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SLENDER GIRDERS LONGITUDINALLY STIFFENED

BY

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Abstract. This paper presents several aspects regarding the bending resistance and the shear buckling resistance of a slender plate girder having a longitudinal stiffener in the compression zone of the web.

This paper analyzes the influence that the rigidity of the longitudinal stiffener has on the bending resistance of a slender girder whose web belongs to Class 4 sections, starting from the hypothesis that the transversal stiffeners are infinitely rigid and then considering an elastic behavior of the longitudinal stiffener.

Also, the paper analyzes the influence of the longitudinal stiffener on the shear buckling resistance of the girder cross-section.

A numerical analysis is presented and some final conclusions are mentioned.

Keywords: steel plate girders; longitudinal stiffener; bending resistance; shear buckling resistance; Eurocodes.

1. Introduction

Steel plate girders with webs made by thin plates falling into the Class 4 section are frequently utilized in steel structures because they are economically efficient.

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Because some part of the web is buckled when the limit stresses are reached, this zone becomes ineffective, but it remains important with regard to the rigidity of the girder.

By disposing one or more longitudinal stiffeners the girder web will be divided in sub panels of a lower section class so that the ineffective zones can be diminished or eliminated.

In this paper we carry out a comparative analysis regarding the bending resistance of a Class 4 steel plate girder more exactly, of a girder without a longitudinal stiffener as compared to the same girder with a longitudinal stiffener of different compression stiffness. A simplified method to evaluate the bending resistance of the girder, useful in the pre-design is proposed.

Also, the shear buckling resistance of this girder cross-section is analyzed in the hypothesis of unstiffened girder and in the hypothesis of a stiffened girder with a longitudinal stiffener.

2. Slender Plate Girders

In so far the taking over of the normal unit bending stresses σ is concerned, plate girders can be considered as having high slender features, when they range in Class 4 of cross sections, according to norm EC3-1-1 (SR EN 1993-1-1/2006. Eurocod 3, Part 1-1) for which local stability is lost before the yield limit is touched.

The plate is contained in cross section Class 4, if its slenderness given by ratio c/t_w, Fig. 1, satisfies the conditions:

$$s = \frac{c}{t_w} \geq \begin{cases} 124 \cdot \varepsilon & \text{– Fig. 1.b} \\ 42 \cdot \varepsilon / \left(0{,}67 + 0{,}33\Psi\right) & \text{– for } \Psi > -1 \\ 62 \cdot \varepsilon \left(1-\Psi\right)\sqrt{\left(-\Psi\right)} & \text{– for } \Psi \leq -1 \end{cases} \tag{1}$$

where: $\varepsilon = \sqrt{235/f_y}$; Ψ – according to Fig. 1.c.

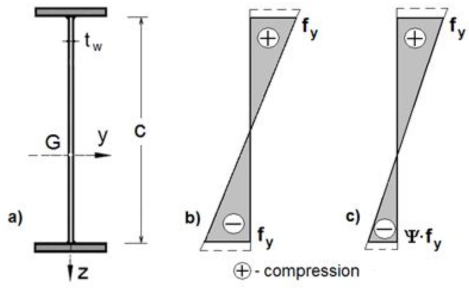

Fig. 1 – Diagram of normal unit stresses σ on plate girder web.

The shear strength and the shear buckling phenomenon, according to the theory of the simple post-critical method of evaluating the plate buckling resistance, ranges in three categories of thickness, Fig. 2 (European Steel Design Education Programme).

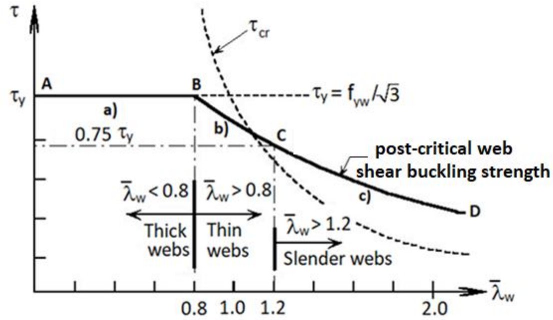

Fig. 2 – Classification of plates function of reduced slenderness

The plate slenderness $\bar{\lambda}_w$ is assessed with the relationship:

$$\bar{\lambda}_w = \sqrt{\frac{f_{yw}/\sqrt{3}}{\tau_{cr}}} = \frac{h_w/t_w}{37{,}4 \cdot \varepsilon \cdot \sqrt{k_\tau}} \tag{2}$$

where: f_{yw} is the yield limit of the plate steel; $\tau_{cr} = \dfrac{\pi^2 E}{12(1-\mu^2)} \cdot k_\tau \cdot \left(\dfrac{t_w}{h_w}\right)^2$ – is the critical buckling resistance in the linear elastic range; k_τ – the shear buckling coefficient.

3. Girder bending resistance and shear buckling resistance

3.1. Girder bending resistance

According to European norms EC3-1-1, the theoretical bending resistance of a steel girder subjected to pure bending $M_{c.Rd}$ is found as follows:

$$M_{c.Rd} = \begin{cases} \dfrac{W_{pl} f_y}{\gamma_{MO}} & \text{– Classes 1 and 2} \\ \dfrac{W_{el.\min} f_y}{\gamma_{MO}} & \text{– Class 3} \\ \dfrac{W_{eff.\min} f_y}{\gamma_{MO}} & \text{– Class 4} \end{cases} \tag{3}$$

In the case of Class 4 cross sections, the inclusion of longitudinal stiffeners in the compressed area can have a favorable effect for the girder bending strength, by diminishing the plate inactive area.

The increase of the bearing capacity is more significant if the contribution of longitudinal stiffening to the increase of the strength module of the girder section is considered.

The longitudinal stiffening consists of a bar compressed on the elastic bed, whose section is made up of a longitudinal stiffener itself and the areas where the two adjacent panels work together as shown in Fig. 3.c (when the stresses in the lower panel change their sign).

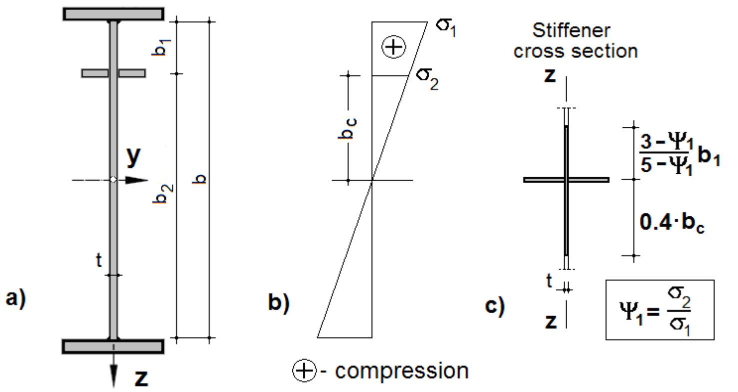

Fig. 3 – Longitudinal stiffener cross section.

To calculate the bearing capacity of the girder having a longitudinal stiffener in the compressed area, the following algorithm is proposed (Guțiu & Moga, 2014):

a. The overall longitudinal stiffening reduction factor, ρ_c, is calculated by interpolating χ_c and ρ, in the interaction between the plate buckling and column buckling types in conformity with EN 1993-1-5 §4.5 and with relationship:

$$\rho_c = (\rho - \chi_c)\,\xi\,(2 - \xi) + \chi_c \tag{4}$$

b. The bearing capacity of the longitudinal stiffener is calculated as an elastic environment compressed bar, necessary to size transverse stiffening so that it has enough stiffening to set up nodal lines (with null displacements):

$$N_{c.Rd} = \rho_c \cdot \frac{A_{s\ell.1.eff} \cdot f_y}{\gamma_{M1}} \tag{5}$$

c. The average stress in the stiffening $\sigma_{com.Ed}$ is checked, and it is required to have a value under $\rho_c \cdot f_{yd} = \rho_c \cdot f_y / \gamma_{M1}$ (EN 1993-1-5, Annex A - §A.2.1(4)), respectively:

$$\sigma_{comEd} \leq \rho_c \cdot f_{yd} \tag{6}$$

where:

$$\sigma_{com.Ed} = \frac{M_{Ed}}{I_{eff}} z_{rigid.}; \quad z_{rigid.} = b_c$$

The resistance bending moment of the girder can be calculated by putting the condition that in the longitudinal stiffener the stress does not exceed $\rho_c \cdot f_y / \gamma_{M1} = \rho_c \cdot f_{yd}$, Fig. 4.

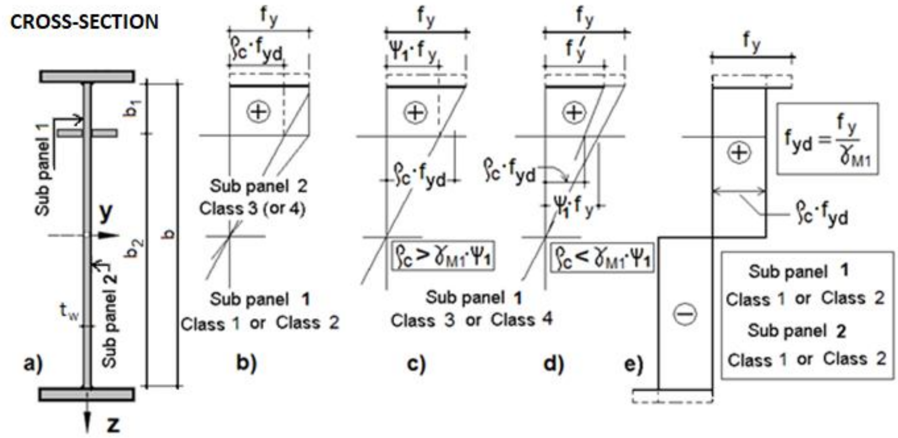

Fig. 4 – Diagrams for evaluating the section bending strength.

Case 1: Upper sub panel, Class 1 or Class2

Considering the unit stress distribution given in Figure 5, where the stress in the longitudinal stiffener is limited to value $\rho_c{\cdot}f_y/\gamma_{M1}{=}\rho_c{\cdot}f_{yd}$, there yield areas with behavior in the plastic range. It yields:

$$M_{c.Rd}^{r} = \frac{W_{el-pl}^{(2)} \cdot f_y}{\gamma_{M0}} \tag{7}$$

where: $W^{(2)}_{el-pl} = A_f \cdot d_f + A_{w.pl} \cdot d_{w.pl} + \frac{t_w h^2_{w.el}}{6}$ (corresponding to diagram (2) in Fig. 5).

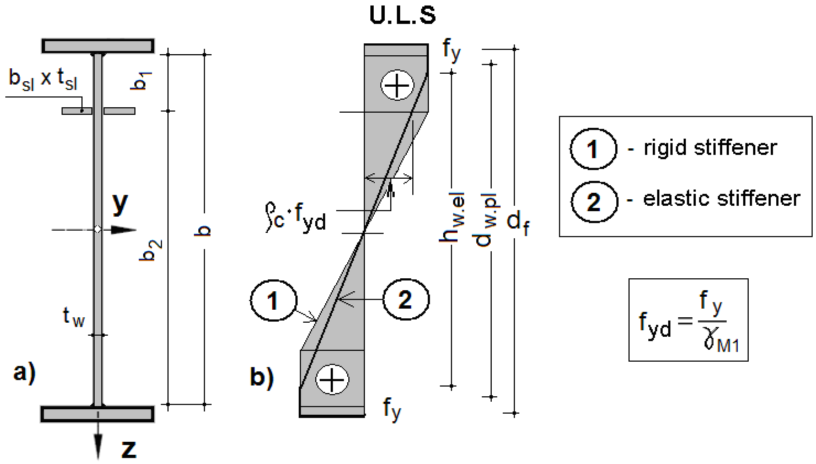

Fig. 5 – Diagrams for evaluating Case 1 bending strength.

Case 2: Upper sub panel Class 3 or Class 4

The bending strength of the section can be calculated with the relationship:

$$M^r_{c.Rd} = \begin{cases} M_{c.Rd} & \text{- for: } \Psi_1 \le \rho_c / \gamma_{M1} \text{ - Figure 4.c} \\ K \cdot M_{c.Rd} & \text{- for: } \Psi_1 > \rho_c / \gamma_{M1} \text{ - Figure 4.d} \end{cases} \tag{8}$$

where: $M_{c.Rd}$ is the resistance of the girder to bending, when the longitudinal stiffening is perfectly rigid; K is the reduction coefficient determined so as stress $\rho_c \cdot f_y / \gamma_{M1} = \rho_c \cdot f_{yd}$ in the stiffener is not exceeded:

$$K = \frac{f_y^{/}}{f_y} = \frac{\rho_c}{\Psi_1} \frac{1}{\gamma_{M1}} \tag{9}$$

When the upper sub panel is Class 3 or 4, it is recommended to result $\rho_c / \gamma_{M1} \ge \Psi_1$ to have a more efficient section.

A favorable effect upon the girder bending resistance occurs when the longitudinal stiffener is taken as girder active cross section part as it is necessary to mix the longitudinal stiffening with the cross-section stiffening, in a proper manner.

3.2. Girder Shear Buckling Resistance

According to norm EC3-1-5 (SR EN 1993-1-5/2006. Eurocode 3, Part 1-5: Plane plate members subjected to stresses in their planes), the buckling bearing capacity is established on the basis of post critical strength theory and the tensile diagonal field.

For stiffened or unstiffened plates, the theoretical shear buckling resistance is calculated with:

$$V_{b.Rd} = V_{bw.Rd} + V_{bf.Rd} \leq \frac{\eta f_{yw} h_w t}{\sqrt{3}\gamma_{M1}} \tag{10}$$

Contribution of the web to shear buckling resistance

The contribution of the web to the shear buckling resistance is assessed with the relationship:

$$V_{bw.Rd} = \frac{\chi_w f_{yw} h_w t}{\sqrt{3}\gamma_{M1}} \tag{11}$$

For the plates with transverse stiffeners only for supports and for plates with transverse or longitudinal stiffeners or both, the factor χ_w for the web contribution to the buckling resistance should be obtained from Table 1.

Table 1

Contribution of the web to shear buckling resistance, χ_w

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

The slenderness parameter $\bar{\lambda}_w$ is determined with the relationship:

$$\bar{\lambda}_w = \begin{cases} \dfrac{h_w}{86{,}4 t_w \varepsilon} & \text{– transverse stiffeners only on supports} \\ \dfrac{h_w}{37{,}4 t_w \varepsilon \sqrt{k_\tau}} & \text{– girders with intermediate stiffeners} \end{cases} \tag{12}$$

For transverse plates and with longitudinal stiffeners, the condition below is verified

$$\bar{\lambda}_w \geq \frac{h_{wi}}{37.4 t\, \varepsilon \sqrt{k_{\tau i}}} \tag{13}$$

where: h_{wi} and $k_{\tau\, i}$ refer to the sub panel with the highest coefficient $\bar{\lambda}_w$, while $k_{\tau\, i}$ is calculated considering $k_{\tau\, st} = 0$.

Contribution of the girder flanges to the section shear buckling resistance

If the flanges are not fully used to take over the bending moment ($M_{Ed} <$ $< M_{f\,Rd}$), then the contribution of the flanges to the buckling resistance can be considered, using the relationship in EC3-1-5 (SR EN 1993-1-5/2006. Eurocode 3, Part 1-5):

$$V_{bf.Rd} = \frac{b_f t_f^2 f_{yf}}{c\, \gamma_{M1}} \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd}} \right)^2 \right]. \tag{14}$$

In general, the contribution of the girder flanges to the section shear buckling resistance is limited and can be neglected.

4. Numerical Analysis

It is determined the bending resistance and the shear buckling resistance of the section of a plate girder (Guțiu *et al*., 2011; Moga *et al.*, 2015).

The girder constructive composition is presented in Fig. 6.

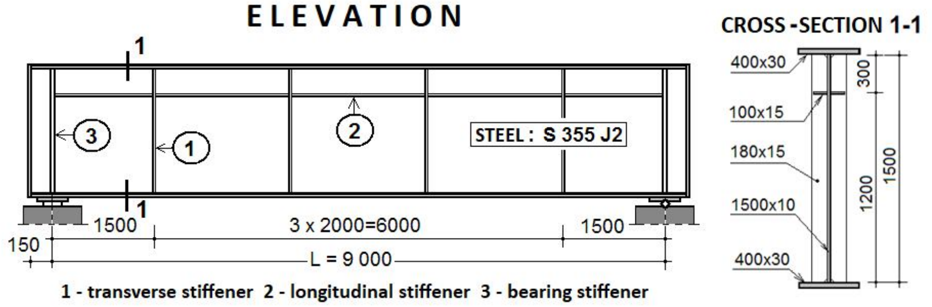

Fig. 6 – Plate girder.

The gross section calculation features are: A_b = 390 cm^2, I_y = 1.686 × 10^6 cm^4, $W_{y.el}$ = 2.161 × 10^4 cm^3, $W_{y.pl}$ = 2.398 × 10^4 cm^4.

4.1. The Girder Computational Bending Resistance

a) Girder without longitudinal stiffening

It yields Class 4 of the cross section; the effective section of the girder and the distribution of the normal unit stresses being given in Fig. 7.

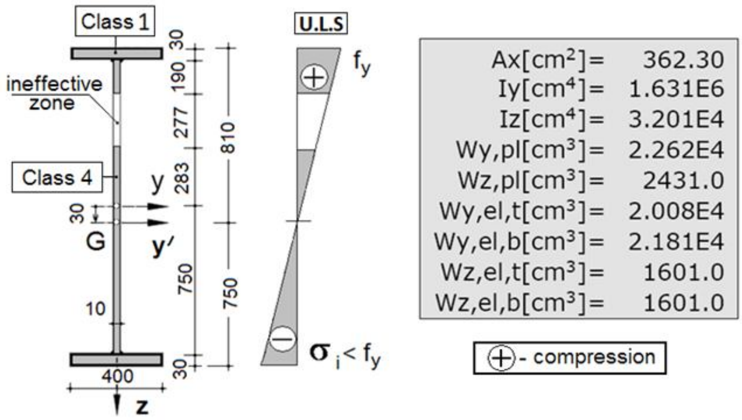

Fig. 7 – Effective zone of unstiffened girder.

The girder computational strength is obtained as:

$$M_{c.Rd} = \frac{W_{eff.min} f_y}{\gamma_{M0}} = \frac{2.008 \cdot 10^4 \cdot 3550}{1.0} \cdot 10^{-4} = 7\,128 \text{ kNm}$$

b) Girder with a longitudinal stiffening, taken as perfectly rigid

The longitudinal stiffening was made towards the middle of the inactive zone, respectively at 300 mm under the compressed flange, Figure 8, at the distance of 0.20·*b*.

By longitudinal stiffening, the plate height is divided in two sub panels, of the sides: $b_1 = 300\,\text{mm}$ (subpanel 1) and $b_2 = 1200\,\text{mm}$ (subpanel 2).

The cross-section Class is found to be Class 2 in the upper sub panel and Class 3 in the lower sub panel.

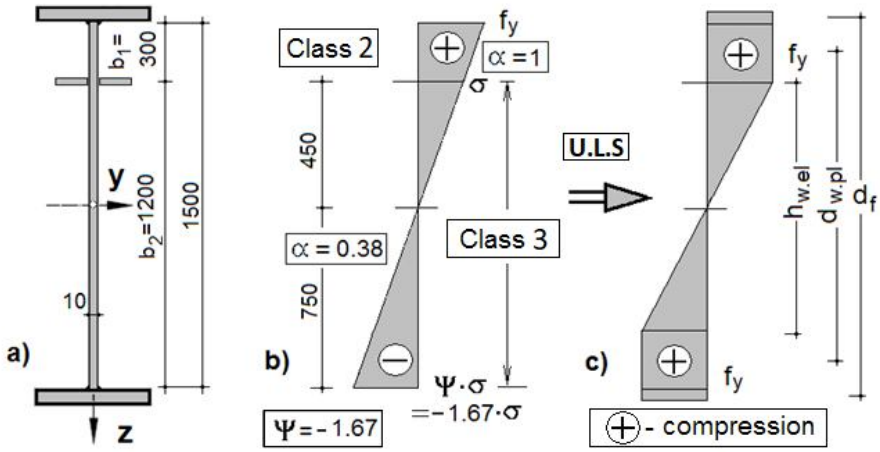

Fig. 8 – Girder with longitudinal stiffener.

Using the normal unit stresses diagram in Fig. 8.c, one can find the section bending strength:

$$M_{c.Rd}^{el-pl} = \frac{W_{el.pl} f_y}{\gamma_{M0}} = \frac{2.331 \times 10^4 \times 3{,}550}{1.0} \times 10^{-4} = 8{,}275\,\text{kN} \cdot \text{m}$$

where:

$$W_{el.pl} = A_f d_f + A_{w.pl} d_{w.pl} + \frac{t_w h_{w.el}^2}{6} = 40 \times 3 \times 153 + 30 \times 1 \times 120 +$$

$$+ \frac{1 \times 90^2}{6} = 23{,}310\,\text{cm}^3.$$

c) Bending resistance of the section taking into account the stiffening flexibility influence upon girder resistance.

Longitudinal stiffening is made with two steel strips fitted on both sides of the plate, of dimensions 100 × 15 mm, Fig. 9, the computational characteristics of the stiffening (steel strips and plate related zones) being presented in Fig. 9 c.

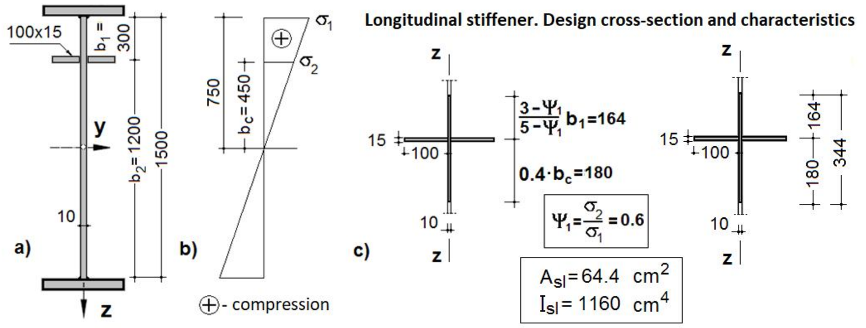

Fig. 9 × Longitudinal stiffening features.

Overall reduction factor ρ_c

Plate type behavior:

– theoretical buckling length:

$$a_c = 4.33\sqrt[4]{\frac{I_{s\ell}\, b_1^2\, b_2^2}{t^3\, b}} = 4.33\sqrt[4]{\frac{1160\times 30^2\times 120^2}{1^3\times 150}} = 433\text{cm} > a = 200\text{cm}$$

– critical buckling stress for $a < a_c$:

$$\sigma_{cr.s\ell.1} = \frac{\pi^2 E\, I_{s\ell}}{A_{s\ell}\, a^2} + \frac{E\, t^3 b\, a^2}{4\pi^2\left(1-\upsilon^2\right) A_{s\ell} b_1^2\, b_2^2} =$$

$$= \frac{\pi^2 2.1\times 10^6\times 1160}{64.4\times 200^2} + \frac{2.1\times 10^6\times 1^3\times 150\times 200^2}{35.9\times 30^2\times 64.4\times 120^2} = 9744\text{daN/cm}^2$$

– critical stress for the plate type behavior:

$$\sigma_{cr.p.1} = \frac{b_c}{b_{st.1}}\sigma_{cr.s\ell.1} = \frac{750}{450}\times 9{,}744 = 16{,}240\,\text{daN/cm}^2$$

– reduced slenderness coefficient:

$$\bar{\lambda}_p = \sqrt{\frac{\beta_{A.c} f_y}{\sigma_{cr.p.1}}} = \sqrt{\frac{1\times 3{,}550}{16{,}240}} = 0.47 < 0.673 \;\Rightarrow \rho = 1 \quad \text{(curve } \boldsymbol{c}\text{)}$$

Column type behavior

– elastic critical unit stress:

$$\sigma_{cr.c.1} = \frac{\pi^2 E I_{s\ell}}{A_{s\ell} a^2} = \frac{\pi^2 2.1\times 10^6 \times 1{,}160}{64.4\times 200^2} = 9{,}324\,\text{daN/cm}^2$$

$$\sigma_{cr.c} = \frac{b_c}{b_{st.1}}\sigma_{cr.c.1} = \frac{750}{450}\times 9{,}324 = 15{,}540\,\text{daN/cm}^2$$

– relative column slenderness:

$$\bar{\lambda}_c = \sqrt{\frac{\beta_{A.c}\cdot f_y}{\sigma_{cr.c}}} = \sqrt{\frac{1\times 3{,}550}{15{,}540}} = 0.48 \;\Rightarrow \chi_c = 0.85 \quad \text{(curve } \boldsymbol{c}\text{)}$$

– overall reduction factor ρ_c:

$$\rho_c = (\rho - \chi_c)\,\xi\,(2-\xi) + \chi_c = 0.86$$

$$\xi = \frac{\sigma_{cr.p}}{\sigma_{cr.c}} = \frac{16{,}240}{15{,}540} - 1 = 0.045 < 1.$$

The limit stress in the stiffening area is obtained:

$$\rho_c f_{yd} = 0.86\times 3{,}550/1.10 = 2{,}775\,\text{daN/cm}^2 = 0.78\cdot f_y$$

The bending resistance can be determined by plotting the unit stresses diagram in the limit state, Fig. 10.

It yields the elastic-plastic resistance module, corresponding to diagram (2):

$$W_{el.pl}^{(2)} = A_f d_f + A_{w.pl}^{(2)} d_{w.pl}^{(2)} + \frac{t_w h_{w.el.(2)}^2}{6} = 22{,}875\ \text{cm}^3.$$

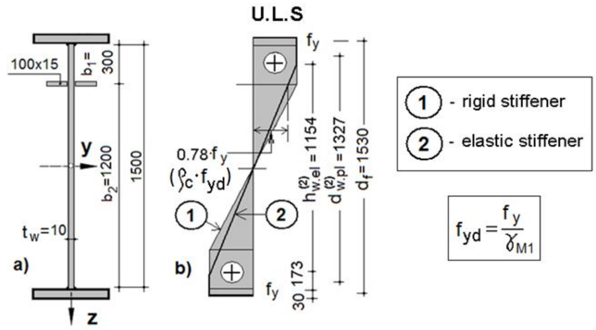

Fig. 10 – Diagram of normal unit stresses.

The girder bending resistance will be:

$$M^r_{c.Rd} = \frac{W^{(2)}_{el.pl} \cdot f_y}{\gamma_{MO}} = \frac{2.2875 \cdot 10^4 \cdot 3550}{1.0} 10^{-4} = 8120 \mathrm{kN \cdot m}$$

4.2. Computational Shear Buckling Resistance of the Section

The influence of the stiffening (vertical and horizontal) upon the shear buckling resistance of the girder plate in the support panel is analyzed.

a) Girder without longitudinal stiffening

$$\frac{h_w}{t_w} = 150 > 31 \frac{\varepsilon}{\eta} \sqrt{k_\tau} = 63.9 \Rightarrow \text{the buckling verification is necessary}$$

where: $k_\tau = 5.34 + 4.00 \left(\frac{h_w}{a} \right)^2 = 9.34$

The computational shear buckling resistance is calculated with relationship:

$$V_{b.Rd} = V_{bw.Rd} + V_{bf.Rd} \le \frac{\eta f_{yw} h_w t}{\sqrt{3}\, \gamma_{M1}}$$

The plate contribution to the value of the shear buckling resistance is assessed with:

$$V_{bw.Rd} = \frac{\chi_w f_{yw} h_w t}{\sqrt{3} \gamma_{M1}} = \frac{0.59 \times 3,550 \times 150 \times 1}{\sqrt{3} \times 1.1} \times 10^{-2} = 1,649 \,\mathrm{kN},$$

where: $\bar{\lambda}_w = \dfrac{h_w}{37.4 t_w \varepsilon \sqrt{k_\tau}} = 1.62 > 1.08 \Rightarrow \chi_w = 1.37/\left(0.7 + \bar{\lambda}_w\right) = 0.59.$

After making the evaluation according to EC3-1-5 (SR EN 1993-1-5/2006, *Eurocod 3)*, the contribution of the base was found $V_{bf.Rd} = 25.6$ kN.

The buckling resistance of the girder section will be:

$$V_{b.Rd} = V_{bw.Rd} + V_{bf.Rd} = 1{,}675 \text{ kN};$$

b) Girder with longitudinal stiffening

The section and computational features of the longitudinal and transverse stiffening (metal strips and plate related areas) are given in Figure11**.**

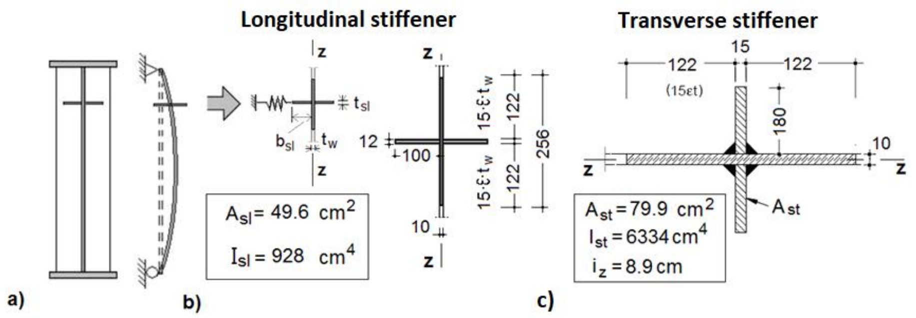

Fig. 11 – Section and features of longitudinal and transverse stiffening.

$$k_\tau = 4.1 + \frac{6.3 + 0.18\dfrac{I_{s\ell}}{t^3 h_w}}{\alpha^2} + 2.2\sqrt[3]{\frac{I_{s\ell}}{t^3 h_w}} = 15.53 \quad (\text{for } \alpha = 1 < 3)$$

$$\bar{\lambda}_w = \frac{h_w}{37.4 t_w \varepsilon \sqrt{k_\tau}} = \frac{1{,}500}{37.4 \times 10 \times 0.81 \times \sqrt{15.53}} = 1.26\,.$$

For longitudinally stiffened panels, condition 5.3(5) in EC3-1-1 is verified:

$\bar{\lambda}_w \geq \bar{\lambda}_{wi} = \dfrac{h_{wi}}{37.4 \varepsilon \sqrt{k_{\tau i}}} = \dfrac{120}{37.4 \times 0.81 \sqrt{7.9}} = 1.4$. The condition is not verified.

where: $k_{\tau i} = 5.34 + 4\left(\dfrac{120}{150}\right)^2 = 7.9\,.$

Consequently, it is taken $\overline{\lambda}_w = 1.4 > 1.08 \Rightarrow \chi_w = 1.37/\left(0.7+\overline{\lambda}_w\right) = 0.65$

$$V_{bw.Rd} = \frac{\chi_w f_{yw} h_w t}{\sqrt{3}\gamma_{M1}} = \frac{0.65\times 3{,}550\times 150\times 1}{\sqrt{3}\times 1.1}\times 10^{-2} = 1{,}817 \text{ kN}$$

The buckling resistance of the section will be:

$$V_{b.Rd} = V_{bw.Rd} + V_{bf.Rd} = 1{,}843 \text{ kN}$$

5. Conclusions

The results obtained in the numerical analysis are collected in Table 2.

Table 2
Numerical Analysis Results

		$M_{c.Rd}$, [kNm]	$\frac{M_{c.Rd}}{M_{c.Rd}^{nerig.}}$	$V_{b.Rd}$, [kN]	$\frac{V_{b.Rd}}{V_{b.Rd}^{nerig}}$
Non stiffened girder $M_{c.Rd}^{nerig.}$		7,128	1.00	1,675	1.00
Stiffened girder	$M_{c.Rd}^{el-pl}$	8,275	1.16	1,843	1.10
	$M_{c.Rd}^{r}$	8,120	1.14		

Following the numerical analysis performed, one can see the efficiency degree and the role played by longitudinal stiffeners in the bending and shear buckling behavior of metal girders, presenting high slenderness plates in their composition (Class 4 for cross section and thin plate) according to the simple post critical resistance calculation method.

Often used as a consolidation variant, the disposition of a longitudinal stiffener on the slender plates of the girders can be a method through which the bearing capacity of the girders is increased both by diminishing the inactive zone of the plate of Class 4, and by magnifying the area resistance module.

The longitudinal stiffening also has the effect of increasing the shear buckling resistance of the girder plate (in our case by 10%).

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GRINZI CU INIMI ZVELTE RIGIDIZATE LONGITUDINAL

(Rezumat)

Eficiente din punct de vedere structural, grinzile cu inimi având zveltețea ridicată (Clasa 4) intră frecvent în alcătuirea tablierelor de poduri metalice şi a construcțiilor metalice în general, fiind acceptată ipoteza de lucru în care anumite părți ale secțiunii elementului se află temporar într-o formă de echilibru deformată (voalare locală).

Zonele voalate contribuie la rigiditatea grinzii, dar nu se iau în considerare la preluarea eforturilor unitare normale de compresiune, iar din punct de vedere al voalării din forfecare, în starea limită ultimă după ce se produce voalarea locală a inimii grinzii, există o rezervă de rezistență denumită *postcritică,* care asigură comportarea mecanică a elementului structural în condiții de siguranță corespunzătoare.

Prin dispunerea unor rigidizări longitudinale se poate micşora sau elimina zona inactivă a inimii, scăzând clasa secțiunii şi ca urmare creşte rezistența la încovoiere, dar şi rezistența la voalare din forfecare a grinzii în ansamblu.

În lucrare este prezentată o analiză numerică privind rezistența la încovoiere şi rezistența la voalare din forfecare a unei grinzi prevăzută cu o rigidizare longitudinală, luând în considerare flexibilitatea rigidizării.