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Steel plate girders design aspects according to EC3 & KTP-78

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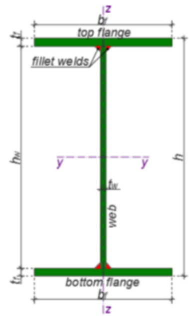
Abstract:

This paper presents the design procedures related to bending, shear, and in particular, local buckling resistance of Steel Plate Girders according to the requirements of Eurocode 3 and Albanian National Design Code, KTP-78. At this aim, a steel plate girder is analyzed. Important conceptual differences, but not only, are discussed. The approach of both codes for the first two design aspects is basically the same, while meaningful and non-negligible differences can be noticed regarding buckling resistance. Eurocode 3 accounts for the web's post critical/ buckling resistance and its evaluation are based on the "Rotated stress field theory" proposed by T. Höglund, while KTP-78, even because of preparation and publication time period, is based on classic studies over plate stability, not considering the web's post critical contribution, so resulting conservative. The differences presented can serve as indications and arguments for further related technical discussions, also at the aim of preparation of National Annexes in function of the short time implementation of Eurocode requirements in Albania.

Key words: Steel plate girder, Eurocode 3, KTP-78, bending, shear & local buckling resistance, Effective width method, Rotated stress field theory.

1. INTRODUCTION

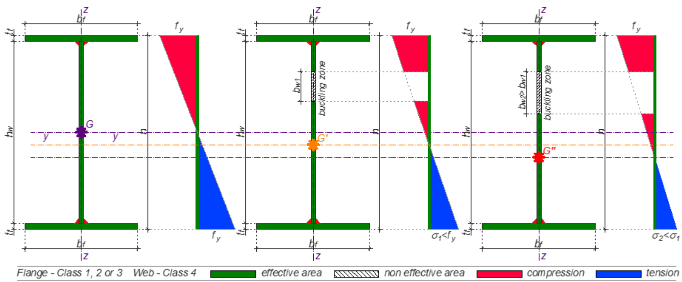
Steel Plate Girders (SPG) are structural elements, usually composed by at least three steel plates, welded or bolted together, forming a deep I-section. They can be used in different types of structures, civil and industrial ones, and also as bridge components. Section dimensions, defined in order to satisfy the loads demands in terms of bending moments and shear forces, can vary along the girder's length. Several important aspects characterize the design process of these complex elements, like the ones listed, related to the ULS: bending and shear resistance, lateral torsional buckling and plates local buckling. This paper is focused on the bending, shear and local buckling resistance, aspects that will be discussed according to Eurocode 3 requirements and Albanian National Structural Codes, known as "Kushte Teknike të Projektimit" (KTP), aiming to show the most important qualitative and quantitative differences.



2. EUROCODE 3 DESIGN APPROACH

2.1 Bending resistance

The design of structural steel elements, in general, is closely related to the cross section classification, whose scope is to identify the extent to which the resistance and rotation capacity of cross-section is limited by its local buckling resistance [1]. Four classes of cross-sections are defined according to Eurocode 3. On this basis the flanges of Steel Plate Girders are classified as Class 1, or 3 and the web, almost inevitably Class 4 (Class 4 cr.-sections are those in which local buckling will occur before the attainment of yield stress in one or more parts of the cross-section).



The method used to evaluate bending resistance of steel plate girders, is known as "Effective width" method (EWM) [1], [2], [3]. According to this method, part of compressive web becomes ineffective because of local buckling, so a relocation of the centroid occurs. Although compressive flange yielded, tension flange remains elastics at ULS, thus bending resistance is:

$$M_{c,Rd} = W_{eff} f_y / \gamma_{M_0} \qquad (1)$$

W_{eff} - effective elastic section modulus; f_y - steel yield strength.

"Flanges-only" (FOM) method is an alternative to the one discussed. This method is simpler for design and in particular for dimensioning, and it is based on the general assumption that the flanges are mainly responsible for bending induced normal compression/ tension stresses. These cross-section elements are supposed to be fully in plastic at the ULS.

$$M_{f,Rd} = 2b_f t_f \left(h_w/2 + t_f/2\right) f_y = b_f t_f \left(h_w + t_f\right) f_y = A_f \left(h_w + t_f\right) f_y \qquad (2)$$

$M_{f,Rd}$ - design plastic moment of resistance of a cross-section consisting of the flanges only; h_w, t_w, b_f, t_f - section dimensions; A_w, A_f - web/ flange area. Eurocode 3 not explicitly allow or disallow the use of "Flanges - only" method, hence, it is safe to use if:

$$M_{f,Rd} \le M_{c,Rd} \Leftrightarrow A_f \left(h_w + t_f\right) \cancel{f_y} \le W_{eff} \cancel{f_y} \qquad (3)$$

Research done, have shown that Eq. (3) is verified within most design range, meaning that the "Flanges - only" method is conservative, sometimes in excess.

In many cases can be verified that Eq. (4) is true:

$$R = A_f\left(h_w + t_f\right)/W_{eff} \leq 0.75 \quad (4)$$

The "Modified flanges - only" method is suggested to improve the efficiency in design regarding bending resistance [4]. The ratio above can be modified.

$$\left(M_{y,Rd} = W_{eff} f_y\right) \approx \bar{M}_{f,Rd} = \bar{R} A_f\left(h_w + t_f\right) \quad (5)$$

ε=√(235/f_y) - accounts for steel resistance class

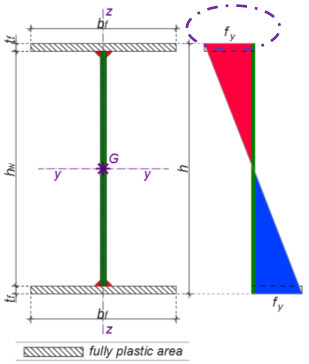


Figure 1. "Flanges - only" method

The bending resistances predicted by this proposed method are in a very close range to those predicted by the more complicated "Effective width" method.

If no axial force is present [1], verification should be performed as below using η_1:

$$\eta_1 = M_{Ed}/M_{y,Rd} \leq 1.0 \quad (6)$$

2.2. Shear resistance

The design value of the shear force V_{Ed} at each cross section shall satisfy [1]:

$$\eta_3 = V_{Ed}/V_{c,Rd} \leq 1.0 \quad (7)$$

$V_{c,Rd}$ - is the design shear resistance for plastic or elastic design. For plastic design $V_{c.Rd}$, in absence of torsion, is evaluated as:

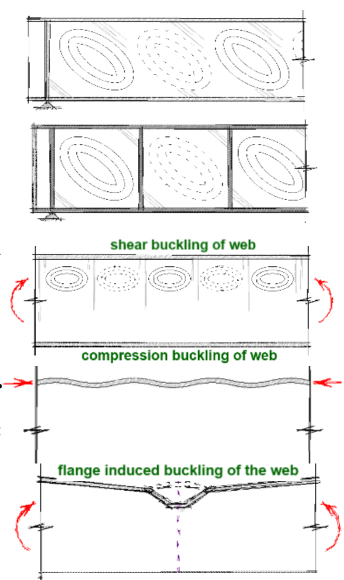


Figure 2. SPG buckling phenomena

$$V_{pl,Rd} = A_v \left(f_y / \sqrt{3} \right) / \gamma_{M0} \qquad (8)$$

A_v - is the shear area - for welded I-sections loaded parallel to the web $A_v = h_w t_w$; γ_{M0} - partial factor for resistance of cross-sections whatever the class is.

For verifying the design elastic shear resistance $V_{c,Rd}$ the following criterion for a critical point of the cross section may be used unless the buckling verification in section 5 of [2] applies; elastic shear τ_{Ed} stress to be according to Dmitrii I. Zhuravskii, Eq. (9). This verification is conservative [1] as it excludes partial plastic shear distribution, which is permitted in elastic design. Therefore it should only be carried out where the verification on the basis of $V_{c,Rd}$ according to Eq. (7) cannot be performed.

$$\tau_{Ed} / \left[f_y / \left(\sqrt{3} \gamma_{M0} \right) \right] \le 1.0, \ \tau_{Ed_{el.}} = \frac{VS}{I \cdot t}, \ \tau_{Ed_{pl.}} = \frac{V_{Ed}}{A_w}, \text{if} \left(\frac{A_f}{A_w} \ge 0.6 \right) \qquad (9)$$

2.3 Local buckling resistance

Various forms of plate buckling must be taken into account: local buckling of the compression flange, compression & shear buckling of the web, flange induced buckling of the web and local buckling of the web due to vertical loads, Fig. 3.

To prevent flange induced buckling according [2]:

$$\frac{h_w}{t_w} \le k \frac{E}{f_{yf}} \sqrt{\frac{A_w}{A_{fc}}} \qquad (10)$$

k=0.3 ÷ 0.55 - reduction factor; A_w - area of the web; A_{fc} - effective cross-section area of the

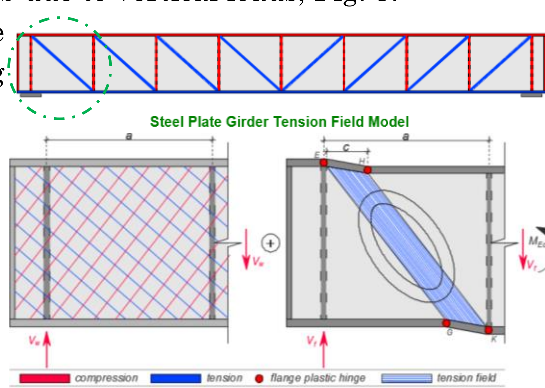


Figure 1. Tension field carried by bending resistance of flanges

compression flange; E - Young's modulus.

Shear buckling resistance presents the most important aspect among the others listed. Unless the web is Class 1, it is necessary to check whether the web is stocky or not for shear buckling, for unstiffened/ stiffened case. Equation (11) and (12) apply:

$$h_w/t_w \le 72\varepsilon/\eta \quad h_w/t_w \le 31\sqrt{k_\tau}\,\varepsilon/\eta \tag{11/ 12}$$

k_τ - minimum shear buckling coefficient in the web panel, values on Annex A.3, [2].

If the web is stocky, no shear buckling of web shall occur and the shear strength of the web is given by Eq. (8). Otherwise, shear buckling can occur. Eurocode 3 [1], [2], considers the web's post-buckling resistance and its evaluation are based on the "Rotated stress field theory" proposed by T. Höglund, Fig. 4. Design shear is:

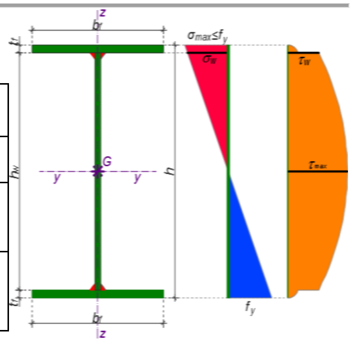
$$V_{b,Rd} = \underbrace{V_{bw,Rd}}_{\text{web contribution}} + \underbrace{V_{bf,Rd}}_{\text{flange contribution}} \le \eta f_{yw} h_w t_w \Big/ \left(\sqrt{3}\gamma_{M1}\right) \tag{13}$$

$$V_{bw,Rd} = \frac{\chi_w f_{yw} h_w t_w}{\sqrt{3}\gamma_{M1}} \qquad V_{bf,Rd} = \frac{b_f t_f^2 f_{yf}}{c\gamma_{M1}}\left(1-\left(\frac{M_{Ed}}{M_{f,Rd}}\right)^2\right) \tag{14/ 15}$$

χ_w - reduction factor for the shear resistance of the sole web, depending on its slenderness; λ_w - modified slenderness of the web; $\gamma_{M1}=1.0$ - partial factor for the resistance to instability; η - coefficient that includes the increase of shear resistance at smaller web slenderness $\eta=1.2$, S235; $M_{f,Rd}$ - moment of resistance of effective area of the flanges only (as in the "flanges only" method); c - see Fig. 4.

Table 1. Contribution from the web χ_w to shear buckling resistance.

	Rigid end post	**Non-rigid end post**
$\bar{\lambda}_w < 0.83/\eta$	η	η
$0.83/\eta \le \bar{\lambda}_w < 1.08$	$0.83/\bar{\lambda}_w$	$0.83/\bar{\lambda}_w$
$\bar{\lambda}_w \ge 1.08$	$1.37/(0.7+\bar{\lambda}_w)$	$0.83/\bar{\lambda}_w$



The web's modified slenderness is given for transverse stiffeners at supports only or for the presence of end and intermediate transverse stiffeners respectively:

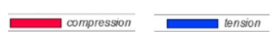
$$\bar{\lambda}_w = h_w / 86.4 t_w \varepsilon \qquad \bar{\lambda}_w = h_w / 37.4 t_w \varepsilon \sqrt{k_\tau} \qquad (16/\ 17)$$

Flange plastic hinge distance, almost the width of the tension field is :

$$c = a\left(0.25 + 1.6 b_f t_f^2 f_{yf} / t_w h_w^2 f_{yw}\right) \qquad (18)$$

2.4. Interaction of bending and shear

Interaction of bending and shear stresses should be considered only if certain conditions are fulfilled. So, the resistance to bending moments need not to be reduced to account for shear



stresses action if:

Fig. 5 Elastic normal/ shear

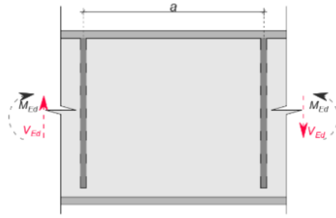
$$\left(\bar{\eta}_3 = \frac{V_{Ed}}{V_{bw,Rd}} \le 0.5 \wedge M_{Ed} < M_{f,Rd}\right)$$

(19)

Otherwise, addition verification are needed and precisely it is necessary that:

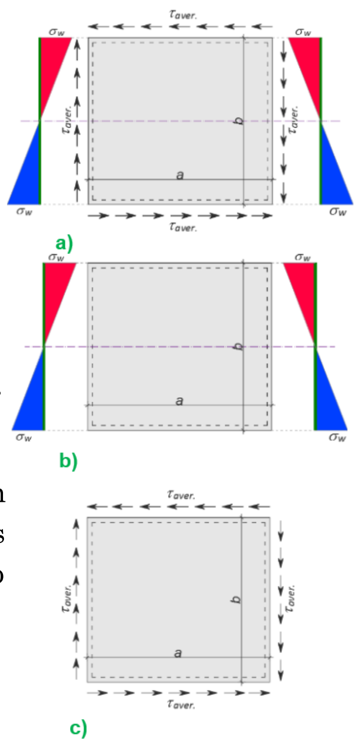
$$\left(\bar{\eta}_1 = M_{Ed}/M_{pl,Rd}\right) + \left(1 - M_{f,Rd}/M_{pl,Rd}\right)\left(2\bar{\eta}_3 - 1\right)^2 \le 1.0 \qquad (20)$$

$M_{pl,Rd}$ - is the design plastic resistance of the cross section consisting of the effective area of the flanges and the web (every class).



3. KTP-10-78 DESIGN APPROACH

Albanian National Structural Codes known as "Kushte Teknike të Projektimit" (KTP) [5], were written in 1978, basing mainly on the literature and Codes of the former Soviet Union. Their last important but partial review was made on 1989, and during the last almost 25 years no serious efforts were made in order to update their content in relation with the civil engineering scientific and technical developments. In order to have a better interpretation possibility, the terminology of KTP is approximated/ translated according to Eurocode 3 with any needed comment.

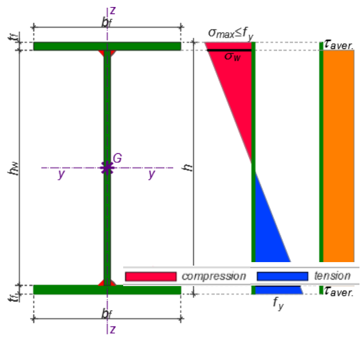


3.1. Bending resistance

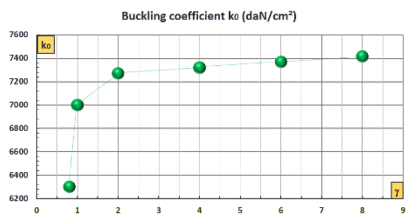
Bending resistance of Steel Plate Girders can be evaluated as for any other steel element, in the same working conditions, on basis of normal stress elastic distribution [5]~[8]. The extreme fibers of the cross-section should in maximum reach yielding values ≈ f_y. This expression might look the same as the one based on "Effective width" method, but very significative conceptual differences exist. Verification according to Eq. (6).

$$M_{c,Rd} = \left([M] = W_{elastic} R \right) \quad (21)$$

3.2. Shear resistance



Shear resistance refers to elastic shear resistance [5]~[8]. Elastic shear τ_{Ed} for a critical point of the cross section can be evaluated according to Dmitrii I. Zhuravskii, Eq. (9). Usually this verification is made only for the web fiber in correspondence with the horizontal neutral axis position. Verification should be performed according to Eq. (7) where $V_{c,Rd}$ is the elastic design shear resistance, or comparing the maximum stress τ_{max} to R_{pr}.

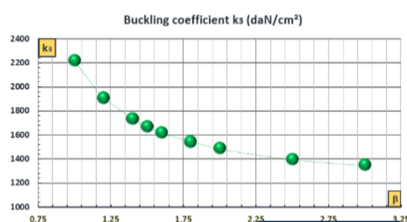


$$\tau_{max} = VS/I \cdot t \leq \left(R_{pr} \approx 0.6R \right) \quad (22)$$

R_{pr} - shear strength, where R (as resistance from Russian literature) is the steel compression/ tension strength: $R = k \cdot R^n$, and k=0.9 - homogeneity factor; $R^n \approx f_y$.

Interaction of bending and shear stresses should be in practice considered only for the flange-web boundary fiber, Fig. (5), according to Strength of Material theory:

$k_0 = f(\gamma)$ $\gamma = (c = 0.8) \cdot (b_f/h_w) \cdot (t_f/t_w)^3$



$k_3 = f(\beta)$ $\beta = \max(a;b)/\min(a;b)$

$$\sqrt{\sigma_w^2 + 3\tau_w^2} \leq R \quad (23)$$

Fig. 12. Web buckling

3.3. Local buckling resistance

Evaluation of local buckling is based on S. Timoshenko work related to elastic plate buckling. KTP-78 does not consider post-buckling resistance of steel plate elements, so the main issue to be discussed is how to define the critical stress value, σ_o (σ_{cr}) and τ_o (τ_{cr}). Plates can have different loading and support conditions. In analogy with EC3, few of them related to the web plate are presented, Fig. 5, for the case of SPG with IT stiffeners only (longitudinal).

Table 2. Shear resistance function of the web.

analog with $\varepsilon=\sqrt{235/f_y}$

Case	Verification	Not necessary if:	Description
a	$\sqrt{\left(\frac{\sigma}{\sigma_0}\right)^2+\left(\frac{\tau}{\tau_0}\right)^2}\le m$	$\frac{h_w}{t_w}=110\sqrt{\frac{2100}{R}}$	$\sigma_0=k_0\left(100\frac{t_w}{h_w}\right)^2$ τ_o - as in Case c)
b	$\frac{\sigma}{\sigma_0}\le m,\ (\sigma_0\Leftrightarrow\sigma_{cr})$	$\frac{h_w}{t_w}=170\sqrt{\frac{2100}{R}}$	$\sigma_0=6300\left(100\frac{t_w}{h_w}\right)^2$
c	$\frac{\tau}{\tau_0}\le m,\ (\tau_0\Leftrightarrow\tau_{cr})$	$\frac{h_w}{t_w}=100\sqrt{\frac{k_3}{0.67R_{pr}}}$	$\tau_0=k_3\left(100\frac{t_w}{h_w}\right)^2$

m - loading conditions coefficient: m=1.0 - static loading, m=0.9 - dynamic loading

4. CASE STUDY

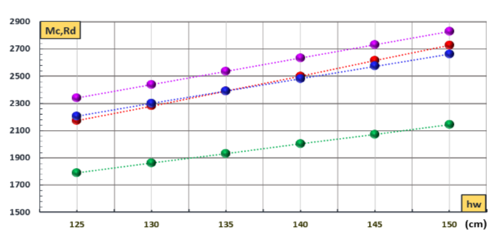
Table 3. Case 1

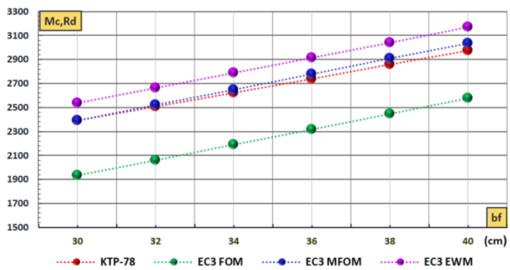
h (cm)	129÷154
hw (cm)	125÷150
tw (cm)	1
bf (cm)	30
tf (cm)	2

Table 4. Case 2

h (cm)	139
hw (cm)	135
tw (cm)	1
bf (cm)	30÷40
tf (cm)	2

EC3 & KTP-78, are different in many aspects, even conceptual, and this fact makes it difficult to formulate discrete comparative case studies. For this reason only some aspects are discussed. Important conclusions, from a qualitative and quantitative



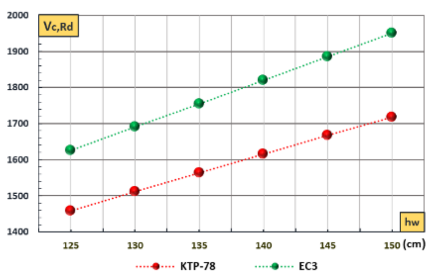


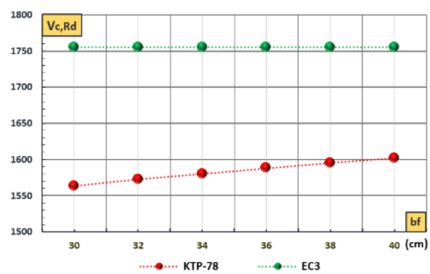
point of view, can be formulated basing in parametric cases.

At this aim, a welded I-section SPG, steel resistance class S235, is analyzed related to bending & shear resistance, and also web buckling resistance, varying its depth or flange width (Table 3, 4), or for fixed dimensions (3rd case, h=139cm, h_w=135cm, t_w=1cm, b_f=30cm, t_f=2cm), varying the values of design bending moment M_{Ed} expressed in part of bending resistance accord. to FOM, $M_{f,Rd}$.

5. Comparison of results

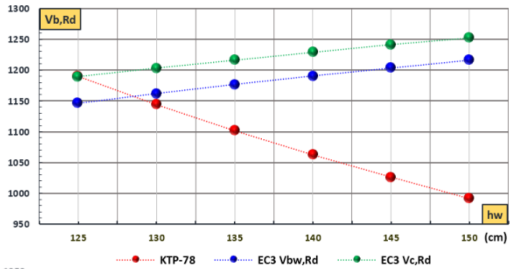
Results are presented in a synthetic graphical form, referring to the same terms, no matter to their conceptual base. Bending resistance according to KTP-78 is quite the same with the proposed "Modified Flanges - only" method, but conservative compared to the "Effective width" method.

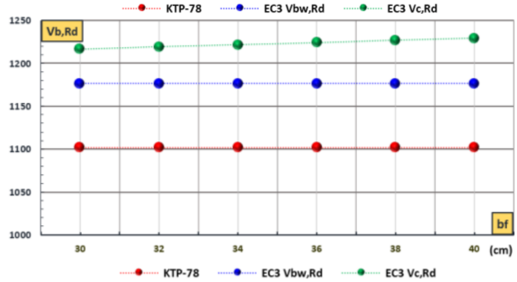


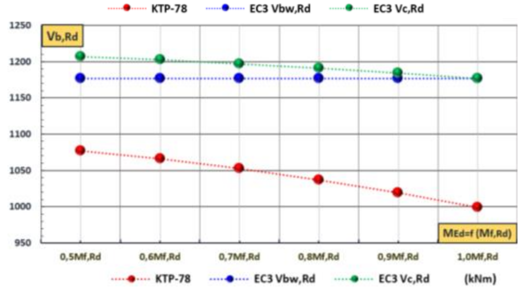


The results for shear resistance, evaluated using Eqs. (8) & (22), clearly confirm that elastic shear stress distribution is very conservative compared to the plastic one. The first two charts present the variation of web shear buckling resistance, for shear action only (bending moment). A rapid and quite constant decrease of shear resistance according to KTP-78 can be noticed with the increase of depth (h).

The opposite is true for EC3 values. The 3rd chart, in correspondence to the 3rd case, shows the web shear buckling resistance for different values of design bending moment M_{Ed}. KTP-78 values, (according to Table 2, Case a)), are again well conservative compared to the EC3 ones.







CONCLUSIONS

Important differences related to SPG design, can be noticed between Eurocode 3 and KTP-78 - they extend in time and concept, and should not be neglected. These new concepts that do somehow represent new studies, research procedures and technologies, must be implemented in Albanian design practice, not only for the focus -element of this paper.

The design based on KTP-78 requirements is in general safe and conservative, even more than needed. These conclusions can serve as indications and arguments for further related technical discussions, also at the aim of the short time implementation of Eurocode requirements in Albania.

REFERENCES

1. Eurocode 3, Design of steel structures - Part 1-1: General rules and rules for buildings, European Standard, CEN, Brussels, 2004.
2. Eurocode 3, Design of steel structures - Part 1-5: General rules - Plated structural elements, European Standard, CEN, Brussels, 2004.
3. Johansson B., Maquoi R., Sedlacek G., Müller C., Beg D.: Commentary and worked examples to EN 1993-1-5 "Plated structural elements", CEN - JRC, Brussels, 2007.
4. Lee C.K., Chiew S.P.: "An efficient modified flanges only method for plate girder bending resistance calculation", Journal of Constructional Steel Research, Volume 89, 2013.
5. Kushte Teknike të Projektimit, Libri III, KTP-10-78: Llogaritja e konstruksioneve prej çeliku Tiranë, 1978.
6. Pistoli V.: Konstruksione Metalike dhe Druri - Bazat e llogaritjes, Pjesa I & II, Tiranë, 1981
7. Pistoli V., Lako N.: Konstruksione Metalike dhe Druri - Konstruksionet, Tiranë, 1981
8. Lako N.: Konstruksione Metalike - Shembuj Numerikë II, Tiranë, 1981