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Buckling resistance of stiffened panels subjected to constant transverse compression stresses

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Abstract

In a box girder cross-section with inclined web panel, transverse compression may be introduced into the bottom panel from both sides in addition to constant longitudinal compressive stresses. In EN 1993-1-5, the only analytical method for determining the resistance of panels to biaxial compression is the Reduced Stress Method (RSM). However, the current rules were mainly derived for longitudinal stresses. Due to missing rules, in practice the longitudinal stiffeners need being verified according to flexural member buckling considering the transverse updrift forces.

Therefore, in order to assess the real behaviour, a large parametric study of stiffened panels with closed longitudinal stiffeners under transverse compressive stresses was carried out using a verified numerical model. LBA and GMNIA are performed for each case. To apply the RSM, all critical buckling stresses were determined considering the torsional stiffness of stiffeners. According to EN 1993-1-5, the support of the edges of the panels parallel to the load direction should be set free when determining the critical stress for column-like behaviour. In the case of a longitudinally stiffened panel under transverse stresses, it is not clear whether the support of the plate and the stiffeners or only the plate should be released. In this paper, these interpretations are discussed and results are compared. Finally, a simplified procedure to determine the global reduction factor of plate buckling for transverse stresses in a single step is proposed.

Keywords

plate buckling, column buckling, longitudinal stiffened panels, transverse stresses, reduced stress method, bridges, EN 1993-1-5, closed section stiffeners

1 Introduction and motivation

For modern bridges with large spans, the use of stiffened panels is almost indispensable. To optimise the load-bearing capacity and the material usage, a box cross-section is used for the bridge cross-section, in which the web and base panels are stiffened with closed longitudinal stiffeners. If the webs are inclined, transverse stresses act in the bottom panel. Due to the large acting stresses, panels are usually slender, therefore they are prone to buckling. Stability or buckling verifications of plated structural elements are provided in EN 1993-1-5 [1].

EN 1993-1-5 [1] provides two analytical methods for buckling verification of panels, namely the effective width method and the reduced stress method. For a stiffened panel subjected to transverse stresses on both sides, as shown in Figure 1, the effective width method does not provide a design procedure and the reduced stress method must be used. In this paper, the application of the reduced stress method on panels subjected to equal transverse stresses on the top and bottom of the panels was investigated, see Figure 1.

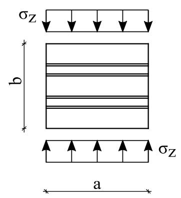

Figure 1 Transverse stresses on both sides of panel

Zizza [2] investigated panels stiffened with open stiffeners under constant biaxial compression stresses. He also found in his evaluation of the regulations according to EN 1993-1-5 [1] that the interpolation between column- and plate -like behaviour led to overestimation of results. To eliminate this overestimation, Zizza proposed to apply the interpolation equation developed by Seitz [3] in the transverse direction. From the results in [2] it can be seen that

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the approach of Seitz [3] can solve the problem for unstiffened panels, but for stiffened panels results can still be observed on the unsafe side, see Figure 2. It can be seen that EN 1993-1-5 [1] overestimates the ultimate capacity of stiffened panel under predominant transverse stresses. The reason for it seems to be the interpolation of the column- and plate-like behaviour of the global analysis in the transverse direction. Therefore, the interpolation between behaviour in the transverse direction is investigated in this paper.

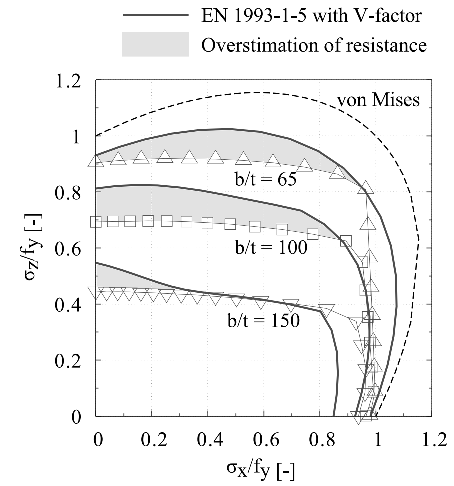


Figure 2 Overestimation of the load-bearing capacity with predominant transverse stresses - for stiffened plates loaded with biaxial compression, $\alpha = 1, n = 4, \gamma = 65$ (Figure C.18 from [2])

New investigations [4], [5] concluded that the torsional stiffness of closed longitudinal stiffeners should be neglected in the calculation of critical buckling stresses to achieve safe results. Pourostad [6] proposes a new interpolation formula for the EN 1993-1-5 buckling design to consider the torsional stiffness of stiffeners, see Equations (2) and (3). The proposal in [6] is derived based on longitudinally stiffened panels under direct longitudinal stresses. The reliability of the new interpolation formula is also investigated in case of transverse stresses in this paper.

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

$$f = \left(\bar{\lambda}_p + 1\right)^{-2/3} \cdot \left(\ln\left(\frac{\sigma_{\mathrm{cr,p}}}{\sigma_{\mathrm{cr,c}}}\right)\right) \qquad 0 \leq f \leq 1 \tag{2}$$

$$\rho_c = \chi_c + \left(\rho_p - \chi_c\right) \cdot f \tag{3}$$

In the equations above, $\sigma_{cr,p}$ is the critical plate buckling stress and $\sigma_{cr,c}$ is the critical column buckling stress. χ_c is the reduction factor for column buckling and ρ_p is the reduction factor for plate buckling and f is the interpolation function.

In the case of transverse compression regarding column-like behaviour, there are different interpretations. These interpretations are discussed below, with respect to the weighting factor ξ, which is give in Equation (1) and it is the essential parameter for determining the behaviour of the panels.

According to the formulation in EN 1993-1-5 [1] or FprEN 1993-1-5 [7] for determining the critical column buckling stresses, the support of edges in the direction of loading are to be set free. It should be pointed out that this regulation in FprEN 1993-1-5 is basically formulated for panels under longitudinal stresses. For the transverse direction, it is unclear how these regulations must be applied, as different interpretations are possible regarding the support of the stiffeners.

In order to take the column-like behaviour into account, the edges parallel to the stresses direction must be released according to EN 1993-1-5 section 4.5.3(1) [1]. If the transverse edges of plate and stiffener are released in case of transverse stresses in the calculation of the column buckling stresses, the longitudinal stiffeners would have no or only a minor influence on the critical buckling stresses behaviour due to the lack of support, see Figure 3. Thus, the elastic critical buckling stresses will be smaller and therefore the weighting factor becomes higher, see Equation (1). The buckling behaviour therefore tend to shift in the direction of the plate-like behaviour, which is the more favourable buckling behaviour. Therefore, this interpretation may lead to an overestimation of the load-bearing capacity of panels.

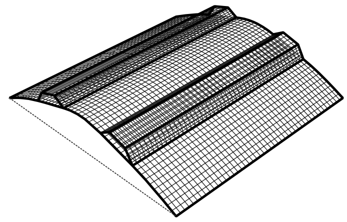


Figure 3 Release the support of the plate and longitudinal stiffeners

Another interpretation of EN 1993-1-5 [1] section 4.5.3(1) or FprEN 1993-1-5 [7] considers that in case of a longitudinally stiffened panel subjected to transverse stresses, only the corresponding edges need to be released, but not the supports of the longitudinal stiffeners, see Figure 4. Thus, the effect of the closed longitudinal stiffeners on the buckling behaviour is considered. Consequently, the results become less favourable and may perhaps be too conservative.

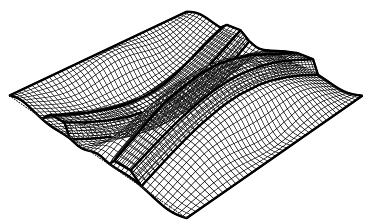


Figure 4 Only the support of the plate released and the stiffeners still fixed

In the following, on the basis of the numerical investigations the question is investigated which interpretations give a better estimation of the load-bearing capacity of the panel.

This paper reflects results of the corresponding author’s dissertation [8], where more in-depth investigations are given and which will be published in near future.

2 Numerical model and parametric study

2.1 General

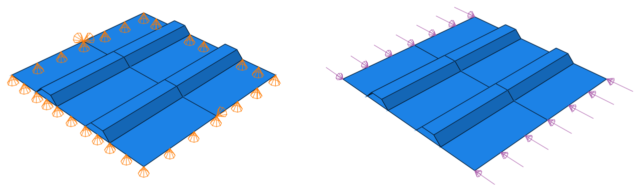
The numerical model was developed and validated with the tests based on panels under biaxial compression as extensively reported in [6], [8], [9] and [10]. In this paper, differently from [6], the plate is loaded in the transverse direction, however the assumptions for material and imperfections are the same as in [6].

2.2 Material

For the parametric study, the steel grade S355 with a yield strength of 355 MPa was used. For the material properties, a bilinear material model without strain hardening according to EN 1993-1-5 [1] was modelled. An elastic modulus 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.

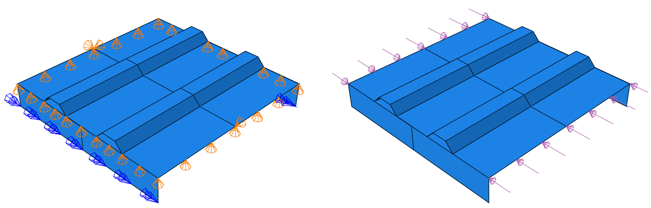
2.3 Boundary condition and loading

Figure 5 shows the simulated boundary conditions and loading for the Linear Buckling Analysis (LBA) to determine the imperfection shape and Figure 6 for the Geometrically and Materially Nonlinear Analysis including Imperfections (GMNIA) to determine the resistance of the panels. The load was applied in the transverse or z-direction. The yield strength of steel f_y was applied along the edges. These applied stresses f_y correspond to the maximum limit stresses for panels in an elastic calculation. Therefore, the load factor determined by numerical GMNIA calculation corresponds to the final reduction factor ρ_c.



a) boundary conditions b) loading

Figure 5 Numerical model for LBA



a) boundary conditions b) loading

Figure 6 Numerical model for GMNIA

2.4 Imperfections

The form of global imperfection was discussed in detail in [6]. Here, the same form of global imperfection by means of a sinus function was applied, as shown in Figure 7. This means 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 Annex C EN 1993-1-5 [1]. It should be noted that in the parametric study the global imperfections were applied in the direction of the stiffeners (positive) and opposite to the direction of the stiffeners (negative).

Two different imperfection approaches according to EN 1993-1-5 Annex C [1] were used as local imperfections. First, a sine function between the stiffeners was applied. The number of assumed waves of the 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. The first buckling modes of each sub-panel were found. These were then combined and applied as local imperfections. As an example, Figure 8 shows the selected buckling mode of a stiffened panel. Figure 9 illustrates in an exaggerated way the superposition of the buckling modes of 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 EN 1993-1-5 Annex C [1] was assumed, where b_i is the width of sub-panel i. The global and local imperfections were then combined according to Table 1. This led to 15 different combinations of imperfections. A reduction of 70 % of the accompanying imperfections according to EN 1993-1-5 Annex C [1] was considered in this investigation.

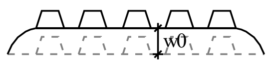


Figure 7 Assumption for the global buckling form

In Tab. 1, the round brackets refer to positive global imperfections and the square brackets refer to negative global imperfections. Figure 10 schematically shows the imperfection combination 8 and 15 for the example of a $\alpha = 1$ $b_{global}/t = 180$ $\gamma^{*}_{\mathrm{sl,i}} = 80$ under transverse compression.

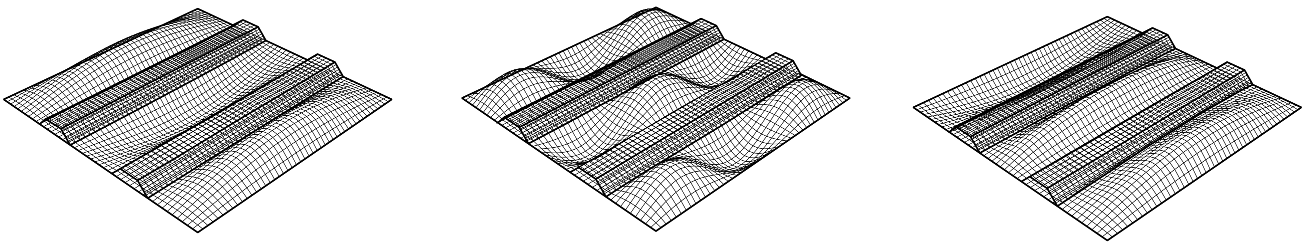
Table 1 Combinations of imperfections

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)	

It should be noted that no residual stresses were simulated in this study, as it was assumed that this type of imperfection was covered by the equivalent geometric imperfection according to EN 1993-1-5 Annex C [1].

a) Buckling mode 1 (Local)   b) Buckling mode 8 (Local)

Figure 8 Selected buckling modes for local buckling of the stiffened bottom plate under transverse stresses ($\alpha = 1$; $b_{global}/t = 180$; $\gamma = 80$)



a) Local -1 Sine half-waves   b) Local -3 Sine half-waves   c) Local (LBA): Superposition of buckling modes 1 and 8

Figure 9 Applied imperfections for local buckling of the stiffened bottom plate under constant compression in transverse direction - (imperfections increased by a factor 75)

a) Combination number (8) - positive   b) Combination number [15] - negative

Figure 10 Examples of combinations of the imperfections of the stiffened buckling panels under constant compression in longitudinal and transverse direction - (imperfections increased by a factor 75)

2.5 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)400 + n_{st} 300 → b = 1100; 1800; 3200 [mm] with n_{st} being the number of stiffeners
- Plate thickness t = 6; 8; 10; 12; 15; 20; 25; 30; 35; 40; 50 [mm]
- Aspect ratios of panels: a/b = 1; 1,5; 2; 3
- 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.

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 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 Figure 11.

3 Evaluation and development of a design approach

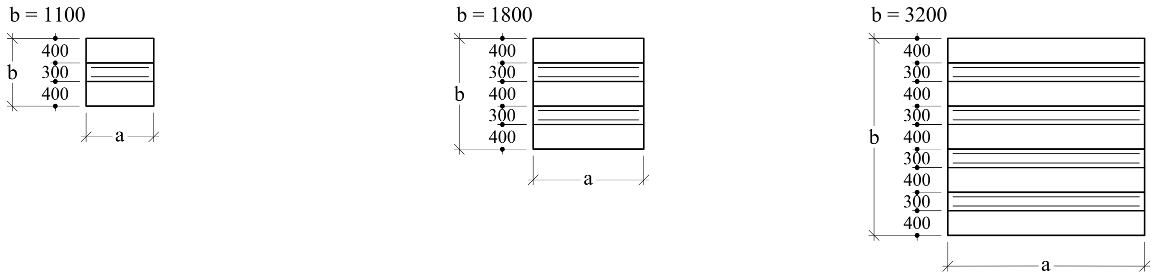
3.1 General

For the global verification, EN 1993-1-5 [1] does not contain clear rules, in particular for the boundary conditions for the calculation of the critical column buckling stresses in a stiffened panel under transverse compression loading. Instead of completely “free” edges in transverse directions, see Figure 3, supports may be assumed for the stiffeners, see Figure 4. In this investigation, the Reduced Stress Method was used. The sub-panel and the global analysis were verified separately with γ_{M1} = 1,0.

For plate-like behaviour, the reduction coefficient for the transverse direction was determined according to Annex B of EN 1993-1-5 [1] or in 12.4 (5) of FprEN 1993-1-5 [7]. The critical plate buckling stresses for sub-panels were calculated numerically considering also the torsional stiffness of the stiffeners. For the global analysis, the critical column buckling stresses and the critical plate buckling stresses were determined by Linear Buckling Analysis (LBA). The new interpolation between plate and column like behaviour as given in [6], based on the investigations of panels under longitudinal stress, were used. In all design approaches, a separate verification of the longitudinal stiffeners according to second order analysis was neglected assuming that the verification of the individual longitudinal stiffeners was covered in the global buckling verification. The design approaches for these investigations are designated as Z N. Z indicates that panels were loaded under transverse stress only. N stands for the number of the design approach.

Table 2 Design approaches for the verification of stiffened panels subjected to transverse stresses

Approach	$\alpha_{cr,c}$ (global) - LBA	Interpolation $\rho_{c,z}$
Z 1	without support of the stiffeners	Equations (2) and (3)
Z 2	with support the stiffeners	Equations (2) and (3)
Z 3	-	$\rho_{c,z} = \chi_c$



a) Stiffener ($n_{st} = 1$) b) 2 Stiffeners ($n_{st} = 2$) c) 4 Stiffeners ($n_{st} = 4$)

Figure 11 Dimensions and arrangement of the stiffeners in the panels in the parametric study

3.2 Design approach Z 1

In this design approach, the support of the plate and the longitudinal stiffeners were set free at the transverse edges. The possible buckling shape of panels for determining the global critical column buckling stress in the transverse direction is shown in Figure 3. Determined ultimate resistance using this design approach are compared with the numerical results in Figure 13 a. It can be seen that this design approach overestimates the resistance of panels.

3.3 Design approach Z 2

In this design approach, the critical buckling stress in the transverse direction $\sigma_{cr,c,z}$ was numerically determined using linear buckling analysis, where the support of the plate at the transverse edges was removed, but the stiffeners were still fixed. Figure 4 shows the buckling shape for Z 2. The results of the design approach are shown in Figure 13 b. It can be seen that the results of this design approach agree very well with the numerical results.

The numerical results of resistance divided by fy were also plotted in a diagram together with the column buckling curves and the relevant plate buckling curves, see Figure 12. Since the column buckling curve depends on the imperfection coefficient α_e, curves were plotted for the largest and smallest α_e from the parameter study. It can be seen that the numerical results approximate the column buckling curves. Furthermore, no plate buckling was observed in the results.

3.4 Design approach Z 3

Based on the results of design approach Z 2, design approach Z 3 was developed. In this design approach, it was assumed that the column-like behaviour in the z-direction always was decisive and that an interpolation between reduction factors regarding to plate-like and column-like behaviour was not necessary. The results of the design approach are shown in Figure 13 c.

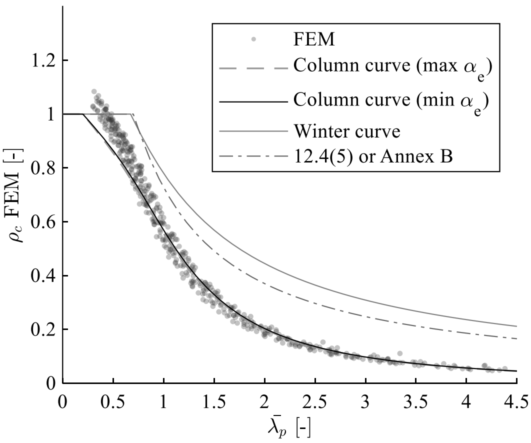
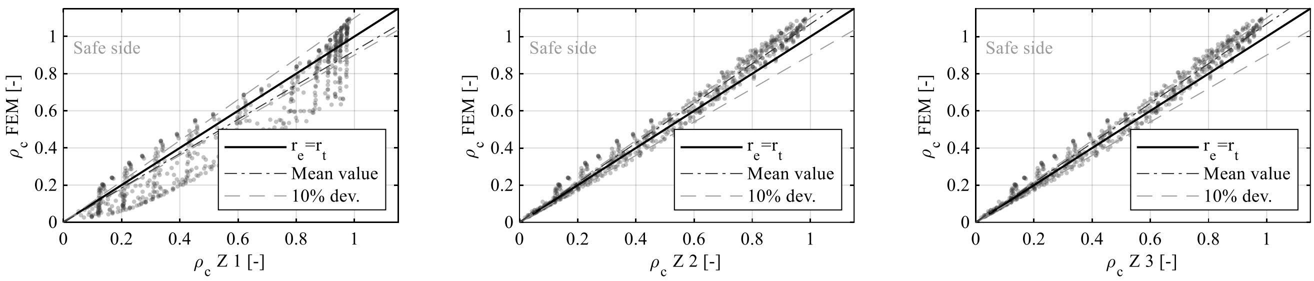


Figure 12 Comparison of the FEM results of resistance with the buckling curves in case of column buckling and the relevant plate buckling curves

It can be seen that this design approach gives similar results to Z 2. With Z 3, the design procedure is considerably simplified because the modelling to determine the critical column plate buckling stress $\sigma_{cr,c,z}$ is omitted and no additional verification of the longitudinal stiffeners according to second-order theory is required.

4 Summary and Outlook

In this paper, stiffened panels under pure constant transverse compression were investigated with a focus on the column-like behaviour in the transverse direction. It was found that for the calculation of the critical buckling stress in the transverse direction, the longitudinal stiffeners should remain supported, while the edges of the plate must be set free.



a) Approach Z1 b) Approach Z2 c) Approach Z3

Figure 13 Comparison of the numerical results with the results of the design approaches: Z 1, Z 2 and Z 3

The investigations showed that the numerical results closely correspond to the column buckling curves. For this reason, it is proposed that in the transverse direction, no interpolation between plate-like and column-like behaviour need to be performed, since column-like behaviour always governs. Furthermore, it is concluded that a separate verification of longitudinal stiffeners subjected to transverse stresses and consequently also to deviation forces according to second-order theory is not necessary and that this verification of longitudinal stiffeners is included in the verification of the global buckling analysis.

In this investigation, these suggestions were developed for the case where the same transverse stresses were applied to the top and the bottom of the panel i.e. on both sides. It should be noted that investigations whether these proposals are also valid for the one-sided transverse stresses led to the discovery of additional internal longitudinal stresses which need to be considered within the verification according to the Reduced Stress Method [8].

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