DOI: 10.1002/stco.202200028

Vahid Pourostad, Ulrike Kuhlmann

Buckling resistance of longitudinally stiffened panels with closed stiffeners under direct longitudinal stresses

The buckling behaviour of panels may be determined according to EN 1993-1-5 [1]. Most of the design rules relating to stiffened panels in EN 1993-1-5 were derived on the basis of open-section stiffeners. Several recent investigations have shown that the application of the design rules according to EN 1993-1-5 considering the torsional stiffness of the stiffeners may overestimate the resistance of the panels. Therefore, the recent Amendment A2 to EN 1993-1-5 states that the torsional stiffness of stiffeners should generally be neglected in determining critical plate buckling stresses. In addition, prEN 1993-1-5 [2] provides rules for considering the torsional stiffness of stiffeners. However, in this article it is shown that even the rules of prEN 1993-1-5 are not sufficient to overcome the safety deficiencies. The article focuses on the investigation of the buckling behaviour of stiffened panels with closed-section stiffeners subjected to constant longitudinal compression stresses. Improved rules have been developed that allow to consider the torsional stiffness of the stiffeners. Based on an extensive numerical parametric study, a new interpolation equation between column- and plate-like behaviour is proposed. In comparison to [3], the investigations have been extended to the effective width method. They show that the proposal provides a safe and economic solution for the reduced stress method and the effective width method when considering the torsional stiffness of stiffeners by calculating the critical plate buckling stresses.

Keywords bridge design; plate-like behaviour; column-like behaviour; stiffened panel; longitudinal; reduced stress method; effective width method; torsional stiffness of stiffener

1 Introduction

The use of stiffened panels in modern bridges with large spans is indispensable. To optimise the structural performance and the use of materials, the bridge cross-section may be built as a box girder cross-section where the web and bottom panels are stiffened with closed stiffeners. The closed stiffeners have significant structural advantages. Due to large acting stresses, the panels are usually slender and the slender panels are prone to plate buckling. The buckling resistance of the panels may be determined using EN 1993-1-5 [1]. These design rules were derived on the basis of stiffened panels with open-section stiffeners. This means that the rules are not validated by investigations when the torsional stiffness is considered when calculating the critical plate buckling stresses. Several investigations have meanwhile shown that the application of the design rules according to EN 1993-1-5, considering the torsional stiffness of the stiffeners when calculating the critical plate buckling stresses, may lead to unsafe results. Consequently, the recent Amendment A2 to EN 1993-1-5 [1] specifies that the torsional stiffnesses of the stiffeners should generally be neglected for the determination of the critical plate buckling stresses. Neglecting the favourable effect of the torsional stiffness of the stiffeners may lead to a significant underestimation of the resistance of panels and consequently to an increase in the weight of the steel structures and to an increase in construction costs. The aim of this study is therefore to find a solution to consider the positive effect of torsional stiffeners in the design of stiffened panels with closed-section stiffeners. prEN 1993-1-5 [2] provides rules for considering the torsional stiffness of stiffeners. However, it is shown in the following that even the rules of prEN 1993-1-5 [2] overestimate the resistance of panels.

Slender panels tend to buckle due to high slenderness. To avoid stability failure, EN 1993-1-5 and prEN 1993-1-5 provide two verification methods, namely, the reduction of the cross-section using the effective width method (EWM) or the reduction of the allowable stresses applying the reduced stress method (RSM). The reduction of the cross-section or stresses is determined by means of a reduction factor ρ_c.

In calculating the final reduction factor, plate- and column-like behaviour should be considered. The final reduction factor according to EN 1993-1-5 and prEN 1993-1-5 may be determined with Eq. (1). This equation may be applied to the EWM and the RSM.

$$\rho_c = \chi_c + \left(\rho_p - \chi_c\right) \cdot f \tag{1}$$

where ρ_p is the reduction factor for plate-like behaviour and χ_c is the reduction factor for column-like behaviour. f is the interpolation function between column- and plate-like behaviour determined as follows:

$$f_{EC} = \left(2 - \xi\right) \cdot \xi \tag{2}$$

Subscript "EC" in Eq. (2) indicates interpolation according to Eurocode and refers to EN 1993-1-5 or prEN 1993-1-5. The actual behaviour of panels usually is in between plate- and column-like behaviour. [1] and [2] therefore

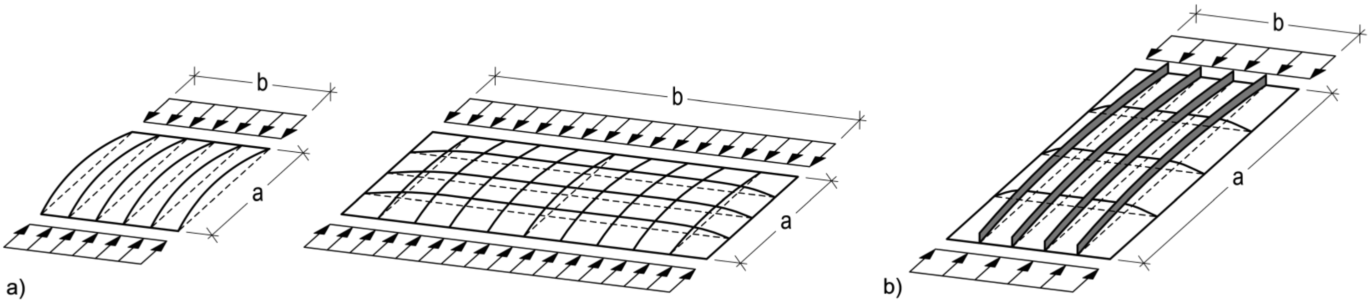

Fig. 1 Column-like behaviour of a) unstiffened and b) stiffened panels [4]

give an interpolation equation between these two kinds of behaviour. The essential parameter to determine the behaviour is the ratio between the critical plate buckling stress $\sigma_{cr,p}$ to the critical column buckling stress $\sigma_{cr,c}$. In this respect, [1] and [2] define a weighting factor as follows:

$$\xi = \frac{\sigma_{cr,p}}{\sigma_{cr,c}} - 1 \qquad 0 \le \xi \le 1 \tag{3}$$

In this formula, ξ equal to zero corresponds to column-like behaviour and ξ equal to one corresponds to plate-like behaviour.

Plate-like behaviour occurs when load transfer occurs in both directions and leads to a failure mode with significant curvature in both directions. In this case, all edges are active for load support. The plate-like behaviour exhibits significant post-critical resistance. In contrast, the plate behaves like a column, when the load transfer is mainly in one direction, i.e., the edges parallel to the load only have little effect on supporting the panel, and the failure mode exhibits significant curvature in only one direction. This behaviour is referred to as a column-like behaviour (see Fig. 1). The expression 'column-like behaviour' refers to buckling cases, where the panel has no post-critical resistance. In stiffened panels subjected to longitudinal compression stresses, the column-like behaviour may occur because the panel is stiffer in the longitudinal direction due to the existing stiffeners, so that the longitudinal edges have little or no influence on the structural capacity of the panel. In these cases, the middle of the panel cannot benefit from the support provided by the longitudinal edges and the panel behaves like a group of adjacent columns.

Torsional stiffness has a significant influence on the critical plate buckling stress $\sigma_{cr,p}$, while it has a rather small influence on critical column buckling stress $\sigma_{cr,c}$, and as a result the standard neglects torsional stiffness in determining critical column buckling stress $\sigma_{cr,c}$. If the torsional stiffness is considered in the calculation of $\sigma_{cr,p}$, ξ increases and there is a nominal shift to plate-like behaviour, which, as shown, may lead to overestimate the resistance of panels.

To consider the torsional stiffness of stiffeners in this study, a new interpolation equation between plate- and column-like behaviour is proposed. This article reflects results of the corresponding author's dissertation [5], where more in-depth investigations are given and which will be published in near future.

2 Literature review

EN 1993-1-5 or prEN 1993-1-5 provides the Effective Width Method (EWM) and the Reduced Stress Method (RSM) for the buckling analysis of slender panels. According to the EWM, after determining the effective widths due to local buckling of sub-panels, the relative panel slenderness of the equivalent plate for global buckling is defined as:

$$\bar{\lambda}_p = \sqrt{\frac{\beta^p_{A,C} \cdot f_y}{\sigma_{cr,p}}} \tag{4}$$

with $\beta^p_{A,C} = \dfrac{A_{c,eff,loc}}{A_c}$

where A_c is the gross area of the compression zone of the stiffened plate except the parts of sub-panels supported by an adjacent plate. $A_{c,eff,loc}$ is the effective area of the same part of the plate with due allowance made for possible plate buckling of subpanels and stiffeners.

$\bar{\lambda}_p$ has to be used to determine the reduction factor for plate buckling ρ_p. The relative column slenderness $\bar{\lambda}_c$ is to be applied for the calculation of χ_c. The relative column slenderness $\bar{\lambda}_c$ is defined as follows:

$$\bar{\lambda}_c = \sqrt{\frac{\beta^c_{A,C} \cdot f_y}{\sigma_{cr,c}}} \tag{5}$$

$$\beta^c_{A,C} = \frac{A_{sl,1,eff}}{A_{sl,1}}$$

Where $A_{sl,1,eff}$ is the effective cross-sectional area of the stiffener and the adjacent parts of the plate with due allowance for plate buckling, and $A_{sl,1}$ is the gross cross-sectional area of the stiffener and the adjacent parts of the plate.

This means that the reduction factors ρ_p and χ_c are to be calculated with two different relative slenderness ratios. The final reduction factor ρ_c is determined considering the interpolation of the behaviour of the panel according to Eq. (1). With the final reduction factor ρ_c, the final effective cross-section area is determined. The verification of direct stresses is then conducted on the basis of the final effective cross-section.

To design a panel according to the RSM in [1] and [2], the load amplification factors $\alpha_{ult,k}$ and α_{cr} are determined using the complete stress state. The factor $\alpha_{ult,k}$ is the smallest factor for increasing the design loads in order to achieve the characteristic value of the resistance at the critical point of the panel and α_{cr} is the load amplifier of the design loads associated to the relevant elastic critical buckling mode under the corresponding complete stress field. It should be noted that the corresponding eigenmodes of the local and global buckling have to be considered. The system slenderness under combined stresses (σ_x, σ_z, τ) can be calculated by means of $\alpha_{ult,k}$ and α_{cr} factors by applying Eq. (6). In case of panel subjected to only direct longitudinal stresses, the slenderness may be determined using Eq. (7).

$$\bar{\lambda}_p = \sqrt{\frac{\alpha_{ult,k}}{\alpha_{cr}}} \quad \text{in general,} \tag{6}$$

$$\bar{\lambda}_p = \sqrt{\frac{f_y}{\sigma_{cr,p}}} \quad \text{if panels subjected to } \sigma_x \text{ only} \tag{7}$$

With $\bar{\lambda}_p$, the reduction coefficients for the plate-like ρ_p and column-like χ_c behaviour with the corresponding buckling curves may be determined. The interpolation between plate- and column-like behaviour must be considered by a weighting factor. Then the final reduction factors for the verification have to be determined according to Eq. (1).

[1] and [2] provide two buckling curves for plate buckling ρ_p. In Section 4 [1] or Section 6 [2], the Winter curve is given. The proposed Winter curve was derived by Winter [6] on the basis of experiments and is formulated as follows:

$$\rho = 1.0 \quad \text{for } \bar{\lambda}_p \le 0.5 + \sqrt{0.085 - 0.055\,\psi} \tag{8}$$

$$\rho = \frac{\bar{\lambda}_p - 0.055(3+\psi)}{\left(\bar{\lambda}_p\right)^2} \quad \text{for } \bar{\lambda}_p > 0.5 + \sqrt{0.085 - 0.055\,\psi} \tag{9}$$

The buckling curve in Annex B [1] or clause 12.4 (5) [2] is known as the general reduction curve for direct and shear stresses and was originally proposed by Müller [7]. Post-critical resistance is considered by this curve; however, the resulting values are more conservative compared to the Winter curve (see Fig. 2). The buckling curves were derived by Müller [7] as a solution of the Ayrton–Perry formulation.

$$\rho = 1.0 \quad \text{for } \bar{\lambda}_p \le \bar{\lambda}_{p0} \tag{10}$$

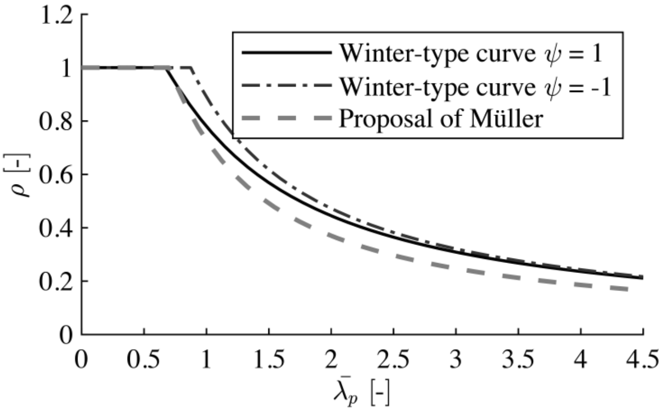

Fig. 2 Comparison of the reduction curves for plate buckling

$$\rho = \frac{1}{\varphi_p + \sqrt{\varphi_p^2 - \bar{\lambda}_p}} \quad \text{for } \bar{\lambda}_p > \bar{\lambda}_{p0} \tag{11}$$

$$\varphi_p = \frac{1}{2}\left(1 + \alpha_p\left(\bar{\lambda}_p - \bar{\lambda}_{p0}\right) + \bar{\lambda}_p\right)$$

According to prEN 1993-1-5, $\bar{\lambda}_{p0} = 0.7$ and $\alpha_p = 0.34$ for welded elements under direct stress $\psi \ge 0$.

Fig. 2 shows both buckling curves for plate buckling according to [1] and [2].

The number of existing investigations, which consider the effect of the torsional stiffness of stiffeners on the buckling behaviour and the design of stiffened panels, is limited. Martin et al. [8, 9] investigated the effects of the torsional stiffness of closed stiffeners on the resistance of longitudinally stiffened panels according to EN 1993-1-5.

In [9], the critical plate buckling stresses are determined using different approaches, like the formula given in Annex A of EN 1993-1-5 [1], which neglects the torsional stiffness of stiffeners. Alternatively, the critical plate buckling stresses are calculated using the software EBPlate [10], modelling the trapezoidal stiffener, which takes the torsional stiffness into account. Finally, the resistances based on these assumptions were numerically compared with geometric and material nonlinear analysis with imperfections (GMNIA) using finite element method (FEM). In this study, it was concluded that if the torsional stiffness of the closed-section stiffeners is considered to calculate the critical plate buckling stress $\sigma_{cr,p}$ (using advanced tools such as finite element software or EBPlate), the load capacity of the panel under pure compression may be overestimated according to the verification given in EN 1993-1-5. In this study, it is concluded that advanced tools such as finite element software or EBPlate can still be used to assess $\sigma_{cr,p}$ when applying the EWM of EN 1993-1-5, provided that the torsional stiffness of closed cross-section stiffeners is neglected.

Kövesdi et al. [11, 12] also investigated the buckling behaviour of stiffened panels with closed-section longitudinal stiffeners subjected to pure constant compression numerically. A study on the column-like buckling was conducted, and it was concluded that the influence of torsional stiffness of the stiffeners on the column-like behaviour is negligibly small. It was also concluded that for the calculation of the critical column buckling stress the formula of EN 1993-1-5 gives sufficiently good results. Also, if the design approach of EN 1993-1-5 is applied which means determination of the critical plate buckling stresses using the given formula in EN 1993-1-5, Annex A and calculation of the interaction behaviour according to current EN 1993-1-5, the predicted resistance is safe, but not economical in many cases. Kövesdi [11] proposed to improve the design procedure assuming that the critical plate buckling stresses should be determined using a finite element model to consider the torsional stiffness of stiffeners and that the critical column buckling stress may be calculated using the formula of EN 1993-1-5. To determine the reduction factor for column-like behaviour, the current rules do not need to be changed. However, the reduction factor for plate buckling should be determine using the curve by Müller, which is given in Annex B of EN 1993-1-5 [1] or Section 12.4 (5) of prEN 1993-1-5 [2].

Kövesdi also proposed a new formula as interpolation between column- and plate-like behaviour for the calculation of the final reduction factor to consider the positive effect of torsional stiffness of closed stiffener in design. The proposal of Kövesdi for the interpolation function is given in Eqs. (12) and (13) and should be used as f in Eq. (1).

$$f_{\text{Kövesdi}} = \left(\frac{\sigma_{\text{cr,p}}}{\sigma_{\text{cr,c}}} - 1\right) \cdot 0.6 \qquad \text{if } \frac{\sigma_{\text{cr,p}}}{\sigma_{\text{cr,c}}} \leq 2.0 \tag{12}$$

$$f_{\text{Kövesdi}} = \left(\ln\left(\frac{\sigma_{\text{cr,p}}}{\sigma_{\text{cr,c}}}\right)^{2.2}\right)^{0.455} \cdot 0.5 \quad \text{if } \frac{\sigma_{\text{cr,p}}}{\sigma_{\text{cr,c}}} > 2.0 \tag{13}$$

3 Numerical model

3.1 General

The model was developed in ABAQUS [13] using a Python script that can be used to create numerical models with different parameters and for different load cases. The direct stresses were applied constantly in longitudinal directions. The same assumptions apply for the numerical model as for the recalculation of the tests.

First, a linear plate buckling analysis (LBA) was performed to obtain the buckling mode for the local imperfections and the critical plate buckling stress for the global buckling verification. Subsequently, a GMNIA was conducted. Both calculations were done with the finite element software ABAQUS. For the modelling of panels in ABAQUS, shell elements of type S4R were chosen [13].

A distinction was made in the modelling between the GMNIA and the LBA investigation. In the GMNIA inves-

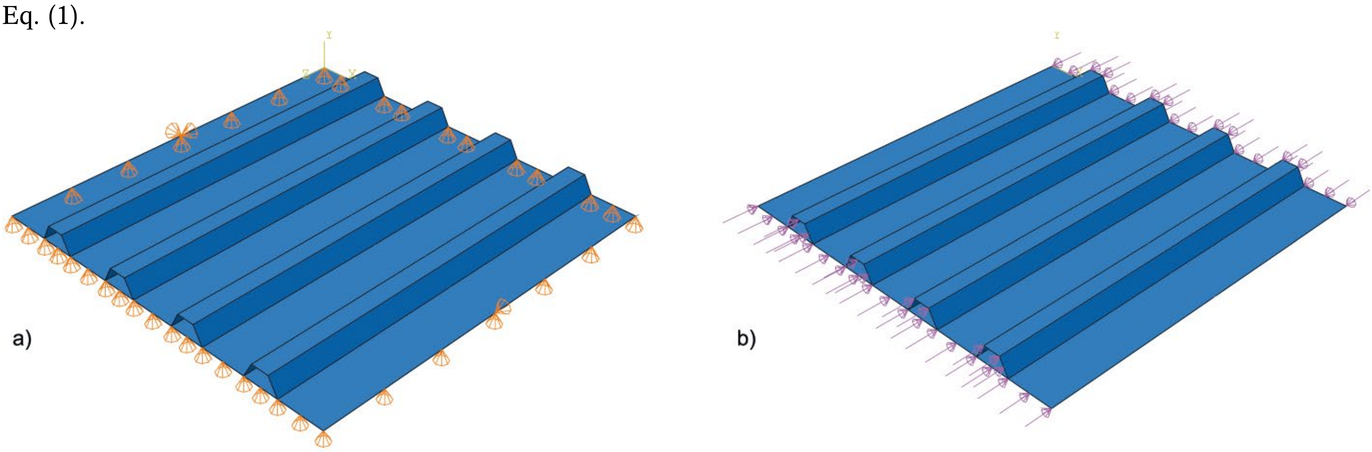

Fig. 3 Numerical model for LBA: a) boundary conditions and b) loading

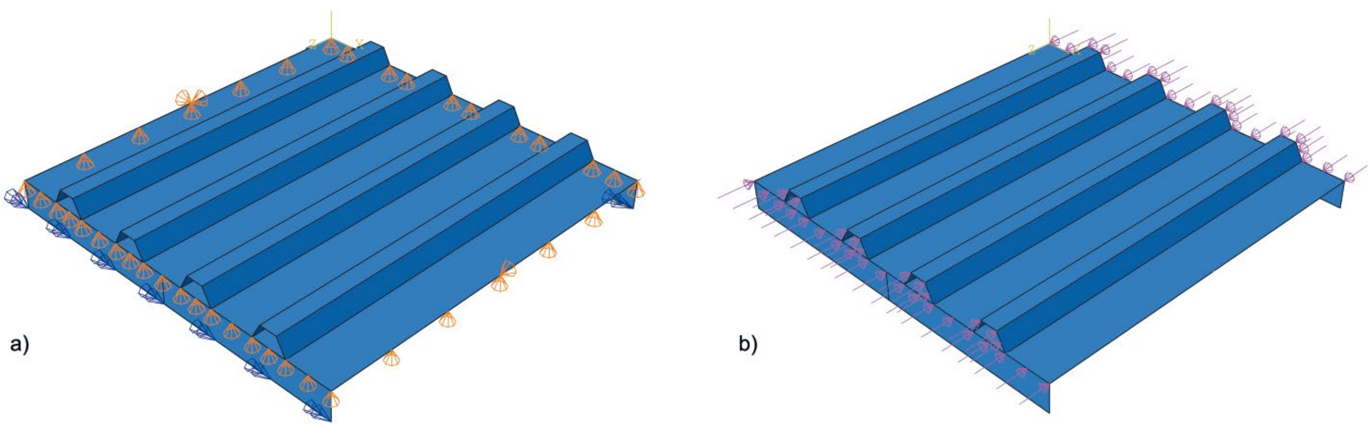

Fig. 4 Numerical model for GMNIA: a) boundary conditions and b) loading

tigation, transverse stiffeners were additionally modelled along both panel ends to give a membrane restraint. Fig. 3 shows the simulated boundary conditions and the schematic loading for LBA and Fig. 4 for the GMNIA calculation.

Fig. 4 shows the arrangement of the transverse stiffeners, which were only modelled below the plate. The lack of contact with the longitudinal stiffeners is intentional to prevent a favourable influence of the transverse stiffener on the longitudinal stiffeners. In addition, the transverse stiffener is hinged to the panel to allow rotation of the panel and to provide a hinged or simply supported boundary condition. The rotation of the lower edge of the transverse stiffeners was supported due to avoid instability of transverse stiffeners.

3.2 Imperfections

Zizza [14] has investigated possible interpretations of applying global imperfections in the case of panels with more than one stiffener (see Fig. 5). Zizza [14] assumes that equal amplitude between the longitudinal stiffeners of imperfection (interpretation 1 in Fig. 5) gives the most realistic and less advantageous imperfection shape. Regarding the behaviour and failure of the plate under pure compression stresses, it should be noted that interpretation 1 with equal amplitude of stiffeners corresponds to column-like behaviour and interpretation 3 corresponds to plate-like behaviour. As the buckling behaviour of stiffened panels is expected to be predominantly column-like behaviour or interaction between column- and plate-like behaviour, global imperfections according to interpretation 1 were applied in this study to be safe and realistic as possible. If there were more than one stiffener, all stiffeners were deformed with the same amplitude min(a, b)/400 of imperfections as initial imperfection according to prEN 1993-1-14 [15] or Annex C EN 1993-1-5 [1]. It should be noted that in the parametric study the global imperfections were applied in both directions.

Tab. 1 Combination of imperfections

Number	Combination
1	Local (LBA)
(2) and [9]	Global (p) [n]
(3) and [10]	Global (p) [n]+ 0.7 local (1 sine half-waves)
(4) and [11]	Global (p) [n]+ 0.7 local (3 sine half-waves)
(5) and [12]	Global (p) [n]+ 0.7 local (LBA)
(6) and [13]	0.7 Global (p) [n]+ local (1 sine half-waves)
(7) and [14]	0.7 Global (p) [n]+ local (3 sine half-waves)
(8) and [15]	0.7 Global (p) [n]+ local (LBA)

( ) Refers to the global imperfection in the direction of the stiffeners (positive direction); [ ] refers to the global imperfection in the opposite direction of the stiffeners (negative direction)

Two different imperfection approaches according to EN 1993-1-5 Annex C [1] were applied as local imperfections. First, the local imperfections between the stiffeners were applied using a sine function. The number of waves of sub-panel was 1 and 3 sine half-waves. In the second approach of the local imperfections, the buckling mode obtained from the LBA was applied (see Fig. 7). The first buckling modes of each sub-panel were found. These were then combined and applied as local imperfections. Fig. 6 shows the selected buckling mode for a stiffened panel as an example. Fig. 7 illustrates in an exaggerated way the superposition of the buckling modes and the local imperfection modes used in the numerical models. For the amplitude of the imperfection, the value of min(a, b_i)/200 as given in prEN 1993-1-14 was applied, where b_i is the width of sub-panel i. The global and local imperfections were then combined according to Tab. 1. This led to 15 different combinations of imperfections. A

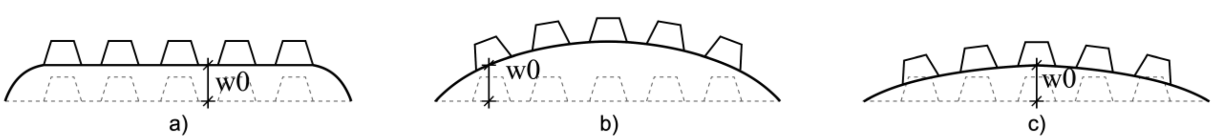

Fig. 5 Possible interpretations of applying global imperfections based on [14]: a) interpretation 1; b) interpretation 2; c) interpretation 3

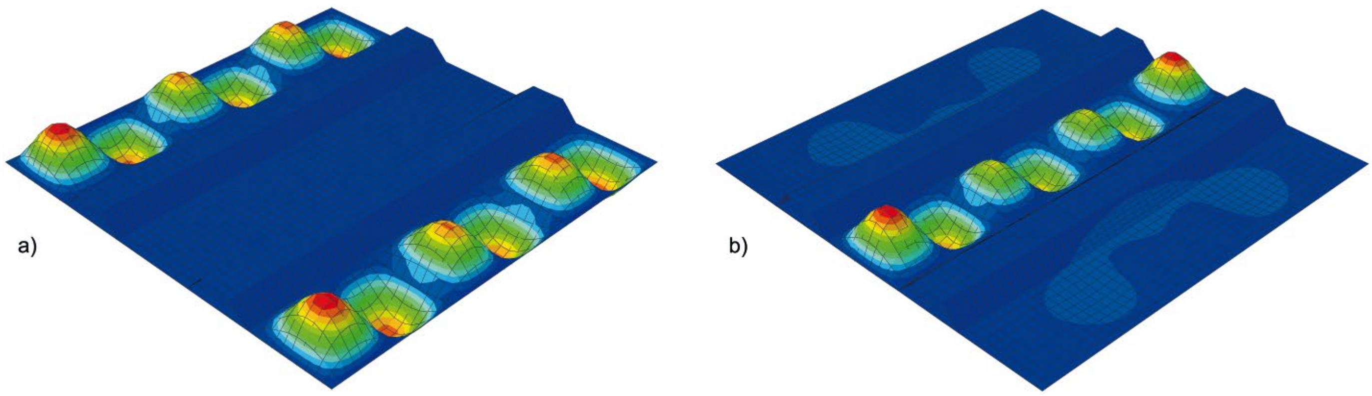

Fig. 6 Selected buckling modes for local buckling of the stiffened bottom plate under constant compression in longitudinal and transverse direction ($\alpha = 1$; $b_{global}/t = 180$; $\gamma = 80$): a) buckling mode 1 (local); b) buckling mode 12 (local)

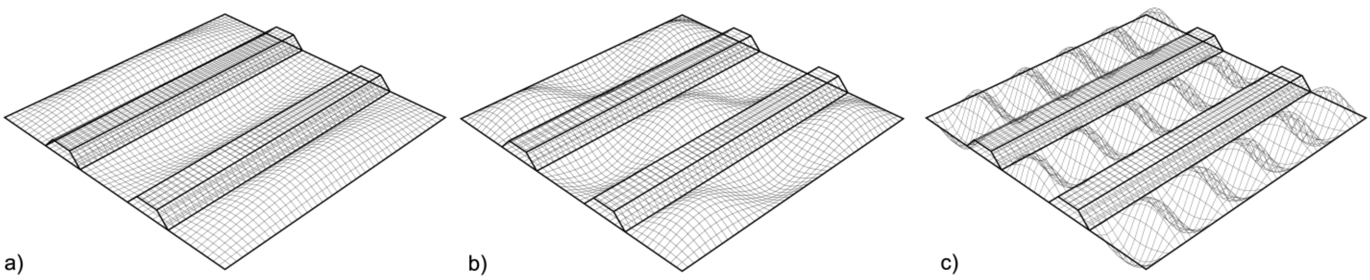

Fig. 7 Applied imperfections for local buckling of the stiffened bottom plate under constant compression in longitudinal and transverse direction ($\alpha = 1$; $b_{global}/t = 180$; $\gamma^*_{sl,i} = 80$) – (imperfection increased by a factor 75): a) 1 sine half-waves; b) 3 sine half-waves; c) local (LBA): superposition of buckling modes 1 and 12

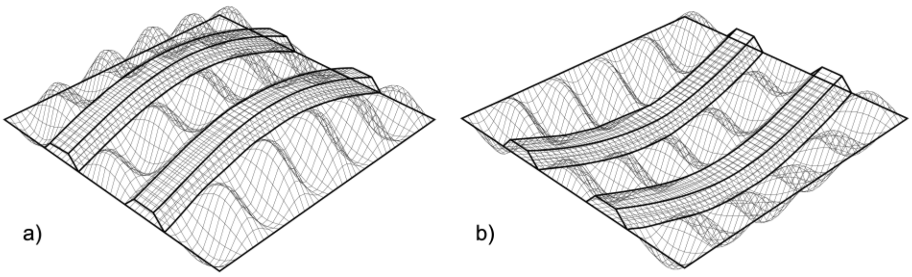

Fig. 8 Combinations of the imperfections of the stiffened buckling panels under constant compression in longitudinal and transverse direction – (imperfections increased by a factor 75): a) combination number (8) positive; b) combination number [15] negative

reduction of 70% of the accompanying imperfections according to prEN 1993-1-14 was considered in this investigation. As a global imperfection, the equivalent geometric imperfection was applied in the direction of the stiffeners and in the opposite direction of the stiffeners. In this investigation, the imperfection in the direction of the stiffeners was denoted as positive global imperfection and in the opposite direction as negative global imperfection. In Tab. 1, the round brackets refer to positive global imperfection and the square brackets refer to negative global imperfection. Fig. 8 shows schematically the imperfection combination 8 and 15 for the example of $\alpha = 1$; $b_{global}/t = 180$; $\gamma^*_{sl,i} = 80$ under pure longitudinal compression.

It should be noted that no residual stresses were simulated in this study and it was assumed that this type of imperfection is covered by the equivalent geometric imperfection. This assumption has been confirmed by the recalculation of the tests. The tests were numerically recalculated using the imperfection amplitude according to prEN 1993-1-14 [15] and it was concluded that the residual stresses can be neglected if the amplitude according to prEN 1993-1-14 [15] is applied and as global imperfection both directions of imperfection are considered.

3.3 Material

The steel grade S355 with yield strength 355 MPa was applied in the parametric study. A bilinear material model without strain hardening was chosen for the material properties according to prEN 1993-1-14 [15]. A modulus of elasticity of 210,000 N/mm^2 and a Poisson's ratio of 0.3 were assumed. The gradient in the plastic range was set at E/10,000 to avoid numerical problems.

3.4 Validation and verification

Twelve tests on web panel subjected to interaction of bending and patch loading were conducted within the research projects [16] and [17] at Technical University of Munich with cooperation of University of Stuttgart. The tests conducted within the framework of the research project [16] are indicated with TH and the tests from [17] with D. Eleven tests were recalculated and the numerical model was validated by this recalculation. The recalculation of the experimental tests shows good agreement with the experimental results for the ultimate load and the failure mode. The deviations between numerical and experimental results are summarised in Tab. 2.

The failure mode of the test is shown as an example in Fig. 9. It should be mentioned that due to the complexity of the test procedure and particularly the biaxial loading, the numerical result obtained can be considered acceptable. Details of this validation of model can be found in [16] and [17]. The numerical model was also validated by recalculating tests on web plates. The web plates were subjected to bending and one-sided transverse stresses, while the bottom plates were subjected to constant stresses.

Tab. 2 Deviations of the ultimate load in transverse direction [5]

Test	Deviation [%]	Test	Deviation [%]
D1	19	TH1	2
D2	15	TH3	8
D3	10	TH4	1
D4	4	TH5	2
D5	4	TH6	4
D6	–12	**Mean value**	**5**

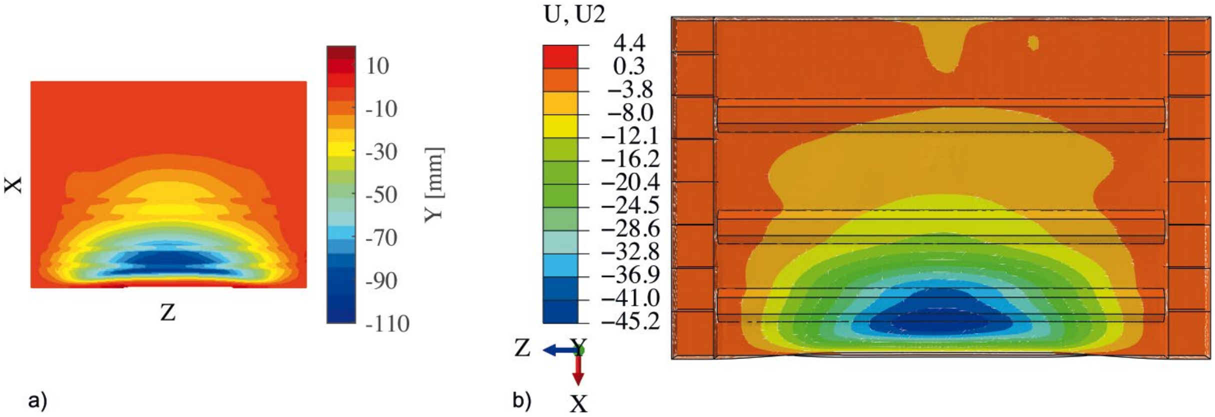

Fig. 9 Failure modes after reaching ultimate load: a) D6 (test); b) D6 (FE)

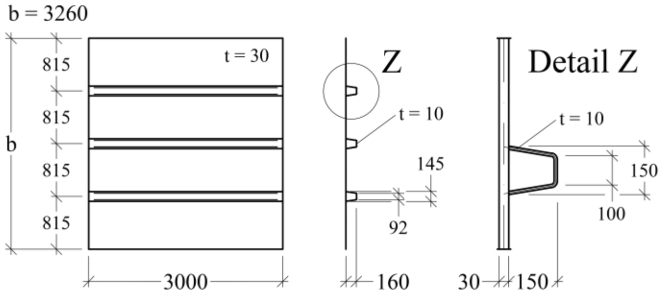

Fig. 10 Bottom plate – example 2 in [18]: a) system lines; b) stiffener detail

In addition, the model is verified by comparison with a detailed numerical analysis of a stiffened bottom plate with closed cross-section as published in [18]. The example concerns a stiffened panel with closed-section stiffeners under pure longitudinal stresses, which was calculated including all verifications according to EN 1993-1-5, namely, the Effective Width Method (EWM), the Reduced Stress Method (RSM) and the finite element method (FEM). In this example, LBA and GMNIA were also performed. This allows to verify the model with both types of analyses. The geometry of the panel is shown in Fig. 10. The imperfections were applied as described in Section 3.2. The comparison between the GMNIA analyses thus gives a test of the assumptions of imperfections.

The corresponding buckling values from [18] were compared with determined buckling values from own calculations and are listed in Tab. 3. The comparison allows the conclusion that the model is sufficiently validated for the linear buckling analysis (LBA).

Tab. 3 Comparison of buckling values from own calculations with Timmers et al. [18]

Buckling mode	**α_{cr} Own calculation [–]**	**α_{cr} Timmers [18] [–]**	**Deviation [%]**
1	1.701	1.712	–0.67
9	5.650	5.744	–1.64
12	5.979	6.075	–1.57
18	6.757	6.888	–1.90
20	7.214	7.307	–1.28
21	7.260	7.385	–1.83

The lowest calculated ultimate stresses resulted from the minimum stresses from combinations 1 to 8, which correspond to the positive imperfections, and were compared with the presented results in [3, 18]. The comparison shows that the imperfection combinations lead to similar and slightly safer results than the example in [18]. The comparison of the results confirms that the model and the assumptions of the imperfection combinations for the bottom plate are also verified and may be used for the parametric study.

3.5 Parameter range

The parametric study was conducted under variation of the following input variables:

- Slenderness of panels: b/t(global) = from 22 to 533
- Width: $b = (n_{st} + 1) \cdot 400 + n_{st} \cdot 300 \rightarrow b = 1100; 1800; 3200$ [mm]
- Aspect ratios of panels: α (global = a/b = 1.0; 1.5; 2.0; 3.0
- Relative bending stiffness of the stiffeners $\gamma^*_{sl,i}$ = 25; 50; 80; 110; 150
- Number of stiffeners: n_{st} = 1; 2; 4
- For the loading, the yield stress of the steel f_y = 355 MPa was applied on the edges of the plate and the stiffeners in the longitudinal direction.

The relative bending stiffness of the stiffeners $\gamma^*_{sl,i}$ is defined according to prEN 1993-1-5 [2] by:

$$\gamma^*_{sl,i} = \frac{E\, I^*_{sl,i}}{b\, D} \tag{14}$$

where E is the young's modulus and D is the bending stiffness of the plate. $I^*_{sl,i}$ is the second moment of area of the stiffener for out-of-plane bending, its cross-section including a participating width of the plate of $10 \cdot t$ each side of stiffener-to-plate junction.

The panels were stiffened with closed trapezoidal stiffeners. To vary the relative stiffness of the stiffeners, the lower and upper widths of the trapezoidal stiffeners were kept constant at 300 and 150 mm, respectively, and the

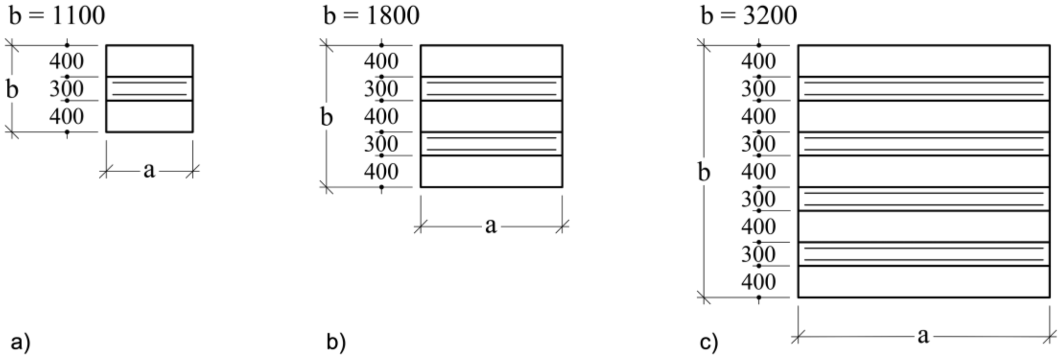

Fig. 11 Dimensions and arrangement of the stiffeners in panels in the parametric study: a) 1 stiffener ($n_{st} = 1$); b) 2 stiffeners ($n_{st} = 2$); c) 4 stiffeners ($n_{st} = 4$)

thickness and height of the trapezoidal stiffeners were varied. The stiffeners were arranged so that the width of all sub-panels equalled each other. The geometry and the arrangement of the stiffeners of panels with one, two and four stiffeners are shown in Fig. 11.

4 Evaluation and development of a design approach

4.1 General

Different design approaches were developed to determine the resistance of longitudinal stiffened panels. In this article, the results from the numerical parametric study were compared with the resistances determined from different design approaches. The aim was to develop a reliable and economical design procedure to determine the resistance of the stiffened panels with closed stiffeners. For this purpose, the torsional stiffness of the longitudinal stiffener was to be considered. This investigation is based on the RSM and EWM according to prEN 1993-1-5 [2] (which is principally the same procedure as EN 1993-1-5 [1]).

Thereby, the longitudinally stiffened panels under direct stresses have been investigated in order to develop a new interpolation equation between plate- and column-like behaviour which allows to calculate the critical plate buckling stress considering the torsional stiffness of the stiffener, e.g. by FEM.

The reduction factor for column-like behaviour χ_c was determined according to prEN 1993-1-5 [2] in all design approaches described below, whereby prEN 1993-1-5 refers to prEN 1993-1-1 [19].

For all design approaches, the local and global verifications were verified separately. The reduction factor for the verification of sub-panels was determined according to the Winter curve.

The design approaches for the investigation of panels under longitudinal stresses are designated 'X N', where X indicates that the panels were only stressed under longitudinal stresses and N represents the number of the design approach. Tab. 4 summarises the differences between the various design approaches.

The same load was applied on the numerical model and the model of the design approaches and then the loading factors to achieve the resistance of panel were determined. As panels are loaded with the yield stress of the

Tab. 4 Design approaches for the verification of stiffened panels subjected to longitudinal stresses

Approach	α_{cr} (global)	$\rho_{p,x}$	Interpolation $\rho_{c,x}$	Method
X 1	Annex A	Winter	EN 1993-1-5	RSM
X 2	LBA	Winter	EN 1993-1-5	RSM
X 3	LBA	Müller	EN 1993-1-5	RSM
X 4	LBA	Winter	Proposal	RSM
X 5	LBA	Müller	Proposal	RSM
X 6	LBA	Winter: $\alpha < 2$ Müller: $\alpha \geq 2$	Proposal	RSM
X 7	LBA	Müller	EN 1993-1-5	EWM
X 8	LBA	Müller	Proposal	EWM
X 9	LBA	Winter: $\alpha < 2$ Müller: $\alpha \geq 2$	Proposal	EWM

steel f_y = 355 MPa, the final reduction factor is the same as the loading factor. So, the load factors or the final reduction factor are calculated as follows:

$$\rho_c = \frac{N_{ult}}{A \cdot f_y} = \frac{\sigma_{ult}}{f_y} \quad (15)$$

where N_{ult} and σ_{ult} are the determined ultimate resistance of the panel.

In this article, the numerically determined ultimate resistances are compared with the determined ultimate resistances of panels, applying different approaches based on the RSM and the EWM to evaluate the results of each of the approaches. In the following, the loading factors are also designated as r_t and r_e according to the definitions of EN 1990 [20], where r_t indicates the loading factor of the design approach as a theoretical model and r_e the loading factor from the numerical simulations as experimental results. r_t is additionally supplemented in the designation with loading type and number of the design approach. Thus, the loading factors are given as $r_{t,XN}$.

For each design approach, two diagrams are shown. The left diagram shows a direct comparison between the loading factors. The right histogram represents the frequency distribution of the ratio r_e/r_t. In the histogram diagrams, μ represents the mean value of the results and σ the standard deviation. The mean value minus and plus the standard deviation are also given, in order to make the quality of the design approaches more obvious. It should be noted that all calculations and verifications were conducted with a γ_{M1} of 1.0.

4.2 Design approach X 1

The design approach X 1 corresponds to the currently valid verification procedure according to EN 1993-1-5 [2] taking Amendment A2 into account. The torsional stiffness of the stiffeners must be neglected in the calculation of the critical buckling stresses according to Amendment A2. To neglect the torsional stiffness, the critical buckling stress according to EN 1993-1-5 Annex A was determined in the design approach X 1.

The formulation in Annex A only considers the flexural stiffness of the longitudinal stiffeners and neglects the torsional stiffness of the longitudinal stiffeners. In this design approach, the reduction factor of plate-like behaviour is determined according to the Winter curve in EN 1993-1-5 Section 4 or prEN 1993-1-5 Section 6. The interpolation between column- and plate-like was determined according to EN 1993-1-5. Fig. 12a compares the results of this design approach with the numerical results. From the comparison it can be seen that design approach X 1 gives very conservative results due to the neglect of the torsional stiffness of the stiffener when calculating the critical plate buckling stress.

4.3 Design approach X 2

Design approach X 2 is comparable to design approach X 1, with the difference that the critical plate buckling stresses are determined with ABAQUS using a LBA. Thus, the torsional stiffness is considered in this approach. Most of the results obtained with this design approach were found to lie on the unsafe side (see Fig. 12b). This shows clearly that the application of the Winter curve with the interpolation equation according to EN 1993-1-5 leads to unreliable results.

4.4 Design approach X 3

Design approach X 3 corresponds to design approach X 2, but the reduction factor of the plate-like behaviour was determined according to curve of Müller in Annex B of EN 1993-1-5 or prEN 1993-1-5 para. 12.4 (5). It should be noted that this design approach corresponds to the procedure according to prEN 1993-1-5 in case of considering the torsional stiffness of stiffeners. The results of this design approach are shown in Fig. 12c. Still many results of X 3 are on the unsafe side. Therefore, the application of the interpolation equation according to EN 1993-1-5, considering the torsional stiffness when calculating the critical plate buckling stress, still leads to unsatisfying reliability.

4.5 Design approach X 4

As the design approach X 3, considering the torsional stiffness of the longitudinal stiffeners and using the more conservative plate buckling reduction curve of Müller still show unsafe results, a new interpolation function between plate- and column-like behaviour has been developed [5].

Based on proposal of Seitz [21], a new interpolation equation was proposed for unstiffened panels in [17]. The preceding interpolation equation has further been developed for stiffened panel with closed stiffeners [5]. The developed interpolation function is given in Eq. (16).

$$f = V \cdot \left(\ln \left(\frac{\sigma_{cr,p}}{\sigma_{cr,c}} \right) \right)^P \qquad 0 \le f \le 1 \quad (16)$$

The parameter f is defined as the function between column- and plate-like behaviour. f can be between zero and one. f equal to zero corresponds to column-like behaviour and f equal to one corresponds to plate-like behaviour. To determine the final reduction factor, this f function should be inserted into Eq. (1).

The proposed interpolation equation may be applied by modifying the parameters V and P for different boundary conditions and loadings. In this article, the focus is on stiffened panels with closed cross-section subjected to constant longitudinal stress, so the equation is given for this case. In [5], the parameters for other cases are investi-

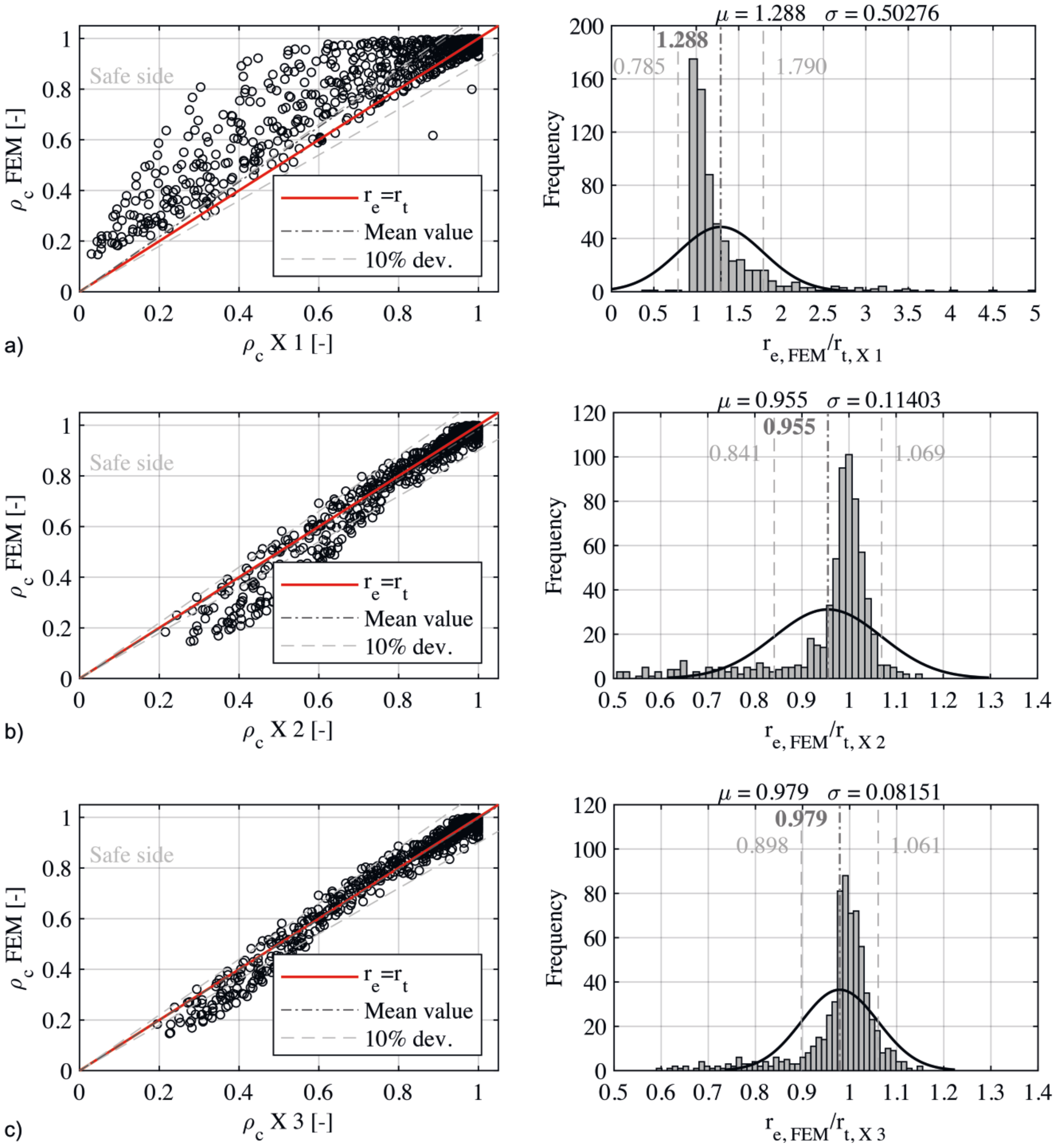

Fig. 12 Comparison of the numerical results with the results of the design approaches: a) X 1, b) X 2 and c) X 3 (left: direct comparison; right: frequency distribution)

gated and given. Based on a recommendation by Balász Kövesdi and additional investigations, the formula given in [2] has been slightly modified. In the case of a global buckling analysis of stiffened panels with closed cross-section under longitudinal stresses, the parameters *V* and *P* are defined as:

$$V = \left(\bar{\lambda}_p + 1\right)^{-2/3} \text{ and } P = 1.0 \quad (17)$$

Application of this function allows the torsional stiffness to be considered when calculating the critical plate buckling stress using an LBA. In design approach X 4, the reduction factor of the plate-like behaviour ρ_p is determined according to the Winter curve and the proposed interpolation function Eq. (16) is used. Compared to design approaches X 2 and X 3, this design approach X 4 gives better results, but in some cases still unsafe results occur (see Fig. 13a).

4.6 Design approach X 5

Design approach X 5 corresponds to design approach X 4 with the difference that the reduction factor of plate buckling is determined according to the curve of Müller in Annex B of EN 1993-1-5 or in prEN 1993-1-5 para. 12.4 (5). Fig. 13b shows that the design approach X 5 gives very good results which agree with the numerical results. This design approach shows that the torsional stiffness of the longitudinal stiffeners may be considered when the critical buckling stress is calculated, if the curve of Müller in Annex B of EN 1993-1-5 or para. 12.4 (5) of prEN 1993-1-5 is applied for the plate buckling reduction factor and the new proposed interpolation Eq. (16) is used. The disadvantage of this design approach is the neglect of the positive effect of the stress ratio ψ, e.g. in the case of panels subjected to bending, whereas the Winter curve takes the influence of the stress ratio into account.

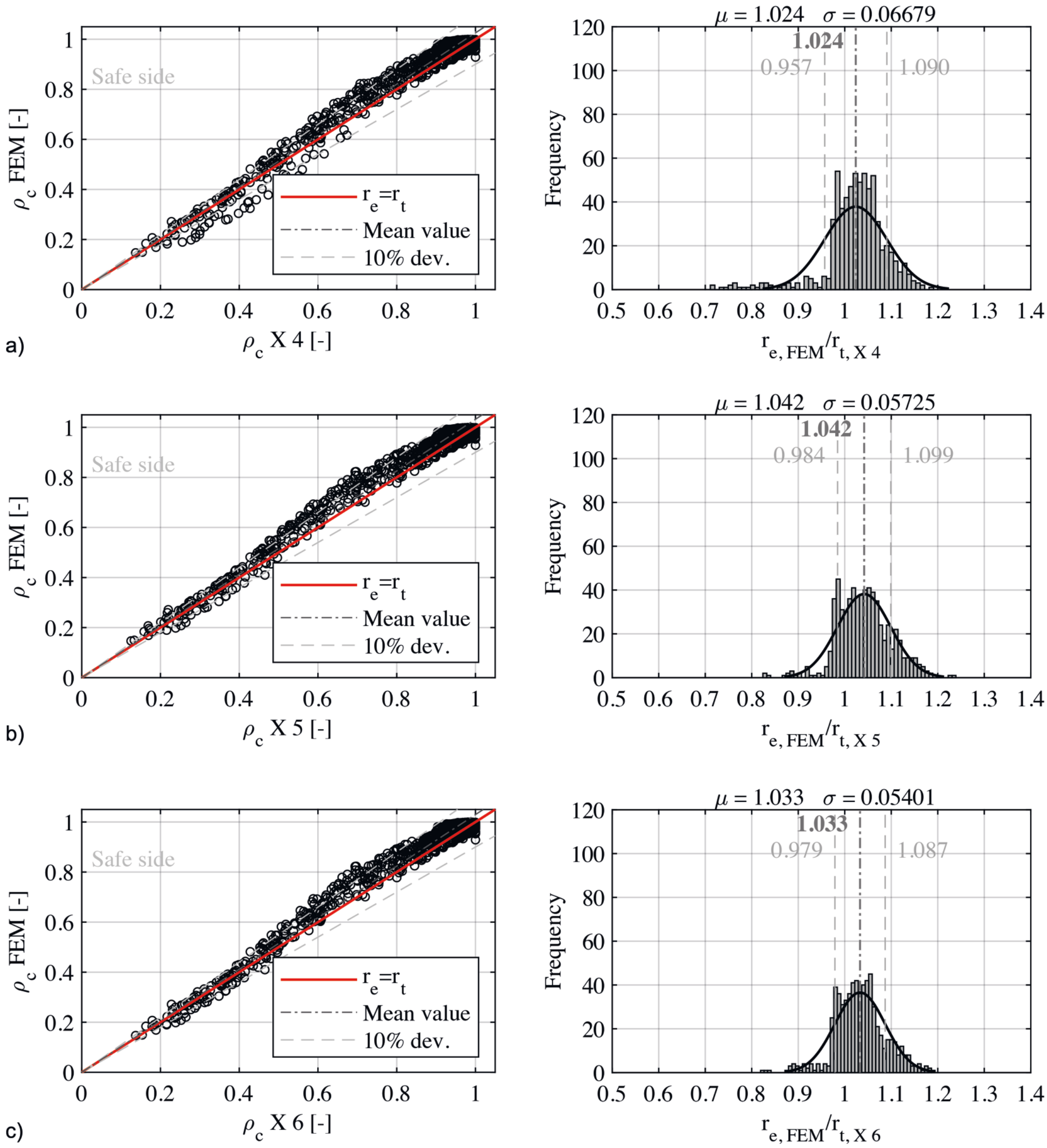

Fig. 13 Comparison of the numerical results with the results of the design approaches: a) X 4, b) X 5 and c) X 6 (left: direct comparison; right: frequency distribution)

4.7 Design approach X 6

The design approach X 4 has been analysed in detail. Fig. 14a shows the results of the design approach X 4 depending on the aspect ratio α. It can be seen that unsafe results only occur for aspect ratios greater than 2, when the Winter curve is applied with the new proposed interpolation Eq. (16). For this reason, the application of the Winter curve for aspect ratios equal to or greater than 2 was excluded in the design approach X 6 and the reduction curve according to Müller in Annex B of EN 1993-1-5 or 12.4 (5) of prEN 1993-1-5 was applied instead. For aspect ratios smaller than 2, the Winter curve was used. Fig. 14b shows the corresponding results for these cases using the design approach X 6.

In Fig. 13c, all results of the design approach X 6 were compared with the numerical results. The comparison showed that the torsional stiffness of the longitudinal stiffener may be considered when calculating the critical plate buckling stress by applying the new proposed interpolation equation between column- and plate-like behaviour. The Winter curve for aspect ratios smaller than 2 should be used as the reduction factor for plate buckling; for the other cases, the reduction curve according to Müller in Annex B of EN 1993-1-5 or 12.4 (5) prEN 1993-1-5 has to be applied.

4.8 Design approach X 7

Design approach X 7 corresponds to design approach X 3 with the difference that the EWM is used to determine the resistance of panels. The design approach X 7 fulfils the rules given in prEN 1993-1-5 considering the torsional stiffness of the closed stiffeners when calculating the critical plate buckling stresses. In this design approach, the interpolation equation according to

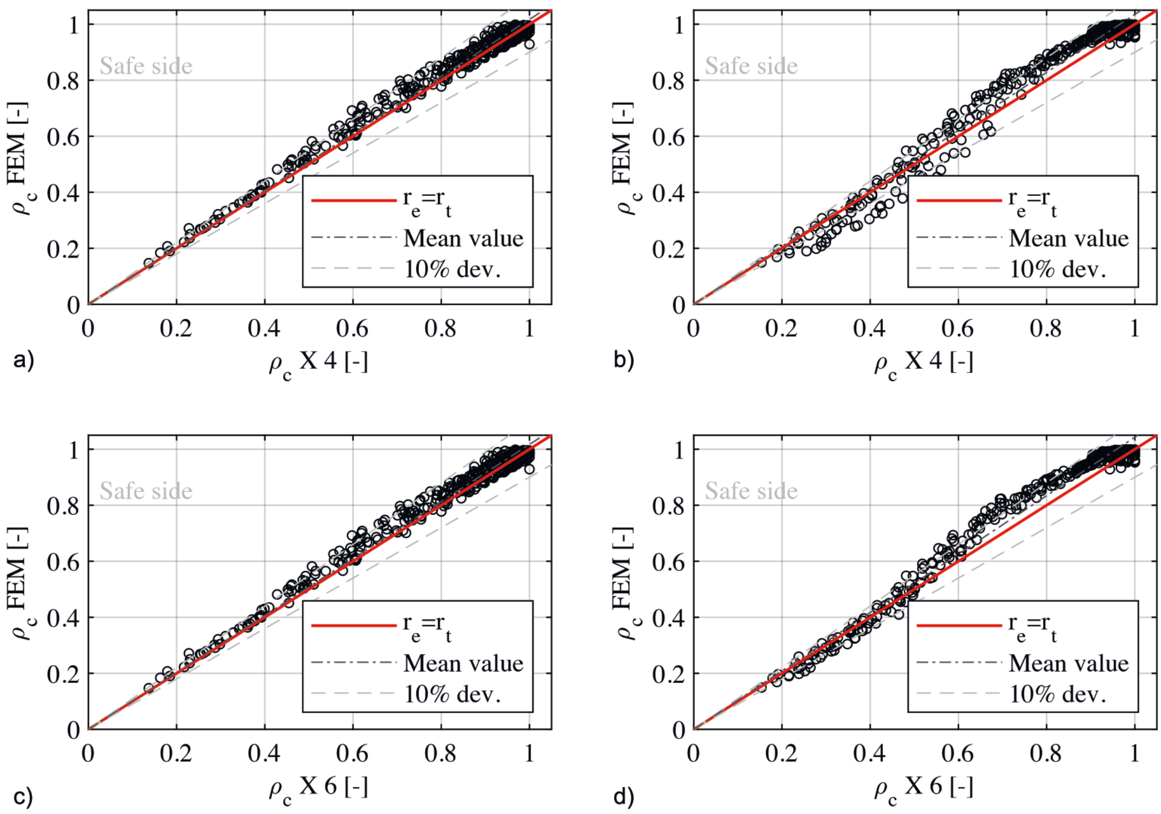

Fig. 14 Comparison of the numerical results with the results of the design approaches X 4 and X 6 in dependence on the aspect ratios: a) design approach X 4: $\alpha = 1$; 1.5; b) design approach X 4: $\alpha = 2$; 3; c) design approach X 6: $\alpha = 1$; 1.5; d) design approach X 6: $\alpha = 2$; 3

EN 1993-1-5 and the reduction curve according to Müller are used to determine the reduction factor for the plate-like behaviour.

The results of this design approach are shown in Fig. 15a. There are many results obtained with X 7 that are still on the unsafe side. Hence, using the Effective Width Method (EWM) and the interpolation equation according to EN 1993-1-5, considering the torsional stiffness when calculating the critical plate buckling stress, still leads to unsatisfactory reliability.

4.9 Design approach X 8

The design approach X 8 corresponds to the design approach X 5 with the difference that the EWM is used to determine the resistance of the panels.

Fig. 15b shows that the design approach X 8, analogous to the design approach X 5, gives very good results that agree with the numerical results. This design approach shows that the torsional stiffness of the longitudinal stiffeners can be considered in the calculation of the critical plate buckling stresses if the curve of Müller is used as a reduction factor for plate buckling and the proposed interpolation Eq. (16) is applied. As already mentioned, the disadvantage of this design approach is the neglect of the positive influence of the stress ratio ψ.

4.10 Design approach X 9

The design approach X 9 corresponds to the design approach X 6 with the difference that the EWM is used to determine the resistance of panel.

Fig. 15c compares all results of the design approach X 9 with the numerical results. It can be concluded that the proposed design approach for the Reduced Stress Method (RSM) can also be used for the Effective Width Method (EWM). Hence, the torsional stiffness of the longitudinal stiffener may be considered when applying the new proposed interpolation equation between column- and plate-like behaviour. The Winter curve should also be used as a reduction factor for plate buckling for aspect ratios smaller than 2 and the reduction curve of Müller for the other cases.

5 Statistical evaluation

Tab. 4 summarises the design approaches. Figs. 12, 13 and 15 show the comparison between the results of the design approaches and the numerical results.

For each design approach, the mean b, the mean value of logarithm of the error terms $\overline{\Delta}$, the standard deviation s_{δ}^{2}, the coefficient of variation of the dispersion variable V_{δ}, the partial factor γ_{Rd}^{*} and final partial safety factor γ_{M}^{*} are determined according to Annex D of EN 1990 [20].

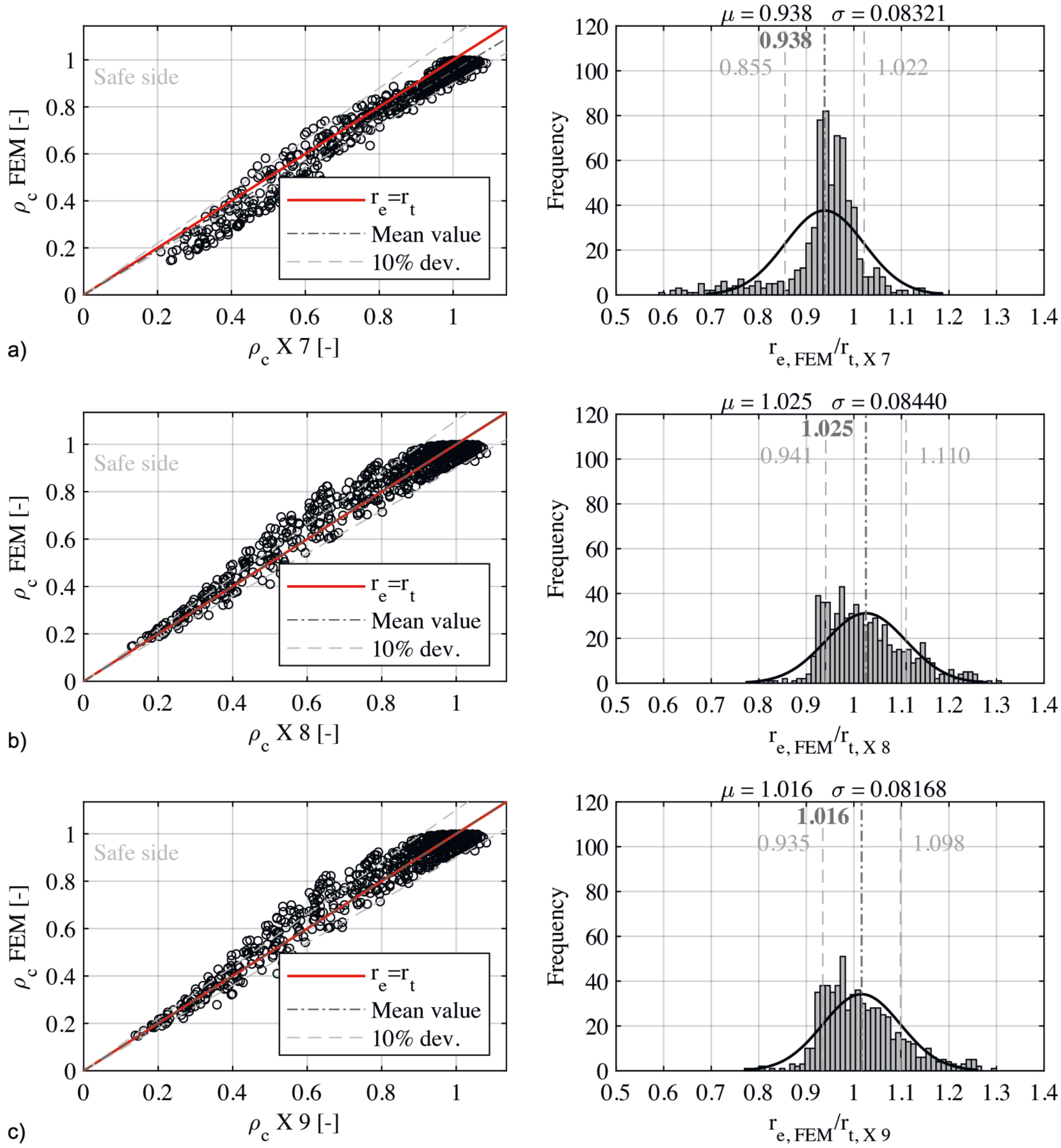

Fig. 15 Comparison of the numerical results with the results of the design approaches: a) X 7, b) X 8 and c) X 9 (left: direct comparison; right: frequency distribution)

In Tab. 5, the results of the statistical evaluation of the design approaches are given. The design approaches X 1–X 6 are developed based on the application of the RSM and X 7–X 9 on the application of the EWM.

The results show that the design approach X 6 gives the best results statistically. The mean value b equal to 1.02 shows that the results are generally on the safe side.

According to the RSM, a partial factor $\gamma_{M1} = 1.1$ is specified and should be applied in the verification. The statistical evaluation for the application of X 5 and X 6 shows that $\gamma_M^* = 1.09$ is required to achieve the safety requirements of EN 1990. As 1.09 is smaller than 1.1, it can be concluded that sufficient reliability is provided. The small standard deviation s_δ^2 and the coefficient of variation of the error terms V_δ indicate that the dispersion of the results is smaller than for the other design approaches. The coefficients of variation of the error terms of the design approaches X 5 and X 6 are small. It can be assumed that the compatibility test is fulfilled. Also, both design approaches may be used, but the X 6 design approach gives slightly better results in view of economy.

Similar results are observed when the Effective Width Method is applied. Approaches X 8 and X 9 give similar

Tab. 5 Statistical evaluation of the design approaches for stiffened panels under direct longitudinal stresses

Approach	b	$\overline{\Delta}$	s_δ^2	V_δ	γ_{Rd}^*	γ_M^*
X 1	1.05	0.1134	0.0908	0.3083	1.48	1.40
X 2	0.98	–0.0341	0.0182	0.1355	1.29	1.21
X 3	0.99	–0.0169	0.0077	0.0877	1.21	1.14
X 4	1.02	0.0018	0.0074	0.0863	1.17	1.11
X 5	1.03	0.0138	0.0052	0.0725	1.15	1.09
X 6	1.02	0.0076	0.0045	0.0670	1.15	1.09
X 7	0.95	–0.0193	0.0093	0.0965	1.27	1.20
X 8	1.00	0.0192	0.0065	0.0806	1.19	1.12
X 9	1.00	0.0127	0.0062	0.0789	1.19	1.12

results, but the statistical values of approach X 9 are a little better than those of approach X 8.

It can be concluded that the proposed interpolation formula leads to safe and economical results for the EWM and the RSM.

6 Summary and conclusions

In this article, the buckling behaviour of stiffened panels with closed-section stiffeners subjected to longitudinal compression stresses has been investigated.

In this investigation, the numerical model was verified and validated using the numerical recalculation of the tests and known example from the literature. Based on the validated and verified numerical model, an extensive parametric study has been executed.

Geometrically and materially nonlinear analyses with imperfections have been conducted to predict the resistances of the panels. The goals of this investigation were to check of current design rules of EN 1993-1-5 in case the torsional stiffness of stiffeners is considered when calculating the critical plate buckling stress.

According to the Amendment A2 of EN 1993-1-5, the torsional stiffness should generally be neglected in the buckling design. prEN 1993-1-5 allows the consideration of the torsional stiffness of stiffeners by means of the reduction curve by Müller in Annex B of EN 1993-1.5 or 12.4 (5) prEN 1993-1-5. In this study, it was shown that this design method is not safe-sided in all cases.

However, the complete neglect of the torsional stiffness leads to conservative results. Therefore, a new interpolation equation between plate- and column-like behaviour is proposed.

Eq. (18) shows the proposed interpolation equation in a general form, which may be applied to different boundary conditions and loadings by modifying the V and P parameters:

$$f = V \cdot \left(\ln\left(\frac{\sigma_{\mathrm{cr,p}}}{\sigma_{\mathrm{cr,c}}} \right) \right)^{P} \qquad 0 \le f \le 1 \qquad (18)$$

In the case of a global buckling analysis of stiffened panels with closed cross-section under longitudinal stresses, the parameters V and P are defined as:

$$V = \left(\bar{\lambda}_{\mathrm{p}} + 1 \right)^{-2/3} \text{ and } P = 1.0 \qquad (19)$$

In case the new proposed interpolation equation is used, the torsional stiffness may be considered when calculating the critical plate buckling stresses. If the aspect ratio of the panel is equal to or greater than 2, the reduction factor of plate buckling has to be determined according to the curve of Müller or 12.4 (5) prEN 1993-1-5, otherwise the Winter curve suffices.

In this study, the panels subjected to constant stresses were investigated. The application of the proposal for the panel subjected to bending is given in [5].

Acknowledgements

The work presented was conducted as part of the research project ‘Longitudinally stiffened panels under multi-axial stresses’. This project of the research association DASt is financed over the AiF within the development programme for industrial community research and development (IGF) from the Federal Ministry of Economic Affairs and Energy (BMWi) based on a decision of the German Bundestag. Special thanks are addressed to MCE GmbH, Max Bögl, SEH Engineering, Plauen Stahl Technologie GmbH, ZSB Zwickauer Sonderstahlbau GmbH, Haslinger for their support. The authors thank Prof. Dr.-Ing. Martin Mensinger as well as Nadine Maier, M.Sc. and Dr.-Ing. Joseph Ndogmo of Technical University of Munich especially for conducting the experimental investigations and for the very good cooperation as project partners. The support by ass. Prof. Dr. Balász Kövesdi within our discussion is greatly appreciated.

References

[1] EN 1993-1-5 (2006) *Eurocode 3: Design of steel structures – Part 1–5: Plated structural elements*. European Standard, October 2006 including AC:2009 + A1:2017 + A2:2019.

[2] prEN 1993-1-5 (2021) *Eurocode 3: Design of steel structures – Part 1–5: Plated structural elements*. Final Document, CEN/TC 250/SC 3 N 3306, February 2021.

[3] Pourostad, V.; Kuhlmann, U. (2022) *Buckling resistance of longitudinally stiffened panels with closed stiffeners under direct longitudinal stresses*. International Colloquium on Stability and Ductility of Steel Structures (SDSS 2022), Aveiro, Portugal, 14–16 September 2022.

[4] Pourostad, V.; Kövesdi, B.; Schillo, N.; Degée, H.; Kuhlmann, U. (2021) *Neue Entwicklungen in prEN 1993-1-5:2020* in: Kuhlmann, U. (Ed.) *Stahlbau-Kalender 2021*. Berlin: Ernst & Sohn, pp. 571–697. https://doi.org/10.1002/9783433610503.ch10

[5] Pourostad, V. (2023) *Stabilitätsverhalten von nicht-rechteckigen Beulfeldern und ausgesteiften Beulfeldern unter mehrachsigen Beanspruchungen* (Stability behaviour of non-rectangular and stiffened panels subjected to multiaxial stresses) [Dissertation]. University of Stuttgart, Institute of Structural Design.

[6] Winter, G. (1947) *Strength of thin steel compression flanges*. Transactions of the American Society of Civil Engineers 112, pp. 527–554.
[7] Müller, C. (2003) *Zum Nachweis ebener Tragwerke aus Stahl gegen seitliches Ausweichen* [Dissertation]. Heft 47, RWTH Aachen.
[8] Martin, P.-O.; Nguyen, T. M. (2017) *Elastic critical stress for global buckling: Closed section stiffeners* (TWG83-2017-011). Presentation at "ECCS Technical Working Group 8.3 (TWG8.3) & TC250/SC3/WG5 (EN 1993-1-5)" meeting. Innsbruck, February 2017.
[9] Martin, P.-O.; Nguyen, T. M.; Davaine, L. (2019) *Effect of the torsional stiffness of closed-section stiffeners on plate buckling in Eurocode 3, Part 1–5*. Steel Construction 12, No. 1, pp. 10–22. https://doi.org/10.1002/stco.201800001
[10] CTICM (2022) *EBPlate software: version 2.01.* [online]. www.cticm.com
[11] Ziad Haffar, M.; Kövesdi, B.; Ádány, S. (2019) *On the buckling of longitudinally stiffened plates, part 2: Eurocode-based design for plate-like behaviour of plates with closed-section stiffeners*. Thin-Walled Structures 145, p. 106395. https://doi.org/10.1016/j.tws.2019.106395
[12] Kövesdi, B. (2019) *Buckling resistance of orthotropic plates subjected by compression. interpolation between plate and column-like behavior.* Journal of Constructional Steel Research 160, pp. 67–76.
[13] Dassault Systèmes (2011) *Abaqus/CAE 6.11-5 mit Abaqus/Standard*. Providence, Rhode Island, USA.
[14] Zizza, A. (2016) *Buckling behaviour of unstiffened and stiffened steel plates under multiaxial stress states* [Dissertation]. Publication of Institute of Structural Design Nr. 2016-1, University of Stuttgart. https://doi.org/10.18419/OPUS-8994
[15] prEN 1993-1-14 (2021) *Eurocode 3: Design of steel structures – Part 1–14: Design assisted by finite element analysis*. Dokument CEN/TC250/SC3/N3499, November 2021.
[16] Mensinger, M.; Ndogmo, J.; Maier, N.; Kuhlmann, U.; Pourostad, V. (2019) *Beuluntersuchungen*. Report No. 118001: BAB A7, Würzburg – Fulda, BW613a, Talbrücke Thulba, Technical University of Munich, MPA Bau Abteilung Metallbau.
[17] Kuhlmann, U.; Mensinger, M.; Maier, M.; Pourostad, V.; Ndogmo, J. (2021) *Längsausgesteifte Beulfelder unter mehrachsiger Beanspruchung*. Final report, Research project DAST/AiF – IGF20455.
[18] Timmers, R.; Lener, G.; Sinur, F.; Kövesdi, B.; Chacón, R. (2015) *Stabilitätsnachweise nach EN 1993-1-5 – Theorie und Beispiele* in: Kuhlmann, U. (Ed.) *Stahlbau-Kalender 2015*. Berlin: Ernst & Sohn, pp. 209–286. https://doi.org/10.1002/9783433605219.ch3
[19] FprEN 1993-1-1 (2021) *As transmitted to Formal Vote, Eurocode 3 – Design of steel structures – Part 1-1: General rules and rules for buildings*. CEN/TC 250/SC 3 N 3504, August 2021.
[20] EN 1990 (2010) *Eurocode 0: Eurocode*. Basis of Structural Design 2002 including A1:2005 + A1:2005/AC:2010.
[21] Seitz, M. (2005) *Tragverhalten längsversteifter Blechträger unter quergerichteter Krafteinleitung* [Dissertation]. Publication of Institute of Structural Design, Nr. 2005-2, University of Stuttgart.

Authors
Vahid Pourostad (corresponding author)
vahid.pourostad@ke.uni-stuttgart.de
University of Stuttgart
Institute for Structural and Design
Pfaffenwaldring 7
70569 Stuttgart, Germany

Prof. Dr.-Ing. Ulrike Kuhlmann
sekretariat@ke.uni-stuttgart.de
University of Stuttgart
Institute for Structural and Design
Pfaffenwaldring 7
70569 Stuttgart, Germany

How to Cite this Paper

Pourostad, V.; Kuhlmann, U. (2022) *Buckling resistance of longitudinally stiffened panels with closed stiffeners under direct longitudinal stresses.* Steel Construction 15, No. 4, pp. 229–243.
https://doi.org/10.1002/stco.202200028

This paper has been peer reviewed. Submitted: 31. August 2022; accepted: 2. October 2022.