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Monte Carlo Simulations for Global and Local Interaction Buckling of Welded Box-sections

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Abstract
The present study focuses on investigating the interaction behavior of local and global buckling resistance of welded box-section columns by using stochastic analysis. Previous studies mainly investigated this failure mode using experimental or numerical investigations but using deterministic approaches. The current research employs Monte Carlo simulations as a robust numerical tool to estimate the interaction buckling resistance. Stochastic variables are defined, encompassing variabilities in geometrical and material properties, as well as local and global imperfections. To facilitate these simulations, a rigorously validated numerical model is employed, utilizing geometrical and materially nonlinear analysis with imperfections (GMNIA). This advanced modeling approach accounts for the complex nonlinearities inherent in the behavior of welded box-section columns. The characteristic resistance derived from the Monte Carlo simulation is compared to 1. test results, 2. analytical buckling resistance according to the Eurocode design approach, and 3. previously proposed deterministic approaches. Within the research program, an improved generalized design formula is developed to estimate the buckling resistance for pure local, global, and interaction buckling modes. The new design equation is compatible with the design rules of the Eurocode and enhances the ease of use of the standard regulations.
Keywords
local buckling, flexural buckling, welded box-section, interaction buckling, equivalent imperfections, Monte Carlo simulations

1 Introduction

Welded box-sections are extensively employed in the construction industry, primarily due to their advantageous properties and limited stability issues, making them practical structural components in various projects. More advanced techniques must be utilized to ensure the proper utilization of their structural performance. In general, welded box-sections experience three types of instabilities, namely, global, local, and interaction buckling. Global buckling in this paper is the flexural buckling of a pinned-pinned compressed member, having stability failure in a sine-wave form. Local buckling is a series of half sin-wave form deformations that manifest at the various locations within the plates comprising the structural section having a large width-to-thickness ratio. These deformations result in a substantial reduction in the load-carrying capacity of sections, hindering their ability to develop a fully plastic behavior of the section. An interesting phenomenon that is observed in sections susceptible to both local and global buckling is known as interaction buckling, where both buckling behaviors occur almost simultaneously, causing a significant reduction of the buckling capacity. While many research studies were conducted on pure local and global buckling, there has been a relatively limited focus on investigating the interaction behavior between these two types of buckling and developing advanced design methods estimating the interaction buckling resistance.

The primary objective of the current research is to investigate the structural behavior of welded box-section columns subjected to pure compression through advanced geometrically and materially nonlinear analysis with imperfections (GMNIA) and Monte Carlo simulations to accurately estimate the interaction buckling resistance of welded box-section columns.

The design method currently available in the Eurocode for the estimation of the interaction buckling resistance utilizes a simplified approach based on two reduction factors to account for the effect of global and local buckling. The global reduction factor is based on a modified

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Ayrton-Perry formula that was calibrated to estimate the global buckling resistance. A set of buckling curves specified in EN 1993-1-1 [1] can be utilized, depending on the geometrical and material properties of the section. The buckling curve that is currently used to estimate the buckling resistance of slender welded box-section columns is the buckling curve *b*. This curve takes into account the effect of geometrical and structural imperfections as well as all other uncertainties (e.g., eccentric loading …). The other reduction factor is used to take into account the effect of local buckling, which can be determined using a modified Winter-type buckling curve specified in EN 1993-1-5 [2]. This reduction factor is used in the effective width method to reduce the gross cross-sectional area of sections susceptible to local buckling. The Eurocode designates these sections as "Class 4" sections. To determine the characteristic interaction capacity according to this method, the reduction factors are multiplied by the gross cross-sectional area and the yield strength of the column. Within the current study, columns having class 4 cross-sections will be analyzed.

Schillo [3] conducted recent research on welded box-section columns to study the interaction buckling behavior of high-strength steel (HSS) columns. A new design formula was proposed that utilizes an equivalent geometrical imperfection to account for the effect of local buckling. The proposed method employed a similar approach as outlined in EN 1993-1-1 [1] for the assessment of global buckling. However, an additional equivalent imperfection was introduced to account for the loss of stiffness attributed to local buckling. Based on numerical and experimental study, it was emphasized that the Eurocode results show a large scatter compared to the obtained results. Degée et al. [4] performed a numerical and experimental study to investigate the interaction buckling of welded box-section columns. A new definition for global slenderness was proposed to account for the loss of stiffness due to the local buckling, where the local buckling reduction factor ρ was included in β factor to modify the global slenderness. Both research studies showed the currently adopted design method in the Eurocode is not accurate, and further investigation is needed over a large slenderness range. Radwan and Kövesdi [5] investigated the interaction buckling resistance using GMNIA to accurately estimate the buckling capacity of welded box-sections. It was found that the currently proposed Eurocode method provides an average estimation of the buckling resistance, with some results on the safe side and others on the unsafe side.

This research program commences by utilizing a validated numerical model that employs the GMNIA technique to accurately estimate the global and local interaction buckling resistance. The numerical model was thoroughly validated against experimental tests in previous research studies conducted by the authors [5–13]. Stochastic Monte Carlo simulations are conducted using the validated numerical model to accurately estimate the buckling capacity. The study takes into account several stochastic variables, including the geometrical and material properties, imperfections, and residual stress patterns. Therefore, the obtained buckling resistance highly represents the characteristic value of the buckling resistance of a real experimental test. The 5% lower quantile value is considered the characteristic resistance according to the Eurocode design rules. The 5% quantile resistance is obtained using Monte Carlo simulations. A new buckling resistance formula is fitted to the Monte Carlo simulations [5]. The obtained quantile resistance is compared to the deterministic resistance obtained in the previously developed method in the previous research of the authors [5] and the new fit formula. The study program is carried out according to the following steps:

1. Monte Carlo simulations are performed for pure local and pure global buckling resistances.
2. Monte Carlo simulations are performed for interaction buckling of local and global resistances.
3. A new generalized buckling formula is proposed.
4. Comparison is made between the Monte Carlo simulation results and both the deterministic design method and the new fitted formula.

2 Literature review: Previous research on interaction buckling of welded box sections

Degée et al. [4] conducted a series of experiments on rectangular welded box-section columns composed of S355 steel. The experimental program encompassed six specimens, each characterized by a consistent local slenderness $\bar{\lambda}_p = 0.9$ and varying global slenderness of $\bar{\lambda}_g = 0.35$, 0.55, and 0.7 to investigate the interaction buckling capacity. A numerical parametric study was conducted using a global imperfection of $L/1000$ and a local imperfection of $b/1000$ with applied residual stresses. It was found that curve *b* in EN 1993-1-1 [1] produces conservative results, while curve *a* exhibited a closer alignment with experimental data. In light of these observations, the researchers put forth an improved design method that incorporates the loss of stiffness attributable to the local buckling effect.

A modified global slenderness ratio, denoted as $\bar{\lambda}_{int}$, was established, which takes into account the effective moment of inertia and area as well as the local reduction factor ρ. This leads to higher estimated resistance as the value of $\bar{\lambda}_{int}$ is generally smaller than $\bar{\lambda}_g$.

Khan et al. [14] conducted a comprehensive series of experiments on welded box-section columns composed of S690 steel, with the primary focus being the investigation of the interaction buckling resistance. The program encompassed a total of fifteen specimens, supplemented by a numerical study. Local and global imperfections were introduced in the numerical model to estimate the buckling resistance. The local imperfections were set at a magnitude of $b/1000$ and the global imperfections at $L/1000$, with applied residual stresses. It was observed that specimens of intermediate lengths failed due to the interaction of both local and global buckling behaviors. The buckling curve b in EN 1993-1-1 [1] was recommended for the estimation of the buckling resistance as all the numerical results were above it, representing a lower bound for the numerical results. Usami and Fukumoto [15] conducted an extensive experimental study involving welded box-section columns composed of high-strength steel grades of S460 and S960. A total of twenty-seven compression tests were carried out as part of their study. Among these tests, twenty-four were subjected to concentric loading conditions, while the remaining three were subjected to eccentric loading configurations. Schillo [3] conducted a comprehensive investigation involving thirteen buckling tests performed on welded box-sections fabricated from S500 and S960 high-strength steels. The authors proposed a new method based on the EN 1993-1-1 [1] approach to estimate the buckling capacity of welded box-sections. The method involved a noteworthy modification to account for the loss of stiffness attributed to local buckling, using an equivalent local imperfection in the reduction formula.

Radwan and Kövesdi [5] conducted a large deterministic parametric study on welded box-sections columns to accurately estimate the interaction buckling capacity taking into account the nonlinear interaction effect. The authors utilized a new approach where imperfections that depend on the yield strength and slenderness were introduced into the numerical model to accurately estimate the buckling resistance [13]. A new method for estimating the interaction buckling resistance was proposed. The method introduced a new modification factor (f_{mod}) that depends on the local and global slenderness ratios, as shown in Eq. (1). The modification factor (f_{mod}) is determined according to Eqs. (2), (3). The global and local interaction effects were separated, and the design calculations followed the rules of Eurocode. The introduced method showed high agreement with the performed deterministic numerical simulations.

$$N_{b,int,Rd} = f_{mod} \times \chi \times \frac{A_{eff} \times f_y}{\gamma_{M1}} = f_{mod} \times \chi \times \frac{\rho A f_y}{\gamma_{M1}} \tag{1}$$

Where

$$f_{mod} = \begin{cases} 1, & \bar{\lambda}_g \le 0.4 \\ 1 + \left(\bar{\lambda}_g - 0.4\right) \times \left(f_{\max} - 1\right), & 0.4 < \bar{\lambda}_g < 1.4 \\ f_{\max}, & \bar{\lambda}_g \ge 1.4 \end{cases} \tag{2}$$

$$f_{\max} = \begin{cases} 1, & \bar{\lambda}_p \le 0.67 \\ 1 + \left(\bar{\lambda}_p - 0.67\right) \times 1.36, & 0.67 < \bar{\lambda}_p < 1 \\ 1.45, & \bar{\lambda}_p \ge 1 \end{cases} \tag{3}$$

χ is the reduction factor related to global flexural buckling according to EN 1993-1-1 [1], ρ is the reduction factor related to local plate buckling according to EN 1993-1-5 [2] – Annex B buckling curve is used in the A_{eff} calculation. The $\bar{\lambda}_p$ is the local slenderness ratio according to EN 1993-1-5 [2], $\bar{\lambda}_g$ is the global slenderness ratio according to Eq. (6.5) of EN 1993-1-1 [1].

3 The developed numerical model

The modeling process was carried out using Ansys software [16], utilizing SHELL81 elements. In order to capture the interaction behavior in the columns accurately, a geometrical and material nonlinear analysis with imperfections (GMNIA) approach is adopted. If appropriate imperfections and material models are applied in the numerical model, this method allows for accurate estimation of the buckling resistance of the columns under investigation. Within the numerical model, both global and local imperfections have been precisely defined, as illustrated in Fig. 1. The local imperfections are implemented in the numerical model as a series of half sin-waves with alternating amplitudes for adjacent edges. The number of half sin-waves corresponds to the length of the plate (L) divided by the width of the plate (b), as shown in Fig. 1 (a). The global imperfections are implemented as one half sin-wave spanning the overall length of the column, as shown in Fig. 1 (b) Both imperfections are implemented simultaneously in the case of interaction buckling, as shown in Fig. 1 (c) To define the boundary conditions and applied load in the numerical model, master nodes were placed

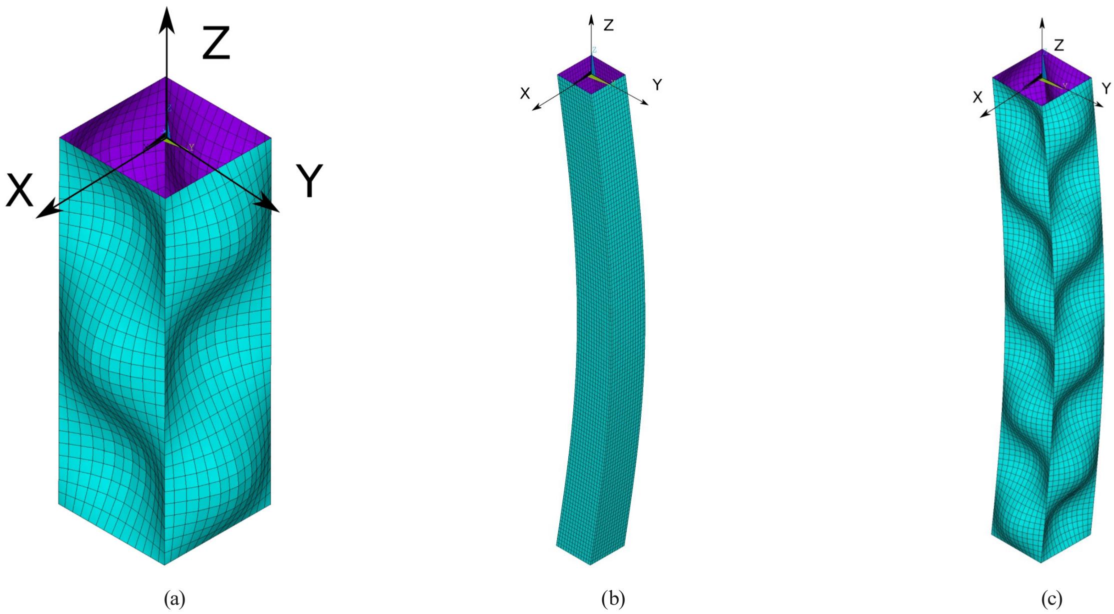

Fig. 1 Imperfection definition for columns experiencing (a) local, (b) global, (c) interaction buckling

at the center of the end cross sections of both sides of the column. Effective rigid diaphragms are introduced to connect all six degrees of freedom between the master nodes and the nodes at the end cross-sections. Specifically, translations in all global directions are fully restrained at the first master node, while at the second master node, only the translation in the X and Y directions are restrained, as the applied load is in the Z direction. Moreover, rotational freedom in the Z direction is restrained at both nodes.

The employed residual stress model has been substantiated to provide an accurate estimation of the buckling resistance, and it precisely adheres to the recommendations of ECCS [17] as well as the preliminary version of Eurocode prEN 1993-1-14 [18]. A light welding pattern was chosen in the study as the vast majority of the sections under study have a maximum thickness of 8 mm with a b/t ratio of 40, where a single-pass weld can be used. The details of the applied residual stress pattern are shown in [5].

In the context of this research, it was determined that a mesh sensitivity of 20 elements along the plate width produces a reliable estimation of the buckling resistance, deviating by only 1% from the values obtained using the smallest applied mesh size in the sensitivity analysis study, as shown in Fig. 2.

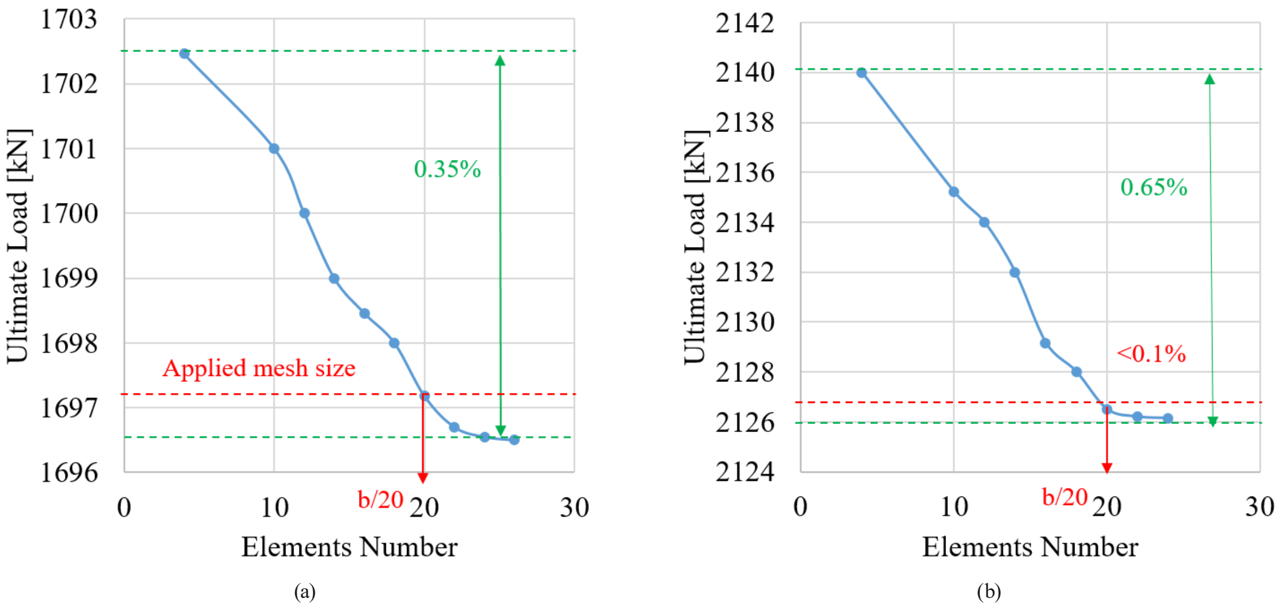

Fig. 2 The result of a mesh sensitivity analysis (a) for the smallest plate width (200 mm), (b) for the largest plate (450 mm) width in the study

The validation process of the numerical model is comprehensively elucidated in a preceding research study [5]. A rigorous validation process was performed, encompassing both normal and high-strength steel materials. The validation was performed using samples from various experimental research programs [3, 19]. The results of the validation process affirmed that the numerical model accurately estimates the buckling capacity of the tested specimens. A sample of the validated tests is shown in Table 1. Specifically, the statistical measures showed a mean ratio $F_{\text{numerical}}/F_{\text{experimental}}$ of 1.02 and a coefficient of variations of 0.05, indicating a close alignment between the numerical predictions experimental outcomes.

4 Probabilistic analysis

Within this research program, several variables have been defined as stochastic variables, encompassing yield strength, geometrical properties, and both local and global imperfections. Table 2 provides a concise summary of the distribution type, mean value, and the coefficient of variations for each of these variables. In accordance with the recommendations of the Joint Committee on Structural Safety (JCSS) [20], the yield strength is modeled as a stochastic variable, utilizing a log-normal distribution characterized by a coefficient of variation (V_{fy}) equal to 0.07. To comply with EN 1993-1-1 [1] guidelines, the material properties of steel are determined based on nominal values, with characteristic value defined as 5% quantile value according to EN1990 [21]. Therefore, the mean of yield strength is calculated according to Eq. (4), where 1.583 represents the z-value of 5% cumulative probability in log-normal distribution.

$$f_{y,\text{mean}} = \frac{f_{y,\text{nominal}}}{\left(1-1.583\times 0.07\right)} = \frac{f_{y,\text{nominal}}}{0.8892} \quad (4)$$

As per the guidelines specified in JCSS [20], the geometrical parameters, including the height (h), the width (b), the thickness (t), and the length (L), are characterized

Table 1 Results of the validation process

Steel grade	λ_g	λ_p	b (mm)	h (mm)	t (mm)	L (mm)	f_y (MPa)	f_u (MPa)	$F_{u,exp}$ (kN)	$F_{u,num}$ (kN)	$F_{u,num}/F_{u,exp}$
Yang et al. [19]											
S235	0.27	0.90	135.5	254.1	5.48	2251.6	309	458	1057.7	1039.4	0.98
S235	0.39	1.08	158	309	5.59	4159.9	309	458	1176.9	1102.4	0.94
S235	0.23	1.46	210	404.5	5.44	3585.3	309	458	1284.6	1229.8	0.96
S235	0.21	1.78	251.4	491.2	5.44	4378.1	309	458	1287.6	1286.0	1.00
S345	0.28	1.19	161.8	312.5	5.69	2786.9	385	546	1414.6	1423.2	1.01
S345	0.24	1.52	209.5	403.4	5.82	3582.4	385	546	1456.7	1563.1	1.07
S345	0.22	1.87	251	491.7	5.8	4375.6	385	546	1431.1	1524.7	1.07
S345	0.40	1.17	313.2	312.6	5.83	5035.1	385	546	1472.2	1601.0	1.09
Schillo [3]											
S500	0.33	1.06	159.75	159.5	4.1	1599	562	640	880.3	933.0	1.10
S500	0.37	1.07	160	159.25	4.0	1800	562	640	883.9	918.1	1.04
S500	0.40	1.08	160	159	4.0	2000	562	640	858.2	892.5	1.04
										Mean	1.02
										CoV	0.05

Table 2 Properties of the probabilistic variables

Variable	Distribution Type	Mean Value	CoV
Yield strength f_y	Log-normal	Nominal/0.8892	0.07
Height h	Normal	Nominal	0.005
Width b	Normal	Nominal	0.005
Thickness t	Normal	Nominal	Min(0.05, 0.3 × (1/t) × (1/3))
Length L	Normal	Nominal	0.005
Global imperfection	Log-normal	L/3200	0.6
Local imperfection b-direction	Log-normal	b/400	0.55
Local imperfection h-direction	Log-normal	h/400	0.55

by a normal distribution. The mean value for each parameter is set equal to the nominal value. Moreover, the coefficient of variations (CoV) is defined as 0.005 for h, b, and L. However, for the thickness (t), a different approach is taken to align with realistic considerations and adhere to the quality management standards of steel producers. Here, the CoV for the thickness (t) is taken as the minimum value of 0.05 or $0.3 \times (1/t) \times (1/3)$, as demonstrated by Schillo [3]. This adjustment ensures that the thickness variability reflects the steel manufacturing standards. The global and local imperfections are characterized by a log-normal distribution, as was shown by different research studies [22]. Based on statistical data and research findings [3, 23], the mean value of the global imperfections is equal to $L/3200$, and CoV is equal to 0.6. For local imperfections, a comprehensive database of local imperfection measurements was compiled from various research programs [3, 19]. The statistical assessment of this database revealed that the mean value for local imperfections aligns closely with $b/400$, and the associated CoV is determined to be 0.55. These values effectively characterize the variability associated with local imperfections.

Latin Hypercube Sampling (LHS) is the chosen method for sample selection in Monte Carlo simulation. LHS is preferred over other methods due to its ability to prevent the clustering of samples during the random generation process, which enhances the representativeness of the sample set. The determination of the sample size primarily hinges on the desired probability level. In general, to achieve reliable results, a minimum sample size within the range of $30/P$ to $100/P$ is recommended. For instance, in the case of estimating the 5% lower quantile, this translates to an approximate sample size ranging from 600 to 2000 samples [24]. This sample size has been selected to ensure the robustness and accuracy of the Monte Carlo simulation results. The Response Surface Method (RSM) is employed as a strategy to reduce the computational burden by approximating the results of Monte Carlo simulations through the fitting of a surface. In this approach, a second-order regression model is employed to formulate the response surface, and it has been determined that a minimum of 45 samples is required to adequately fit the response surface when dealing with eight stochastic variables [16]. The response surface is subsequently utilized to efficiently estimate load capacities, offering a faster alternative to conducting new numerical analyses. In the present study, 70 numerical simulations were performed for each to establish the response surface. A total number of 10,000 samples were derived for each test using the response surface. This study encompasses a total of 44 geometries with 8000 Monte Carlo simulations. This method ensures both computational efficiency and reliability in obtaining the desired outcomes.

5 Results and discussion

The results of Monte Carlo simulations are summarized in Fig. 3, where the top figure displays the performed Monte Carlo simulations represented as filled cyan circles. Five different surfaces are shown, corresponding to the buckling resistance surfaces determined using various methods.

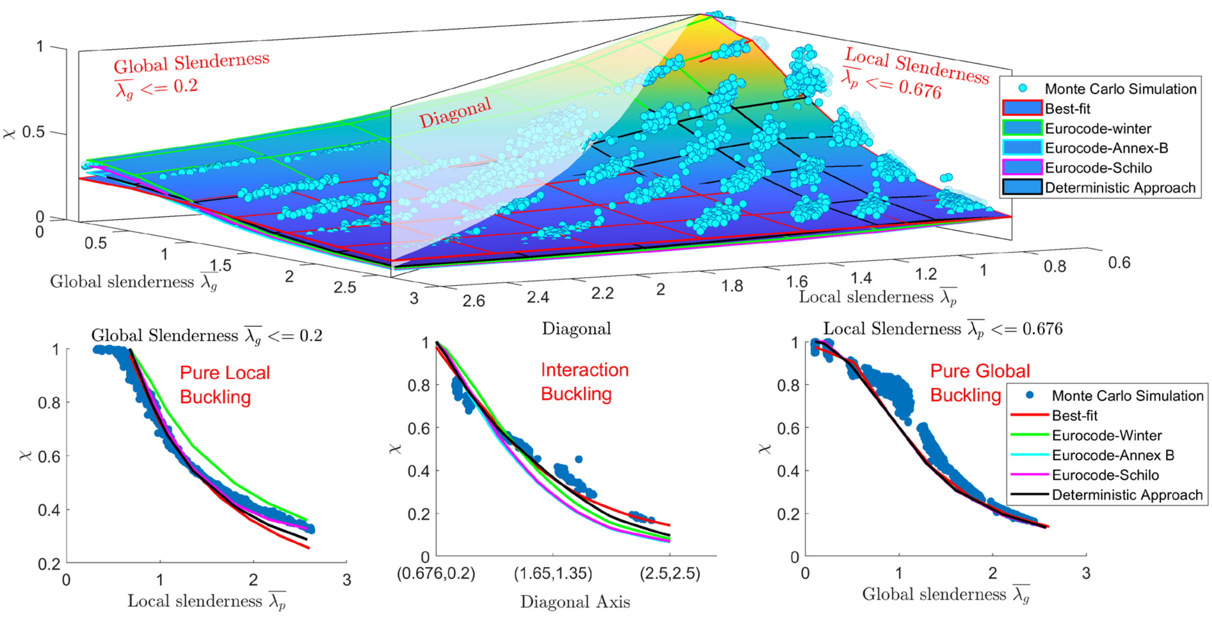

Fig. 3 The performed Monte Carlo simulations and the buckling resistance surfaces using the Eurocode 3 method [1] and the developed solution [5]

These include the Eurocode 3 [1] method, using the currently available local buckling curves, namely Winter-type, Annex B curves of EN 1993-1-5 [2], and Schillo [3] curve. Additionally, a buckling surface is presented using a previously developed deterministic method by the authors [5], along with a newly proposed best-fit surface. The bottom subplots are the cross-sections of the top 3D surfaces, where the first plot shows a cross-section at $\bar{\lambda}_g \leq 0.2$, representing pure local buckling. The second plot shows a diagonal cross-section, representing interaction buckling, and the third plot, at $\bar{\lambda}_h \leq 0.676$, representing pure global buckling. It can be clearly seen that the Eurocode approach using the Winter-curve highly overestimates the local buckling resistance of welded box sections, as was previously shown by different researchers [3]. Also, it overestimates the interaction buckling resistance up to a global slenderness of $\bar{\lambda}_g \leq 1.0$ and underestimate it for higher global slenderness. In a previous investigation [5], the authors investigated this issue and proposed a new solution based on GMNIA results. The developed method, shown in Fig. 3 as the "Deterministic Approach", according to Eqs. (1)–(3), notably yields more accurate estimations of the buckling resistance compared to the Eurocode method along the studied slenderness range for pure local, global, and interaction buckling. As an additional outcome of the stochastic analysis, an alternative simplified formula is developed to accurately estimate the buckling resistance of local, global, and interaction buckling, as depicted in Eq. (5) and shown in Fig. 3 as "Best-fit". The proposed fit is used with the gross cross-sectional area (Agross), as the local buckling effect is implemented in the formula.

$$\chi = \frac{1.45}{0.36 \times \bar{\lambda}_p^2 + \bar{\lambda}_p + 1.60 \times \bar{\lambda}_g^2 - 0.70 \times \bar{\lambda}_g + 0.70} \leq 1.0 \quad (5)$$

A comparison based on the 5% quantile response surface resistance is shown in Fig. 4, where the *x*-axis shows the global slenderness ratio and the *y*-axis shows the obtained buckling capacities. Four groups of buckling resistances are shown that correspond to four different slenderness ratios ($\bar{\lambda}_p$), namely, 0.9, 1.40, 1.80, 2.20. It can be observed that the proposed deterministic solution and the best-fit provide the lower bound resistance for the 5% quantile response surface resistance along the entire global and local slenderness range, providing a safe solution. It is worth mentioning that the deterministic solution and the best-fit yield very close resistances for the middle slenderness range, while small discrepancies occur in the low and high slenderness range. To achieve a more detailed understanding of the achieved improvement of the developed solutions (Deterministic Approach and Best-fit), a comparison is made for the ratio of the obtained buckling resistance and the Eurocode. The ratios are visually represented in Fig. 5, where the changes are a function of the local and global slenderness ratio presented on the *x* and *y* directions. The visual representation of the ratios has several noteworthy observations. Specifically, for pure local and global buckling phenomena, the ratios are less than 1.0, as the Winter curve overestimates the buckling resistance. However, within the interaction domain, substantial improvement is achieved with a maximum difference of 50–70%. It is worth mentioning that the obtained improvements are notably influenced by the global and local slenderness ratios, emphasizing the importance of these parameters in the obtained structural response. Table 3 shows a statistical evaluation of the studied methods. The shown statistical measures are for the ratio of the 5% lower quantile response surface resistance to the resistances of each method. Table 3

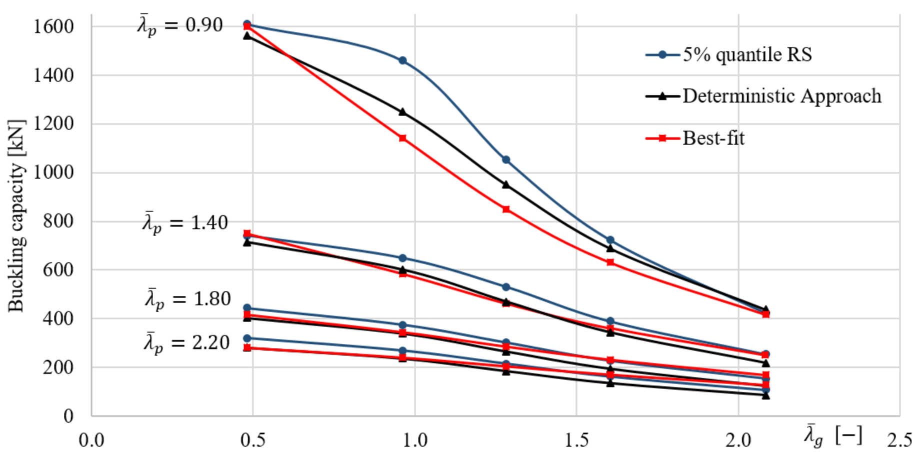

Fig. 4 Comparison of the 5% response surface resistance compared to the resistance obtained using the developed solution and the best-fit curves.

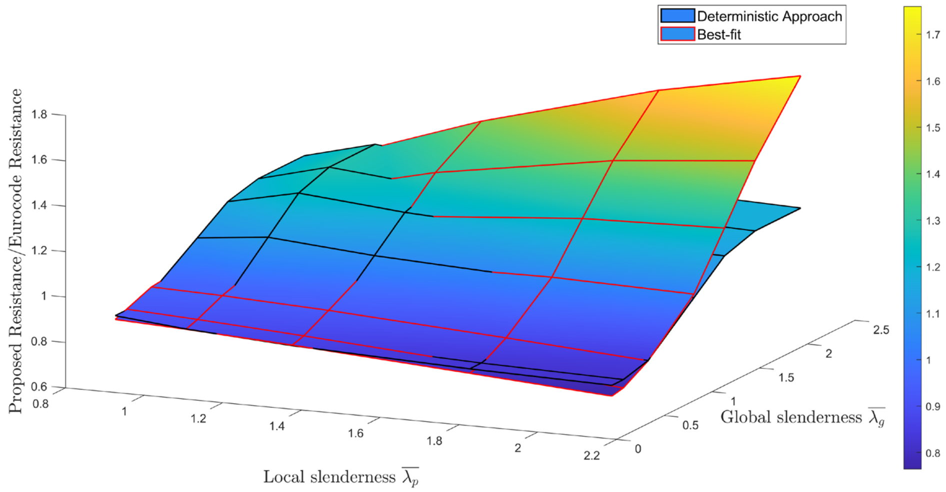

Fig. 5 Relationship between the ratio of Proposed resistances (Developed solution and Best-fit) to Eurocode-based interaction buckling resistance considering Winter curve for local buckling resistance – 3D diagram

Table 3 Statistical evaluation of the studied methods

Method	μ	σ	CoV	Max	Min
Eurocode-Winter	1.18	0.21	0.18	1.49	0.84
Eurocode-Annex B	1.37	0.26	0.19	1.80	0.96
Eurocode-Schillo	1.33	0.25	0.19	1.76	0.93
Deterministic Approach	1.09	0.08	0.07	1.24	0.96
Best-Fit	1.07	0.10	0.09	1.28	0.91

shows the mean (μ), the standard deviation (σ), the coefficient of variation (CoV), the maximum and the minimum of the ratio. It can be observed that the Eurocode method (Eurocode-Winter) shows a very large scatter with a large CoV of 0.18. However, the Deterministic approach and the Best-fit results show a smaller mean value (μ) with a smaller CoV of 0.08 and 0.10, respectively.

6 Conclusions

Several research studies showed that the currently adopted method for estimating the interaction buckling resistance of welded box-section does not take the nonlinear interaction effect into account, leading to highly scattered resistances. In pursuit of an exceptionally high degree of accuracy, this research program employed Monte Carlo simulations that comprehensively account for variabilities stemming from geometrical, material, and imperfection-type uncertainties. The outcomes of these simulations closely mirror the actual buckling resistances observed in experimental tests. This research study began with GMNIA-based Monte Carlo simulations, which were initially conducted to investigate pure local and global buckling behaviors. Subsequently, the investigation was expanded to the assessment of interaction buckling capacity. This approach ensures a thorough understanding of structural performance under various conditions and variations, contributing to achieving highly accurate results. Based on the outcomes of these simulations, a generalized formula for estimating local, global, and interaction buckling was formulated. A comparison was made to the EN 1993-1-1 method [1] for estimating the interaction buckling. Another comparison was made to the previously developed solution by the authors for estimating the interaction buckling based on a deterministic approach. It was found that the Eurocode approach overestimates the interaction buckling resistance of welded box-section columns up to , and underestimate it otherwise. It was also found that the proposed approaches more accurately estimate the buckling resistance of welded box-sections and provide a lower bound to the Monte Carlo simulation results. Therefore, it can be used safely in the design practice.

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