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ДГКС Друштво грађевинских конструктера Србије ASES Association of Structural Engineers of Serbia R-15 Kongres 2022 Congress 2022

Milica Koprivica[1], *Zlatko Marković*[2]

UPOREDNA ANALIZA NOSIVOSTI NA IZBOČAVANJE USLED POPREČNE SILE PREMA POSTOJEĆEM I NOVOM EVROKODU

Summary:

U ovom radu je data uporedna analiza proračuna punih limenih elemenata prema aktuelnom standardu EN1993-1-5:2006 [1] i novoj verziji prEN1993-1-5:2020 [2] koja je u fazi završne izrade. Poseban akcenat je na problemima izbočavanja usled dejstva poprečne sile, interakcija izbočavanja usled normalnih, smičućih napona i poprečne sile i na metodi redukovanog napona. Pored toga, razlike između pravila za proračun datih u postojećem Evrokodu i novom standardu prEN1993-1-5: 2020 [2] su ilustrovane na konkretnim numeričkim primerima.

Key words: Evrokod 3, puni limeni elementi, izbočavanje, izbočavanje usled dejstva poprečne sile, interakcije

COMPARATIVE ANALYSIS OF PATCH LOADING BUCKLING RESISTANCES ACCORDING TO EXISTING AND NEW EUROCODE

Summary:

This paper compares the procedures for calculating steel plate girders according to the current standards EN1993-1-5: 2006 [1] and the new version prEN1993-1-5: 2020, which is in the final stage of development [2]. There is a particular emphasis on problems of resistance to patch loading, the interaction between transverse force, bending moment and shear force, and the reduced stress method. The differences between the current standard and the new version prEN1993-1-5: 2020 [2] are illustrated using specific numerical examples.

Key words: Eurocode 3, plate girders, buckling, resistance to patch loading, interactions

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1. INTRODUCTION

The Eurocodes were developed to enable the design of structural construction works, buildings and civil engineering work on the harmonized European level. Long-term confidence in the Eurocodes requires the Eurocodes to be developed appropriately. The new generations of these standards are focused on new methods, materials and market requirements. This paper deal with standard EN 1993-1-5: Design of plated structures. There is a tendency in the new version of Eurocode 3 prEN1993-1-5 [2] for solutions to be harmonized with other parts of Eurocode 3 and easy to use for practical applications. The leading development of additional rules to extend the scope of use of Eurocode 3 EN1993-1-5 [1] include the shear resistance of longitudinal stiffeners, the resistance of longitudinal stiffeners to direct stresses, the resistance of girders subjected to patch loading, rules for corrugated webs, F-M-V interaction, biaxial compression, consideration of torsional stiffness of closed-section stiffeners, flange-induced buckling. The structure in the new version prEN1993-1-5 [2] has vastly been improved by moving the former Annex C "*Finite Element Method of Analysis*" to EN1993-1-14 and by integrating the former Annex D "*Plate girders with corrugated webs*" and former Annex E "*Alternative methods for determining effective cross-section*" into the main text. Specific innovations also exist in the reduced stress method and effective width method whose field of application has been extended to non-rectangular panels. Modifications in new Eurocode 3 prEN 1993-1-5 [2] related to patch loading resistance, the interaction between transverse force, bending moment and axial force, the interaction between transverse force, bending moment and shear force, and reduced stress method are presented and commented on in this paper.

2. PATCH LOADING RESISTANCE

Girders loaded by the localized load in the plane of the web are common cases in engineering practice, for example crane girders or incremental bridge launching design situations. The resistance of steel plate girders subjected to patch loading is very important in the design of steel bridges. This problem has been investigated for decades [3,4], but a solution has not yet been found that includes all parameters important for the influence of patch loading resistance. The modification of Eurocode 3 EN1993-1-5 [1] Chapter 6, which refers to the patch loading resistance of steel plate girders, is a consequence of recent research, which has shown that the current definition of plastic resistance overestimates patch loading capacity in certain cases, as are hybrid girders. However, this capacity is slightly underestimated for very slender girders. This paper provides a brief overview of the modification of the patch loading resistance model in the new Eurocode 3 EN1993-1-5 [2], based on hundreds of experimental and numerical results.

According to the current version of the Eurocode 3 EN1993-1-5 [1], patch loading resistance F_{Rd} is obtained by reducing the plastic load capacity by the reduction coefficient χ_{F}. The plastic resistance F_{y} includes the length l_{y}, which can be calculated from the geometrical and mechanical properties of the girders using Eqs. (2) and (3).

$$F_{\mathrm{Rd}} = \frac{\chi_{\mathrm{F}} F_{\mathrm{y}}}{\gamma_{\mathrm{M1}}} = \frac{\chi_{\mathrm{F}} f_{\mathrm{yw}} l_{\mathrm{y}} t_{\mathrm{w}}}{\gamma_{\mathrm{M1}}} \tag{1}$$

$$l_{\mathrm{y}} = s_s + 2t_{\mathrm{f}}\left(1+\sqrt{m_1+m_2}\right) \tag{2}$$

$$m_1 = \frac{f_{\mathrm{yf}} b_{\mathrm{f}}}{f_{\mathrm{yw}} t_{\mathrm{w}}} \quad m_2 = 0.02\left(\frac{h_{\mathrm{w}}}{t_{\mathrm{w}}}\right)^2$$

$$\text{if } \overline{\lambda}_{\mathrm{F}} > 0.5\text{, otherwise } m_2 = 0 \tag{3}$$

$$\overline{\lambda}_{\mathrm{F}} = \sqrt{\frac{l_{\mathrm{y}} t_{\mathrm{w}} f_{\mathrm{yw}}}{F_{\mathrm{cr}}}} \tag{4}$$

$$F_{\mathrm{cr}} = 0.9 k_{\mathrm{F}} E \frac{t_{\mathrm{w}}^{\ 3}}{h_{\mathrm{w}}} \tag{5}$$

$$\chi_{\mathrm{F}} = \frac{0.5}{\overline{\lambda}_{\mathrm{F}}} \tag{6}$$

In the Eqs. (1) to (6) the following notations are used to describe the problem of patch loading, compare the results, and derive conclusions: h_{w} - the web depth, t_{w} - the web thickness, t_{f} - the flange thickness, b_{f} - the flange width, s_s - the patch load length, f_{y} - the yield strength, l_{y} - the effective loaded length for resistance to transverse forces, k_{F} buckling coefficient.

Since the publication of the Eurocode 3 EN1993-1-5 [1], significant research has been carried out to improve the existing standard. More significant research was done in France by Davaine [5] and Müller [6], in Sweden and Spain by Chacón [7]. Numerical studies on steel plate girders subjected to patch loading have questioned the validity of the χ-$\overline{\lambda}$ approach and the calculation of the effective loaded length l_{y}. Davaine [5] studied the resistance of longitudinally stiffened steel plate girders subjected to patch loading and questioned the physical meaning of the term m_2. Research work by Müller [6] contributed the most to the recalibration of standards EN1993-1-5 [1] and the introduction of the new resistance function. Chacón [7] has presented a research work on patch loading that refers to the effect of the flange yield strength f_{yf} and it was predicted that $f_{\mathrm{yf}} / f_{\mathrm{yw}}$ does not influence the ultimate load capacity of patch loaded girders. According to the current standard EN1993-1-5 [1], the patch loading resistance increases with the flange and web strength ratio through the parameter m_1.

The new proposal for patch loading resistance omitted the value of m_2 when the localized load acts at the mid-span of the vertical stiffeners (types of load application a and b). Also, the influence of the ratio $f_{\mathrm{yf}} / f_{\mathrm{yw}}$, does not affect on the collapse mechanism, so the effective width is calculated according to the expression given in Eq. (7):

$$l_y = s_s + 2t_f\left(1+\sqrt{m_1}\right)$$

$$m_1 = \frac{b_f}{t_w} \tag{7}$$

In addition to the change in the value of l_{y}, the resistance function for patch loading is improved by new relate of reduction factor χ_{F} and relative slenderness $\overline{\lambda}_{\mathrm{F}}$. Furthemore,

χ-$\overline{\lambda}_F$ function has been changed to harmonize the plastic resistance F_y and the elastic critical buckling load F_{cr}, and to merge both magnitudes in a single formulation. The proposed expression for the value of takes the form of the equation included in Annex B of existing standard EN1993-1-5 [1].

$$\chi_F = \frac{1}{\varphi_F + \sqrt{\varphi_F^{\,2} - \overline{\lambda}_F}} \tag{8}$$

$$\varphi_F = \frac{1}{2}(1 + \alpha_{F0}(\overline{\lambda}_F - \overline{\lambda}_{F0}) + \overline{\lambda}_F) \tag{9}$$

The value of imperfection factor α_{F0} and plateau length $\overline{\lambda}_{F0}$ represents the adaptive magnitudes that can be calibrated to achieve the desired level of safety [8].
The value of the partial safety factor γ_{M1} depends on the combination of the imperfection factor α_{F0} and plateau length $\overline{\lambda}_{F0}$, which was shown by complex statistical studies [8]. Table 1 shows the value of the corrected partial safety factor in function of the α_{F0} and $\overline{\lambda}_{F0}$, which are the result of research work that preceded the changes in the Eurocode standard [8]. The Eurocode 3 prEN1993-1-5 suggest the values of $\alpha_{F0} = 0.5$ and $\overline{\lambda}_{F0} = 0.75$.

Table 1 – Values of the imperfection factor and the plateau length for achieving the desired level of safety [8]

γ_{M1}=1.0		γ_{M1}=1.1	
λ_{F0}	α_{FO}	λ_{F0}	α_{FO}
0.5	1	0.5	0.75

The expression for the buckling coefficient k_F remains the same for both longitudinally stiffened and unstiffened girders. Furthermore, in Eurocode 3 prEN1993-1-5 [2], the value of k_F may be obtained from Annex A and is not given within Chapter 8, which refers to the resistance to patch loading.

The differences in the values of patch loading resistance calculated using EN1993-1-5 [1] and prEN1993-1-5 [2] are illustrated by a numerical example. The example refers to the steel bridge construction, a continuous girder with a span of 3x40 m, and the incremental bridge launching method of construction. The bridge member is a steel plate girder with an open cross-section, as shown in Fig. 1.

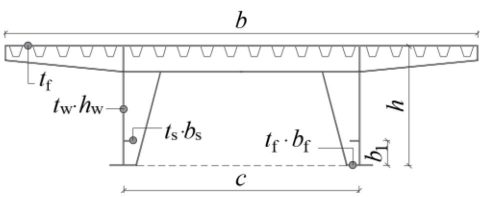

Figure 1 – Cross-section of bridge girder

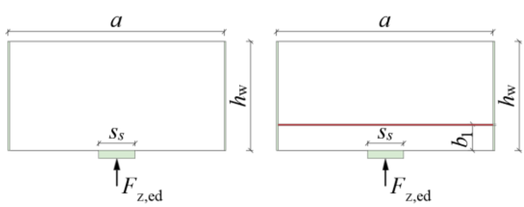

Figure 2 – Introducing the localized load in the plane of the web

The dimensions of the cross-section are set to be: the top flange width b = 10000 mm, the top and bottom flange thickness t_f = 20 mm, the web depth h_w = 2500 mm, the web thickness t_w = 14 mm, the bottom flange width b_f = 600 mm, the thickness of the longitudinal stiffener t_s = 20 mm, the width of the longitudinal stiffener b_s = 200 mm, the distance between vertical stiffeners $a = 2h_w$ = 5000 mm.

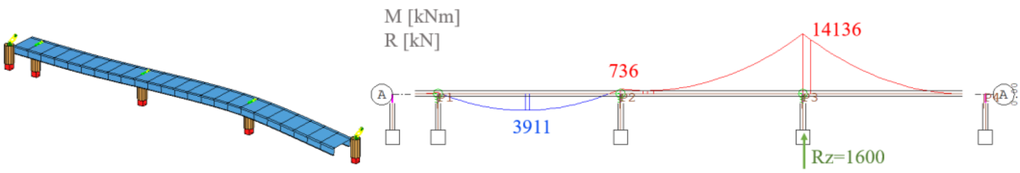

Figure 3 – Critical position of the girder, bending moment diagram and support reaction

The values of the support reactions for the critical position of the girder were calculated - Fig. 3. For the design values of the obtained transverse force, the patch loading resistance was checked for the position of the localized load presented in Fig. 2. The procedure was carried out for the longitudinally unstiffened and stiffened steel plate girder and for two patch load length $s_s = 0.1h_w$ = 250 mm and $s_s = 0.3h_w$ = 750 mm. In addition to the expression in the Eurocode EN1993-1-5 [1] and prEN1993-1-5 [2], the patch loading resistance was also obtained numerically by Abaqus [9], and using the expression proposed by [10] - Eq. (10-11).

$$F_{rf}^{\text{unstiff}} = 0.75 f(\alpha) f(h_w/t_w) t_w^3 \sqrt{E f_{yw}} \sqrt{t_f/t_w}\, f^{\text{unstiff}}(s_s) \tag{10}$$

$$F_{rf}^{\text{stiff}} = F_{rf}^{\text{unstiff}} f(b_1) f^{\text{stiff}}(s_s) \tag{11}$$

For the FE analysis [9], a general-purpose four-node quadrilateral shell element with reduced integration and six degrees of freedom per node S4R from the Abaqus element library was used. Finite element size is adopted to 50 mm for all numerical runs. Geometric imperfections correspond to the first buckling shape mode with the magnitude $h_w/200$ according to the proposition of Annex C EN1993-1-5 [1]. The considered material is homogenous with an elastic modulus of E = 210 GPa and Poisson's ratio of v = 0.3. The stress-strain curve used for material modelling is a simplified bilinear curve.

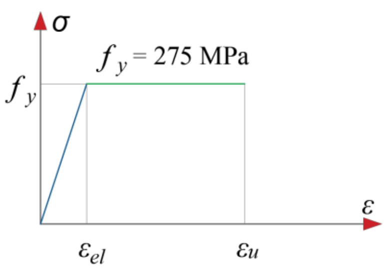

Figure 4 – Material stress-strain curve

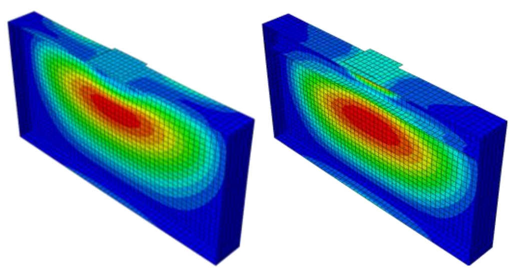

Figure 5 – Deformed shape of steel plate girders after local buckling

Table 2 – The ultimate strength of longitudinally stiffened and unstiffened girder, for various patch loading lengths (us-unstiffened, st-stiffened)

	F_{Rd}^{EC3} / F_{Rd}^{FEA}	F_{Rd}^{prEC3} / F_{Rd}^{FEA}	F_{rf}/ F_{Rd}^{FEA}
$s_s=0.1h_w$, *us*	0.64	0.51	0.72
$s_s=0.1h_w$, *st*	0.69	0.55	0.71
$s_s=0.3h_w$, *us*	0.58	0.54	0.81
$s_s=0.3h_w$, *st*	0.57	0.52	0.75

Table 2 presents the results of the ultimate strengths of longitudinally stiffened and unstiffened steel plate girders subjected to patch loading, analyzed in this paper in the previously described different ways. Patch loading resistances calculated using Eqs. (10,11) and procedures given in the current and new versions of Eurocode are presented and compared to the value of the patch loading resistance obtained numerically. The following notations are used in Table 2 to describe the ultimate strength of steel plate girder subjected to patch loading: F_{Rd}^{EC3} - the ultimate strength obtained by EN1993-1-5 [1], F_{Rd}^{prEC3} - the ultimate strength obtained by prEN1993-1-5 [2], F_{rf} - the ultimate strength obtained using the expression proposed by [10] and F_{Rd}^{FEA} - the numerically obtained ultimate strength [9]. The patch loading resistances F_{Rd}^{EC3}, F_{Rd}^{prEC3} used in the Table 2 and Fig. 6-7 are not divided by the partial safety factor γ_{M1} for steel bridge structures (the partial safety factor was neglected and presented values of ultimate strengths correspond to the characteristic values), so that the results could be compared.

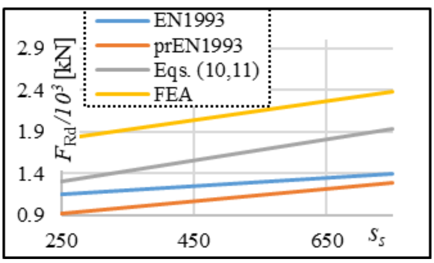

Figure 6 – The ultimate strength of unstiffened steel plate girder-comparison

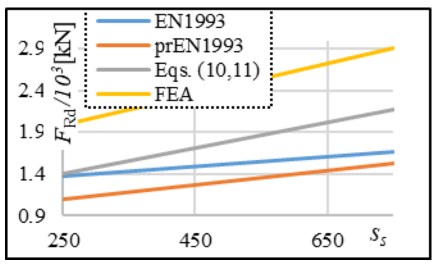

Figure 7 – The ultimate strength of stiffened steel plate girder-comparison

Based on the considered numerical examples, it can be concluded that Eurocode 3 prEN1993-1-5 [2] gives more conservative results than Eurocode 3 EN1993-1-5 [1]. A detailed numerical and statistical analysis should be done to verify whether this is a general trend. Both EN1993-1-5 [1] and prEN1993-1-5 [2] give more conservative results than the numerical results and results obtained by Eqs. (10,11) as shown Fig. 6-7.

3. INTERACTIONS

3.1. INTERACTION BETWEEN DIRECT STRESS AND PATCH LOADING BUCKLINGS

If the girder is subjected to a concentrated transverse force acting on the compression flange in conjunction with bending and axial force, the resistance should be verified using Eq. (12).

$$\eta_2 + 0.8\eta_1 \leq 1.4 \tag{12}$$

$$\eta_1 = \frac{N_{\mathrm{Ed}}}{f_{\mathrm{y}} A_{\mathrm{eff}} / \gamma_{\mathrm{M0}}} + \frac{M_{\mathrm{Ed}} + N_{\mathrm{Ed}} e_{\mathrm{N}}}{f_{\mathrm{y}} W_{\mathrm{eff}} / \gamma_{\mathrm{M0}}} \tag{13}$$

$$\eta_2 = \frac{F_{\mathrm{Ed}}}{F_{\mathrm{Rd}}} \tag{14}$$

In the Eq. (12-14) the following notations are used: A_{eff} - the effective cross-sectio area, e_N - the shift in the position of neutral axis of effective cross section, M_{Ed} - the design bending moment, N_{Ed} - the design axial force, W_{eff} - the effective elastic section modulus, γ_{M0} - the partial factor.

The parameter η_2 represents the ratio between the design transverse force F_{Ed} and the design resistance to local buckling under transverse forces F_{Rd}. The interaction formula has not changed in prEN1993-1-5 [2], however the interaction results are different in relation to EN1993-1-5 [1], because of the design value of patch loading resistance F_{Rd}. The interaction formula gives a higher value if the procedure given in prEN1993-1-5 [2] is used, compared to EN1993-1-5 [1]. The difference between the interaction formula results calculated using EN1993-1-5 [1] or prEN1993-1-5 [2] is the same as the difference in the value η_2 obtained by EN1993-1-5 [1] or prEN1993-1-5 [2].

3.2. INTERACTION BETWEEN TRANSVERSE FORCE, BENDING MOMENT AND SHEAR FORCE

In the case of steel structures are subjected to the combination of transverse force, bending moment and shear force the interaction of stability behaviour is an essential aspect of the bridge design and should be taken into consideration. In the current version of the EN1993-1-5 [1] there is no design method for desing resistance of the steel plate girder under the combined loading situation. Therefore, the situation with these three effects simultaneously can often occur in case of bridge girders during launching. The Eurocode 3 prEN1993-1-5 [2] contains the expression for interaction between transverse force, bending moment and shear force. If the girder is subjected to a concentrated transverse force acting on the compression flange in conjunction with bending moment and shear force, the resistance should be verified using Eq. (15-17).

$$\left(\overline{\eta_1}\right)^{3.6} + \left[\overline{\eta_3}\left(1 - \frac{F_{\mathrm{Ed}}}{2V_{\mathrm{Ed}}}\right)\right]^{1.6} + \eta_2 \leq 1 \tag{15}$$

$$\overline{\eta_1} = \frac{M_{\mathrm{Ed}}}{M_{\mathrm{f,eff,Rd}}} \tag{16}$$

$$\overline{\eta_3} = \frac{V_{\mathrm{Ed}}}{V_{\mathrm{bw,Rd}}} \tag{17}$$

In the Eq. (15-17) the following notations are used: $M_{\mathrm{f,eff,Rd}}$ - the design plastic moment of resistance of the cross-section consisting of the effective area of the flange and the fully effective web irrespective of its section class, $V_{\mathrm{bw,Rd}}$ - the contribution of the web to the design resistance to shear.

This interaction should be verified only if $\eta_2 > 0.1$. If $\eta_2 \leq 0.1$ the verification is limited to a bending moment and shear force interaction.

4. REDUCED STRESS METHOD

The current standard Eurocode 3 EN1993-1-5 [1] provides two different calculation methods for a plate buckling assessment: the effective width method and the reduced stress method as an alternative method. The reduced stress method implies a linear stress distribution until the stress limits are reached in the section's weakest part. After that, there is no redistribution of the stress and the stress limits of the weakest part of the cross-section governs the resistance of the full cross-section. This method gives conservative results compared to the effective width method but is suitable for complex stress states and non-uniform geometries. The reduced stress method applies to any geometries and loadings considering the full stress field and its interaction - Fig. 8. It is shown [11] that the interaction verification in its pure format based on the von Mises yield criterion is not able to represent the actual behaviour of biaxially compressed plates. The current formulation of the reduced stress method may lead to unsafe results for the case of the plate under biaxial compression [11], so a modification has been proposed by introducing a ρ_{V} - factor in the interaction formula in prEN1993-1-5 [2]. This verification - Eq. (18) should be used for each panel and subpanel within the whole cross section.

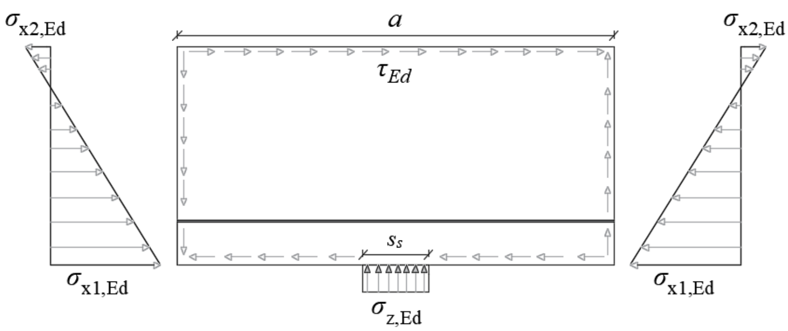

Figure 8 - The plate of girder under the full stress field

$$\sqrt{\left(\frac{\sigma_{\mathrm{x,Ed}}}{\rho_{\mathrm{c,x}}}\right)^2 + \left(\frac{\sigma_{\mathrm{z,Ed}}}{\rho_{\mathrm{c,z}}}\right)^2 - \rho_{\mathrm{V}} \cdot \left(\frac{\sigma_{\mathrm{x,Ed}}}{\rho_{\mathrm{c,x}}}\right) \cdot \left(\frac{\sigma_{\mathrm{z,Ed}}}{\rho_{\mathrm{c,z}}}\right) + 3 \cdot \left(\frac{\tau_{Ed}}{\chi_{\mathrm{w}}}\right)^2} \leq \frac{f_{\mathrm{y}}}{\gamma_{\mathrm{M1}}} \tag{18}$$

In the interaction formula - Eq. (18), parameter $\rho_{\mathrm{V}} = \rho_{\mathrm{c,x}}\rho_{\mathrm{c,z}}$ when $\sigma_{\mathrm{x,Ed}}$ and $\sigma_{\mathrm{z,Ed}}$ are both compression, otherwise $\rho_{\mathrm{V}} = 1/\left(\rho_{\mathrm{c,x}}\rho_{\mathrm{c,z}}\right)$. The following notations are used in Eq. (18): $\rho_{\mathrm{c,x}}$ - the reduction factor for longitudinal stresses, $\rho_{\mathrm{c,z}}$ - the reduction factor for transverse stresses, χ_{w} - the reduction factor for shear stresses, $\sigma_{\mathrm{x,Ed}}$, $\sigma_{\mathrm{z,Ed}}$, τ_{Ed} - the components of the stress field in the ultimate limit state.

The Eurocode 3 prEN1993-1-5 [2] provides the formula - Eq (19) to verify the stress limit from the equivalent effective area, for the class 4 plates that are unstiffened and supported out of their plane along all four edges, within Chapter 12 refers to the reduces stress method. The procedure assumes calculating the thickness of each class 4 plates within the cross-section reduced by their individual reduction factor $\rho_{\mathrm{n}} = \min\left(\rho_{\mathrm{cx}}, \rho_{\mathrm{cz}}, \chi_{\mathrm{w}}\right)$.

The effective cross-section properties leads to the values of the stress field in the ultimate limit state $\sigma_{\mathrm{x,eff,Ed}}$, $\sigma_{\mathrm{z,eff,Ed}}$, $\tau_{\mathrm{eff,Ed}}$.

$$\sqrt{\sigma^2_{\mathrm{x,eff,Ed}} + \sigma^2_{\mathrm{z,eff,Ed}} - \rho_{\mathrm{V}}\sigma_{\mathrm{x,eff,Ed}} \cdot \sigma_{\mathrm{z,eff,Ed}} + 3\tau^2_{\mathrm{eff,Ed}}} \le \frac{f_{\mathrm{y}}}{\gamma_{\mathrm{M1}}} \tag{19}$$

In the interaction formula - Eq. (19), parameter $\rho_{\mathrm{V}} = \rho_{\mathrm{n}}^2$ when $\sigma_{\mathrm{x,eff,Ed}}$ and $\sigma_{\mathrm{z,eff,Ed}}$ are both compression, otherwise $\rho_{\mathrm{V}} = 1$.

In addition to the EN1995-1-5 [1], the prEN1995-1-5 [2] contains suggestions on sections that plate buckling verification of rectangular stiffened panel should be checked, according to the Fig. 9. The plate buckling verification of a rectangular stiffened internal panel may be carried out with the maximum stress values at a distance 0.4*a* or 0.5*b*, section A-A Fig. 9 [2]. For the rectangular subpanel, the maximum stress values may be carried out at a distance $0.4b_{\mathrm{loc}}$, section B-B Fig. 9 [2]. In addition, the elastic gross cross-sectional resistance should be checked at the end of the panel, section C-C, according to the Fig 9 [2].

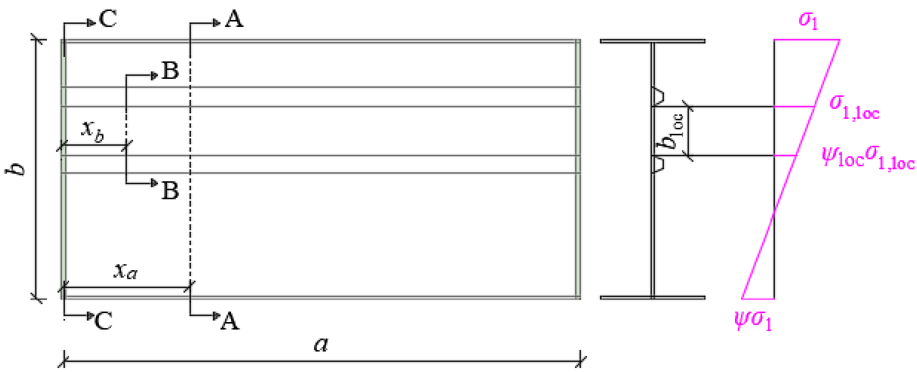

Figure 9 - Application of the reduced stress method for stiffened panels [2]

In the new Eurocode prEN1993-1-5 [2], the reduced stress method is explained in more detail and a flowchart for the procedure application is given. In addition, the flowchart explains which procedure for calculating the reduction factors must be used for transverse, longitudinal, shear stresses, and column buckling behavior.

5. CONCLUSION

In order to harmonize Eurocode 3 regulations, the new generation of Eurocode 3 prEN1995-1-5 [2] has been modified and improved. The problems of steel plate structures are explained more explicitly and in detail. The modifications are the result of numerious experimental and numerical works. In the field of patch loading resistance, the χ-$\bar{\lambda}_F$ approach has been changed. Some modifications have also been made in calculating the effective width l_y. A new interaction equation is introduced that considers the influence of bending moment, shear force and transverse force. The reduced stress method has been expanded, more clearly defined and modified to obtain even more reliable results for biaxially compressed plates. Based on the considered numerical examples, it can be concluded that new version of Eurocode 3 [2] gives more conservative results for patch loading resistance than current Eurocode 3 [1], but a detailed numerical and statistical analysis should be done to verify whether this is a general trend.

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