In memoriam of Darko Beg, Ph.D., Professor of Steel Structures, University of Ljubljana, Slovenia, one of the most active members of ECCS/TWG8.3, passed away on 11th February 2014.

EVOLUTION OF EUROCODE 3 – AMENDMENTS TO EN 1993-1-5 FOR PLATE BUCKLING

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Abstract. Starting in 2010 the European Commission initiated the process of evolution of the first generation of Eurocodes. Based on Mandates M/466 EN and M/515 EN, CEN/TC250 created several expert groups which deal with this evolution work. This paper reports about the common ongoing work of Working Group TC250/SC3/EN 1993-1-5 and ECCS Technical Working Group 8.3 (Plate Buckling). Amendments which have been already prepared in order to improve the ease-of-use and to cover technical development are presented. An outlook which tasks are to be addressed in further work is given.

Key words: Eurocode, EN 1993-1-5, evolution, amendments, plate buckling.

1. INTRODUCTION

The launch of the first generation of Structural Eurocodes marked the outcome of more than 30 years of collaborative work which started in 1975 with the objective to create common European standards for the design of building and civil engineering structures. Today 10 Structural Eurocodes provide rules for basis of design, actions on structures and structural design rules for the use of all major construction materials such as concrete, steel, timber, masonry and aluminum. By December 2010 national standards which were in conflict with any of the 58 Eurocode parts had to be withdrawn.

Starting in March 2010 the European Commission sent Mandate M/466 EN "Programming Mandate addressed to CEN in the field of structural Eurocodes" to the European Committee for Standardization (CEN) in order to initiate the process of evolution of the first generation of Eurocodes. Such an evolution has been considered necessary to sustain the user's confidence in the standards and to take market developments, innovation and research into consideration both through the revision of existing standards and the development of new standards.

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CEN responded to this mandate in June 2011 by submitting a detailed standardisation work programme (consisting of several project proposals) to the European Commission. In December 2012 the European Commission forwarded the follow-up Mandate M/515 EN "Mandate for amending existing Eurocodes and extending the scope of structural Eurocodes" to CEN. In general, the mandate foresees that an additional Eurocode on structural glass and substantial additions to the existing standards are developed.

2. EVOLUTION OF EUROCODE 3 PART 1.5

CEN Technical Committee 250 "Structural Eurocodes" (CEN/TC250) prepared the above mentioned standardisation work programme and has now the mandate to prepare the second generation of Eurocodes. For existing Eurocodes several subcommittees (SC) already exist within CEN/TC250 and it is SC3 which deals with Eurocode 3 on Structural Steel. Following Mandate M/515 EN, CEN/TC250/SC3 created several Working Groups (formerly called Evolution Groups) for the evolution of the different parts of Eurocode 3. This paper reports about the work of Working Group EN 1993-1-5 which deals with the design of plated structural elements (plate buckling).

Today, aside of other tasks, the technical development of rules for plate buckling is mainly driven forward by the European Convention for Constructional Steelwork (ECCS) and its Technical Working Group 8.3 (ECCS/TWG83). Therefore it became obvious that both groups are brought closely together in order to minimize work and time effort. For that reason Working Group EN 1993-1-5 and the ECCS/TWG83 group have common meetings and work jointly together on the further development of EN 1993-1-5. Within the task SC3.T4 of the Mandate M/515 EN the following sub-tasks have been formulated:

- reduction in number of Nationally Determined Parameters (NDPs),
- enhanced ease of use,
- imperfections for flat plate elements,
- improved interaction rules for plates,
- improved patch loading rules for plates,
- stiffener design,
- harmonization of design rules for stiffened plated elements,
- guidance for use of FEM in design,
- development of advanced design rules for extended girder applications such as corrugated webs.

The mandate will have four overlapping phases with EN 1993-1-5 being in the second phase which is planned starting in 2015. Since there will be a large number of amendments dealing with all Eurocode 3 parts which are presented to CEN/TC250/SC3 in the end, it has been decided within ECCS/TWG83 group and

Working Group EN 1993-1-5 to have as many amendments ready as possible and as early as possible. This paper summarizes the amendments which have been presented to CEN/TC250/SC3 for decision. Symbols and abbreviations are given at the end of this paper. Nevertheless, it is recommended to the reader to have the standard EN 1993-1-5 at hand for the following sections.

In the end, an outlook is given on the proposals which are currently under preparation. Besides that, discussions are ongoing in order to harmonize EN 1993-1-5 with other Eurocode parts such as EN 1993-1-1 e.g. on Class-4 cross-sections and EN 1993-1-3, e.g. on Annex D dealing with the use of the effective thickness method.

3. AMENDMENTS COVERING ENHANCED USE

3.1. RESISTANCE OF LONGITUDINALLY STIFFENED PLATES SUBJECTED TO DIRECT STRESSES; INTERACTION BETWEEN PLATE AND COLUMN BUCKLING

The determination of the interpolation coefficient ξ in Clause 4.5.4(1) which identifies whether the plate tends to plate buckling or column-like buckling is an extensive calculation procedure as it requires the use of several different sections of EN 1993-1-5 i.e. Section 4.5.3, Annex A.1, Annex A.2.1 and Annex 2.2. A simplification of this procedure has been already discussed in the COMBRI project [1] but was elaborated in detail by Darko Beg [2–4]. The amendment aims to simplify the existing design rules by giving a direct calculation method for the interpolation coefficient ξ, which will thus lead to a strong consolidation of the whole calculation procedure of Clause 4.5.4(1). For longitudinally stiffened plates the interpolation coefficient ξ may be obtained directly from one of the following equations.

For orthotropic plates with at least three stiffeners the interpolation coefficient ξ may be obtained from:

$$\xi = k_{\sigma,p}\alpha^2 \frac{b_{sl,1}}{b_c}\frac{1+\delta}{\gamma} - 1 \quad \text{for} \quad \psi \neq 1; \tag{1}$$

$$\xi = \frac{\left(1+\alpha^2\right)^2 - 1}{\gamma} \quad \text{for} \quad \psi = 1, \;\; \alpha \geq 0{,}5, \;\; 0{,}5 < \alpha \leq \sqrt[4]{\gamma}, \tag{2}$$

$$\text{but} \quad 0 \leq \xi \leq 1 .$$

Parameters $k_{\sigma,p}$, α, $b_{sl,1}$, b_c, δ, γ and ψ are specified according to Annex A.1 and Figure A.1, EN 1993-1-5.

For orthotropic plates with one or two stiffeners the interpolation coefficient ξ may be obtained from:

$$\xi = k_{\sigma,p}\alpha^2 \frac{b_{sl,1}}{b_c}\frac{A_{sl,1}t^2}{I_{sl,1}12(1-\nu^2)} - 1, \tag{3}$$

where $k_{\sigma,p}$ is known from relevant computer simulations, or according to Annex A.2, EN 1993-1-5.

For the most common case $a \le a_c$ and by using 4.5.3(3), EN 19931-5, for $\sigma_{cr,c}$ the interpolation coefficient ξ can be expressed as:

$$\xi = \frac{a^4 bt^3}{4\pi^4(1-\nu^2)b_1^2 b_2^2 I_{sl,1}} \quad \text{for} \quad a \le a_c. \tag{4}$$

Parameters $k_{\sigma,p}$, α, a, a_c, b, b_1, b_2, t, ν, $A_{sl,1}$ and $I_{sl,1}$, are specified according to Annex A.2, EN 19931-5. The geometrical values $b_{sl,1}$ and b_c are specified in Figure A.1, EN 1993-1-5.

For the case with two stiffeners in the compression zone ξ should be calculated for the three cases given in Annex A.2.1(7), EN 1993-1-5, and the lowest value is taken as relevant.

3.2. RESISTANCE OF LONGITUDINALLY STIFFENED PLATES SUBJECTED TO DIRECT STRESSES; EFFECTIVE AREA

Numerical studies in [1] have shown that Equation (4.5) in Clause 4.5.1(4) and the resulting effective^p areas due to plate buckling may lead to unsafe results for plates with weak stiffeners.

As a result stiffened plates having weak longitudinal stiffeners should be considered as unstiffened plates regarding their resistance to direct stresses and their effective^p area should be calculated as unstiffened plates according to Section 4.4, EN 1993-1-5. Longitudinal stiffeners should be considered as weak stiffeners if their relative bending stiffness γ is less than 25, where γ is defined by:

$$\gamma = \frac{E \cdot I_s}{b \cdot D}, \text{ with } D = \frac{E \cdot t^3}{12 \cdot (1-\nu^2)}. \tag{5}$$

3.3. SHEAR RESISTANCE OF LONGITUDINALLY STIFFENED GIRDERS

Studies in [1, 5] have shown that for unstiffened web panels or panels stiffened by open cross-section stiffeners the assumption of hinged boundary conditions is a requirement for the use of the shear buckling curves in Clause 5.3(1). Thus the factor χ_w according to Table 5.1, EN 1993-1-5, is only valid for slendernesses $\overline{\lambda}_w$ which are determined for plates with hinged boundary conditions. However, it can be shown that closed-section longitudinal stiffeners have a beneficial effect on the overall plate buckling resistance compared to open section stiffeners. This is particularly the case for closed-section longitudinal stiffeners connected to the end-posts. In such a situation, additional rigidity is provided to the end-posts by the longitudinal stiffener. In Clause 5.3(2) and Clause 5.3(4) this effect is not considered so far.

Thus in Clause 5.3(2) it should be added that for webs stiffened by closed-section longitudinal stiffeners connected to the end posts and vertical stiffeners, the end posts may always be considered as rigid. Even in the worst situation of a small closed-section longitudinal stiffener connected to a large transverse stiffener, the torsional restraint remains significant (even higher than in the case of very large open stiffeners connected to small transverse stiffeners) and that the average stiffness ratio in the case of closed sections is of an order of magnitude 100 times higher than for open sections.

Thus Clause 5.3(4) should be modified such that the second moment of area of an open-section longitudinal stiffener should only be reduced to 1/3 of its actual value when calculating the shear buckling coefficient k_τ.

4. AMENDMENTS COVERING TECHNICAL DEVELOPMENT

4.1. RESISTANCE OF STEEL PLATE GIRDERS SUBJECTED TO PATCH LOADING

The contributions of several recent doctoral studies [6–10] underline the need of a further modification of the plastic resistance F_y against patch loading which appears in the current version of EN 1993-1-5. According to these studies, the current definition of the plastic resistance overestimates patch loading capacity in certain cases (hybrid girders) whereas this capacity is slightly underestimated for others (very slender girders).

Thus Chapter 6 should be modified as follows. In Section 6.4 the reduction factor χ_F for effective length for resistance should be obtained from:

$$\chi_F = \frac{1.0}{\varphi_F + \sqrt{\varphi_F^2 - \bar{\lambda}_F}} \leq 1.0\,, \tag{6}$$

where

$$\varphi_F = \frac{1}{2}\left(1 + \alpha_{F0} \cdot \left(\bar{\lambda}_F - \bar{\lambda}_{F0}\right) + \bar{\lambda}_F\right), \tag{7}$$

$$\bar{\lambda}_F = \sqrt{\frac{l_y \cdot t_w \cdot f_{yw}}{F_{cr}}} \tag{8}$$

$$\alpha_{F0} = 0.75, \quad \bar{\lambda}_{F0} = 0.50. \tag{9}$$

It should be noted that the values according to Equation (9) are based on a value of $\gamma_{M1} = 1.1$.

In Section 6.5 the effective loaded length l_y should be calculated as follows:

$$m_1 = \frac{b_f}{t_w}, \tag{10}$$

$$m_2 = 0.02 \cdot \left(\frac{h_w}{t_f}\right)^2 \quad \text{if } \bar{\lambda}_F > 0.5 \tag{11}$$

$$m_2 = 0 \quad \text{if } \bar{\lambda}_F \le 0.5,$$

For box girders, b_f in Equation (10) should be limited to $15 \cdot \varepsilon \cdot t_f$ on each side of the web.

For types a) and b) in Fig. 6.1, EN 1993-1-5, l_y should be obtained using:

$$l_y = s_s + 2 \cdot t_f \cdot \left(1 + \sqrt{m_1}\right). \tag{12}$$

4.2. INTERACTION BETWEEN PATCH LOADING, BENDING MOMENT AND SHEAR FORCE

If steel structures are subjected to the combination of bending, shear and patch loading, the interacting stability behaviour should be taken into consideration in design. The combined loading situation can often occur in case of bridge girders during launching. Therefore the determination of the load carrying capacity under the combined loading situation is an important aspect of the bridge design. In the current version of the EN 1993-1-5 there is no standard design method to take the interaction of these three effects into account and there has been a very small number of previous investigations in the literature about this topic. Consequently, no formulation for the interaction between transverse force, bending moment and shear force is given in Chapter 7. Based on a number of recent studies [11–13], Section 7.2 should be replaced such that 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 Sections 4.6, 5.5, 6.6, EN 1993-1-5, and the following interaction expression:

$$\bar{\eta}_1^{\,3.6} + \left[\bar{\eta}_3 \cdot \left(1 - \frac{F_{Ed}}{2 \cdot V_{Ed}}\right)\right]^{1.6} + \eta_2 \le 1.0, \tag{13}$$

where

$$\overline{\eta_1} = \frac{M_{Ed}}{M_{pl,Rd}}, \tag{14}$$

$$\overline{\eta_3} = \frac{V_{Ed}}{V_{bw,Rd}}. \tag{15}$$

4.3. RESISTANCE OF GIRDERS WITH CORRUGATED WEBS SUBJECTED TO PATCH LOADING

The current version of the Annex D gives no proposal to determine the resistance against transverse force in case of girders with trapezoidally corrugated webs. It should be noted that Annex D, EN 1993-1-5, is the only part of Eurocode 3 which deals with girders with corrugated webs. The objective of this amendment is to improve the standard with a design method to determine the patch loading resistance of girders with trapezoidally corrugated webs based on [14–18].

The design resistance of trapezoidally corrugated webs can be determined according to Equation (16) provided that the compression flange is adequately restrained in lateral direction.

The design method can be used if the load is applied through the flange and restrained by shear forces in the web (Fig. 1).

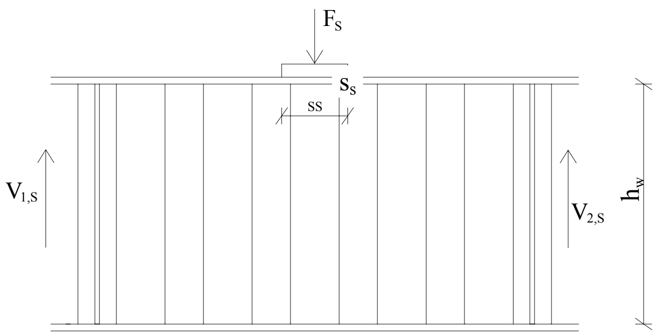

Fig. 1 – Loading type.

In case of girders with inclined webs, the internal forces to be taken into account are the components of the external load in the plane of the web.

The design resistance of the trapezoidally corrugated web to local buckling under transverse force should be taken as:

$$F_{Rd} = \chi \cdot t_w \cdot f_{yw} \cdot s_s \cdot k_\alpha. \tag{16}$$

k_α is the modification factor due to corrugation angle, which should be calculated from, Fig. 2.

$$k_\alpha = \frac{a_1 + a_2}{a_1 + a_4}. \tag{17}$$

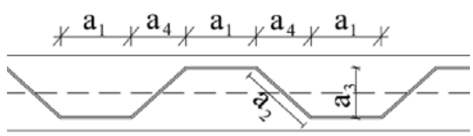

Fig. 2 – Corrugated web geometry.

χ is the reduction factor due to local buckling which should be calculated from:

$$\chi = \frac{1.9}{\bar{\lambda}_p} - \frac{0.798}{\bar{\lambda}_p^2} \leq 1.00, \tag{18}$$

where

$$\bar{\lambda}_p = \sqrt{\frac{f_{yw}}{\sigma_{cr}}} \tag{19}$$

$$\sigma_{cr} = \frac{k_\sigma \cdot \pi^2}{12 \cdot (1 - \upsilon^2)} \cdot E \cdot \left(\frac{t_w}{a_i} \right)^2 \tag{20}$$

$$k_\sigma = 1.11, \; a_i = \max(a_1; a_2), \tag{21}$$

i.e. the maximum fold length to which the load is applied (Fig. 2).

The design method is applicable for girders with a fold length larger than

$$a_i \geq \left(\frac{h_w}{t_w} + 260 \right) \cdot \frac{t_w}{11.5}. \tag{22}$$

5. OUTLOOK

5.1. M-V INTERACTION OF LONGITUDINALLY STIFFENED PLATE GIRDERS

The M-V interaction formula of Section 7.1, EN 1993-1-5, is an extension of the plastic interaction to slender cross-sections. It covers the load bearing behaviour general but has only been checked for longitudinally unstiffened panels

[19]. The current M-V interaction formula is based on an empirical model which was developed on the basis of only few experimental results and that assigns a pure shear loading to the web-core of a symmetric cross-section. Recent experimental work regarding the interaction of bending moment and shear force for longitudenally stiffened girders was conducted in [20] opening some additional questions concerning M-V interaction for stiffened girders.

In the frame of [21] six full scale girders were tested and served as a database for subsequent numerical analyses. It was shown that the gross cross-section check at the edge of the panel can cover also the stability resistance of the panel subjected to M-V interaction in the case of symmetric cross-sections. For unsymmetric cross-sections a new proposal has been derived which, in combination with the gross cross-section check, gives very reliable results. However, the partial safety factor determination is a crucial part in this work. Thus, the amendment will cover not only M-V interaction but also basic issues with respect to partial safety factor determination.

5.2. DESIGN OF INTERMEDIATE TRANSVERSE STIFFENERS

In general, transverse stiffeners at girder ends and at intermediate supports are designed as strong double-sided stiffeners. In contrast to this, intermediate transverse stiffeners are designed as single-sided open-section stiffeners which are intended to increase the strength and the stiffness of the web. The latter kind of intermediate stiffener is usually not subjected to external loads. Instead effects from tension field action and deviation forces from longitudinal stresses dominate. If such a design is based on EN 1993-1-5, the sizing leads to significantly larger cross-sections in comparison to what is necessary according to numerical analysis. This can be mainly attributed to the overestimation of the axial force in the stiffener due to tension field action [22, 23].

Further research has been carried out [24] whether a stiffness-only approach is justifiable in contrast to a combined approach which checks stiffness and force. It can be shown that the design requirements of a rigid intermediate transverse stiffener may be obtained by fulfilling simple stiffness criteria which simplifies the design procedure while considering all relevant effects. The amendment will define a minimum required second moment of area for rigid transverse stiffeners which covers the design requirements imposed by the different loadings.

5.3. TRANSVERSE BENDING MOMENTS IN CORRUGATED WEB GIRDERS

In corrugated web girders which are subjected to combinations of shear force and bending moment, transversal bending moments M_z occur. Current research studies the effect of transverse bending moments in both bridge and building structures. In particular the influence of support conditions and of M_z on the bending resistance is investigated [25]. Only few reports have dealt with the effect of M_z on the bending resistance so far [26].

Therefore, a large number of different girder geometries with trapezoidal and sinusoidal web shape have been analysed numerically. The results show that the influence of M_z is more dominant for bridges than for building structures. In general, support conditions play an important role. In comparison, Eurocode rules give a reduction in bending resistance that is up to seven times higher than the maximum reduction identified in the numerical analyses. It can be concluded that the reduction factor for bending resistance is negligible with respect to ultimate behavior. However, when first yielding is considered as the limit, the reduction can be significant. Thus, a formula which determines the additional stresses in an elastic analysis is under preparation. The amendment will be fully addressed in EN 1993-1-5 since it is the only part in Eurocode 3 where corrugated web girders are dealt with.

5.4. APPLICATION OF THE REDUCED STRESS METHOD

EN 1993-1-5 provides two methods for the design of plated structures: effective width method and reduced stress method. The reduced stress method has been introduced at a very late stage of the development of current EN 1993-1-5 as an alternative verification method to the effective width method. In [27] the background to this method has been outlined and shortcomings with respect to the biaxial compression state have been discussed. It is shown that the reduced stress method offers advantages over today's numerical procedures which allow the elastic critical load factor of the full stress field to be determined in a single step, thus taking interaction into account in a very efficient way. However, a modification has been proposed for biaxially compressed plates in [11] since the interaction verification in its pure format based on the Von Mises yield criterion is not able to represent the actual mechanical behaviour. The proposal leads to appropriate and plausible results without over-complicating the interaction equation.

In general, the reduced stress method offers the possibility to consider the effect of tensile stresses in plates with multiaxial stress state. However, tensile stresses have not only a stabilizing effect on buckling, but also a plastic destabilizing influence. In [28] first results how tensile stresses influence the buckling behavior are presented. This work will be further developed and combined with the research on biaxially compressed plates into the common amendment which will improve the reduced stress method in its actual format.

6. CONCLUSIONS

The decision within ECCS/TWG83 group and Working Group EN 1993-1-5 to prepare as many amendments as early as possible has already generated an amount of considerable amendments for the further development of EN 1993-1-5. These amendments have been presented in this paper in a similar way as presented to CEN/TC250/SC3 for decision and gained the preliminary acceptance. Thus they can be easily implemented in the future code when it will be developed in the frame of CEN/TC250 Mandate M/515 EN. A large number of proposals is still to come including the topics summarized in the outlook and the tasks formulated in Mandate M/515 EN.

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SYMBOLS AND ABBREVIATIONS

α	$= \dfrac{a}{b}$; aspect ratio of length a and width b
α_{F0}	reduction curve parameter
δ	$= \dfrac{\sum A_{sl}}{b \cdot t}$; ratio of gross area sum of longitudinal stiffeners ΣA_{sl} and plate gross area
ε	$= \sqrt{\dfrac{235}{f_y \left[\mathrm{N/mm^2}\right]}}$
γ	$= \dfrac{E \cdot I_s}{b \cdot D}$; relative bending stiffness of a stiffener
γ_{M1}	partial safety factor
$\overline{\lambda}_F$; $\overline{\lambda}_p$; $\overline{\lambda}_p$	slenderness parameters
$\overline{\lambda}_{F0}$	reduction curve parameter
ν	Poisson's coefficient
σ_{cr}	elastic critical buckling stress
ψ	$= \dfrac{\sigma_2}{\sigma_1}$; stress ratio of edge stresses σ_1 (larger) and σ_2 (smaller)

χ; χ_F; χ_w	reduction factors
a	plate length
a_c	$= 4.33 \cdot \sqrt[4]{\dfrac{I_{sl,1} \cdot b_1^2 \cdot b_2^2}{t^3 \cdot b}}$; plate parameter
$A_{\mathrm{sl},1}$	gross area of a longitudinal stiffener
b	plate width
b_1; b_2	subpanel widths
b_c	plate width under compression
b_f	flange width
$b_{\mathrm{sl},1}$	distance between stress neutral axis and longitudinal stiffener
D	$= \dfrac{E \cdot t^3}{12 \cdot (1-\nu^2)}$; plate bending stiffness
E	Young's modulus
f_{yw}	web yield strength
h_w	web height
$I_{sl,1}$	second moment of area of a longitudinal stiffener
I_s	second moment of area of longitudinal stiffener for out-of-plane bending, its cross-section including a participating width of web of $10{\cdot}t$ each side of each stiffener-to-web junction
l_y	effective patch loaded length
k_σ; $k_{\sigma,p}$; k_τ	buckling coefficients
m_1; m_2	patch loading parameters
M_{Ed}	design bending moment
$M_{\mathrm{pl},Rd}$	design plastic moment resistance (irrespective of cross-section class)
s_s	patch loading length
t	plate thickness
t_f	flange thickness

t_w web thickness

V_{Ed} design shear force

$V_{\mathrm{bw,Rd}}$ design shear resistance of the web contribution

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