principal axis; the chord slenderness ratio is based on the distance 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, a comparative analysis is performed in which predicted buckling resistances of built-up columns are compared with generated test [15] and numerical buckling resistances. In the design calculation, f_y was taken as 307 N/mm^2, which is the measured strength of flat sheet steel [14] and a partial safety factor of γ_{M1} was taken as 1.0. The comparisons are presented in Figure 9 and a summary of the obtained results is presented in Table 5.

The mean test-to-predicted buckling load ratio $N_{b,u,test}/N_{b,u,pred}$ is 1.87 and the Coefficient of Variation (CoV) is 6.3% for the columns with bolted interconnections. The mean value of $N_{b,\,test}/N_{b,u,pre}$d 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 test 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 Nb,u/Nb,u,pred ratio is 1.35 and the CoV is 20% for the columns with welded interconnections.

The significant distinctions between test and FE data are strongly associated with a discrepancy in the shapes and magnitudes of initial geometric imperfections of specimens in the test [15] and columns in the main FEPS, respectively. The measured imperfection amplitudes of specimens in the corresponding buckling plane are *L*/3432 to *L*/24000 [15]. Besides, the shapes of measured imperfections do not reflect the lowest overall buckling mode of tested built-up columns. As for the geometric imperfections in the main FEPS, they were taken as sinusoidal shapes with an amplitude of *L*/1000 at columns' mid-height representing the critical (lowest) buckling modes of all FE models in order to obtain lowest buckling resistances.



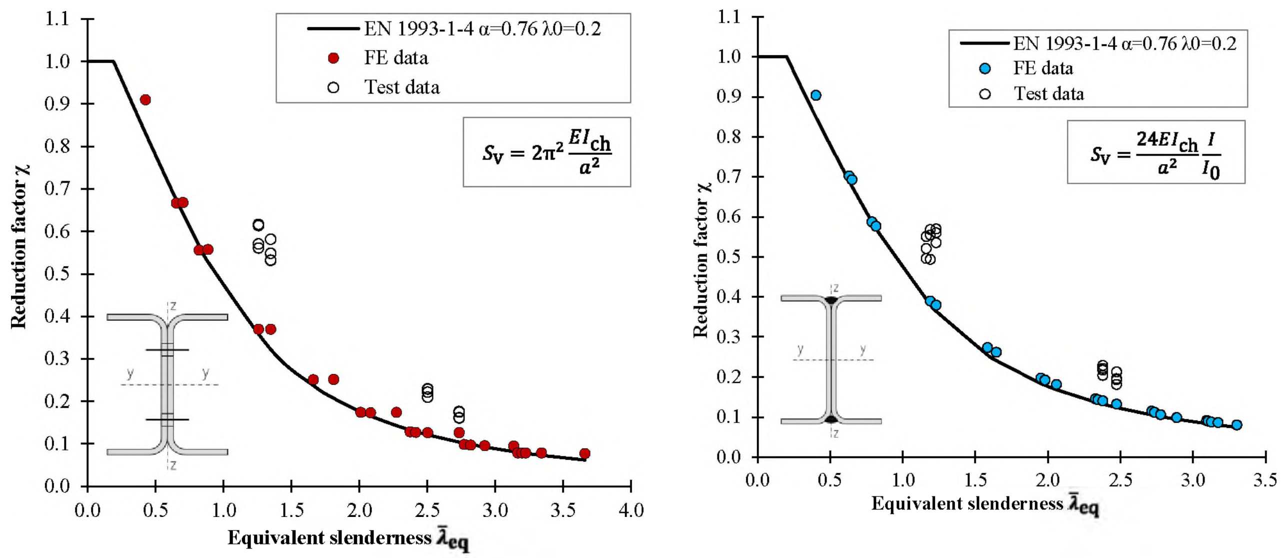
(a) Built-up columns with bolted interconnection

(b) Built-up columns with welded interconnection

Figure 9. Comparison between design resistance predictions and test and FE results

Table 5. Comparison between design resistance predictions and test and FE results

Dataset	Built-up columns with bolted interconnection			Built-up columns with welded interconnection		
	No. of test data/FE data	$N_{b,u}/N_{b,u,pred}$		No. of test data/FE data	$N_{b,u}/N_{b,u,pred}$	
		Mean	CoV		Mean	CoV
Test data	16	1.87	0.063	17	1.66	0.090
FE data	25	1.16	0.083	25	1.16	0.023
Test +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

Figure 10. Comparison between normalised test and FE results and buckling curve d

Graphical comparisons between the predicted design resistances presented by the buckling curve *d* and the normalised FE and test compressive resistances of CFSS built-up columns are also provided in Figure 10. The FE and test ultimate loads are normalised by dividing by the squash load and are plotted against the column equivalent slenderness ratio. The normalised FE and test results are based on the enhanced average yield strength of the cross-section [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}, a statistical analysis based on provisions stated in Annex D of EN1990 [26] was performed. The points, representing pairs of corresponding test ($N_{b,u,test}$) and FE ($N_{b,u,FE}$) data, and design data ($N_{b,u,pred}$), are plotted in Figure 11.

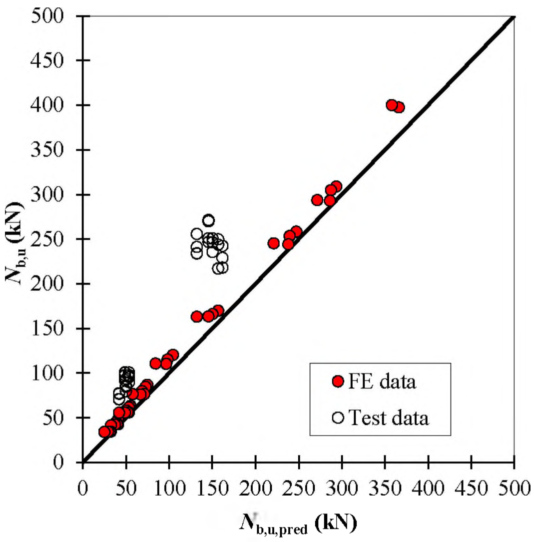


Figure 11. Comparison of test and FE resistance with design resistance predictions

The diagram shown in Figure 11 indicates the expected trend line of FE data regarding to line θ = π/4 for stainless steel alloys. However, the test results show a scatter in comparison with numerical results. Thus, to obtain an economical design resistance function, the generated results are split into two subsets with respect to FE and test results, as per clause D 8.2.2.5 of EN 1990 [26]. Table 6 lists the key statistical parameters for comparisons between predicted design resistances and test and numerical data, respectively. These are:

– the design (ultimate limit state) fractile factor, $k_{d,n}$,

– the correction factor represented is the average test 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 test and FE data relative to the design model resistance, V_δ,

– the combined CoV incorporating both model and basic variable uncertainties, V_r

– the partial factor for member resistance, γ_{M1}.

For the yield strength, an over-strength value of 1.3 and a CoV of 0.06 for austenitic stainless steel are used, as recommended by Afshan et al. [27].

It can be seen from Table 6 that the obtained partial safety factor for the proposed design method, based on FE data, is lower than the codified value of 1.10 in EN 1993-1-4 [12]. However, when only the test data are considered, the partial safety factor γ_{M1} is higher than 1.10; this is due to the variation and scattering of the test data obtained from experiments [15]. This indicates a need for further tests in this range, in order to generate a larger database for more precise statistical analysis.

6 Conclusions

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

1. The structural response of a built-up column is affected by a wide range of influencing parameters which determine the interaction level between individual chord members and the shear forces in the interconnections. The type of interconnections, the number of interconnections and initial overall geometric imperfections have an important effect 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 FEPS in which the effects 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 affects 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 resistance 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 bolt hole clearance which contributes to higher shear deformations.

Table 6. Summary of reliability analysis of proposed design method based on test and FE results

Section type	Material	Dataset	No. of test data / FE data	$k_{d,na}$	b	V_δ	V_r	γ_{M1}
Closely spaced built-up section	Austenitic stainless steel	Test data	33	3.041	1.693	0.100	0.122	1.18
		FE data	50	3.048	1.141	0.060	0.090	1.09

Decreasing the chord slenderness ratio results in a gradual increase of compressive resistance of the built-up column with welded interconnections up to 16% in the high slenderness range, and up to 1.3% in the low slenderness range. 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 to 17% higher compared with columns with bolted interconnections;

2. Based on the imperfection sensitivity study, the shape of initial imperfections significantly affects the column buckling resistance. For considered values of the individual chord slenderness-to-the overall built-up slenderness ratios, up to the value of 0.65, the imperfection shape of individual chords represented as a sine wave 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 from 21% up to 250% over the analysed slenderness range;

3. The FE results generated in the main parametric study and test 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 semi-compact cross-sections. 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 a 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 test and 50 numerical results indicates that the partial safety factors γ_{M1} are close to the codified value of 1.1 in EN 1993-1-4 [12].

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)
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 $I_{\mathrm{ch}} = A_{\mathrm{ch}} i_{\mathrm{min}}^2$
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
$N_{b,u,bolt}$	ultimate buckling load of built-up column with bolted interconnections
$N_{b,u,weld}$	ultimate buckling load of built-up column with welded interconnections
$N_{b,u,test}$	test 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
$\bar{\lambda}_{\mathrm{eq}}$	equivalent non-dimensional slenderness ratio
μ	efficiency factor
v	Poisson's ratio
ϕ	value for determining the reduction factor χ
χ	reduction factor for the relevant buckling mode
ω	local imperfection amplitude

Acknowledgements

This investigation is supported by the Serbian Ministry of Education, Science and Technological Development through the 200092 project.

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, Behaviour of stainless steel press-braked channel sections under compression, J. Constr. Steel Res. 139 (2017) 236–253.
[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. 145 (2018) 552–572.
[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. 140 (2018) 247–260.
[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, Modeling of residual stresses in structural stainless steel sections, J. Struct. Eng., ASCE 135 (1) (2009) 42–53.
[23] L. Gardner, D. A. Nethercot, Numerical modeling of stainless steel structural components a consistent approach, J. Struct. Eng., ASCE 130 (10) (2004) 1586–1601.
[24] R.G. Dawson, A.C. Walker, Post-buckling of geometrically imperfect plates, J. Struct. Div. (ASCE) 98 (1) (1972) 75–94.
[25] Design Manual for Structural Stainless Steel First Edition, Fourth Edition, The Steel Construction Institute, 2017.
[26] Eurocode – basis of structural design EN 1990, Brussels, Belgium, CEN 2012.
[27] 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.