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Analytical study of buckling profile web stability

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Abstract. Elements used in steel structures may be considered as an assembly of number of thin flat walls. Local buckling of these members can limit the buckling capacity of axial load resistance or flexural strength. We can avoid a premature failure, caused by effects of local buckling, by limiting the value of the wall slenderness which depend on its critical buckling stress. According to Eurocode 3, the buckling stress is calculated for an internal wall assuming that the latter is a simply supported plate on its contour. This assumption considers, without further requirement, that the two orthogonal walls to this wall are sufficiently rigid to constitute fixed supports to it.
In this paper, we focus on webs of steel profiles that are internal walls delimited by flanges profiles. The objective is to determine, for a given web, flanges dimensions from which the latter can be considered as simple support for this web.

Keywords: local buckling; thin plates; elastic supports

1. Introduction

The steel profiles, rolled or welded, can be considered as an assembly of a number of flat plates, each of them is delimited by a further plate which is orthogonal or by a free edge. A plate which is delimited by two orthogonal plates is called “internal wall”, on the other hand, a plate which is delimited by another orthogonal plate and by a free edge is called “outstanding wall”.

As the plates of the steel profiles are relatively thin compared to their width, when they are subjected to compression (due to the application of axial loads on the entire cross-section and / or due to bending), they can buckle locally. Thus, the predisposition of any cross-section’s plate can limit the buckling capacity of the axial load resistance or flexural strength of the section, preventing it to reach its elastic limit. We can avoid a premature failure caused by the effects of local buckling by limiting the ratio width / thickness of the cross section walls. This is the basis of the cross-section classification approach adopted by the Eurocode 3 to take into account the incidence degree of local buckling, due to the compression, on the resistance of the section. This classification depends on the value of the wall slenderness which is defined as

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$$\bar{\lambda}_p = \sqrt{\frac{f_y}{\sigma_{cr}}} \tag{1}$$

where f_y is the yield strength of the wall material and σ_{cr} is the critical buckling stress of the wall.

According to Eurocode 3, the elastic buckling critical stress is calculated for an internal wall assuming that the latter is a simply supported plate on its contour.

This assumption considers, without further requirement, that the two orthogonal walls to this wall are sufficiently rigid to constitute fixed supports to it. In this study we focus on webs of steel profiles that are internal walls delimited by the flanges profiles. The objective is to determine, for a given web, flanges dimensions from which the latter can be considered as simple supports for this web. For that reason, we will study the buckling stability of a web profile, considering it as a plate supported elastically by the flanges. By comparing the results of this study with those of a simply supported plate on its contour, we can then reach the objective of this work.

The buckling is an important factor in the structure design, several studies have been carried out in order to analyze it in different situations.

He and Ren (2012) proposed an analytical method to determine the vibration and buckling of rectangular thin plates supported by elastic foundation with different boundary conditions, the impact of the proposed method is discussed through various numerical examples.

Benyoucef *et al.* (2010) examined the static response of simply supported stiffened plates subjected to a uniform transversal load or distributed sinusoidal load supported by an elastic foundation, using a model of hyperbolic displacement. Bodaghi and Saidi (2010) proposed an exact analytical solution for the critical buckling load of stiffened rectangular plates with different boundary conditions, and subjected to non-uniform distributed load in the plane acting on two simply supported opposite edges .

Bedair (2009) developed equations for effective widths of the plates under an inhomogeneous loading applied in the plane of the plate. The proposed analytical expressions are used to calculate the effective width of I-slender sections for beams and columns.

Tian and Fu (2008) studied the effects of damage evolution and the initial deflection in the elastic-plastic post-buckling state of orthotropic plates, while Zhulin (1998) estimated the ultimate strength of simply supported composite thin plates under compression.

Jubran and Cofer (1991) analyzed the ultimate strength of the structural components, using an analytical model to include the effect of the material fracture on the overall structure behavior.

Many studies have been performed to analyze the behavior of plates using different expressions of displacement field according to the studied case. Shear deformation plate theories are usually used in case of thick plates and non-homogeneous material plate such a functionally graded plate (FGP). Bouazza *et al.* (2010) investigated the thermo elastic buckling of FGP using first shear plate theory. Effects of changing plate characteristics, material composition and volume fraction of constituent materials on the critical temperature difference for FGP with simply supported edges are also investigated. A new hyperbolic shear deformation theory taking into account transverse shear deformation effects is presented by El Meiche *et al.* (2011) for the buckling and free vibration analysis of thick functionally graded sandwich plates. Bourada and his coworkers (2012) used four-variable refined plate theory to the thermal buckling behavior of functionally graded sandwich plates. It seems that this theory is simple to use because it has strong similarities with the classical plate theory in some aspects such as governing equation, boundary conditions and moment expressions. Tounsi *et al*. (2013) investigated the thermo elastic bending of functionally graded sandwich plates. The refined trigonometric shear deformation plate theory used by the authors accounts for trigonometric distribution of transverse shear stress, and satisfies the free transverse shear stress conditions on the top and bottom surfaces of the plate without using shear correction factor. An analytical solution to the thermo mechanical bending response of functionally graded plates (FGP) resting Winkler-Pasternak elastic foundations has been developed by Bouderba *et al.* (2013). They used theoretical formulations based on refined trigonometric shear deformation theory developed by Tounsi *et al.* (2013). Thermal buckling analysis of FGP has been developed by Bouiadjra *et al.* (2013) using an efficient sinusoidal shear deformation theory based on exact position of neutral surface and taken into consideration the non-linear strain-displacement relations. Using only five unknown functions as against six or more in case of other shear and normal deformation theories, Bousahla *et al.* (2014) developed a new trigonometric higher-order theory for the static analysis of advanced composite plates.

The present paper is an analytical contribution to the study of buckling walls of steel profiles. Since the walls of steel profiles are often thin plates, classical thin plate theory will be used in this study.

2. Analytical study of the webs buckling stability

In this section, we perform an analytical approach to the study of the plate buckling phenomenon. In fact, the search of the plate deformation whose shape, support user and loading mode are given, is reduced to a mathematical problem that consists to solve the fundamental plates' equation of Saint-Venant (Timoshenko 1963) who satisfies the support conditions on the plate contour.

2.1 Simply supported web on its contour

Examine a structural steel compressed element which is made of an elastic material. In the design code (Eurocode 3 1999), the web of profile is considered as a simply supported plate on its contour Fig. 1, it is able to ruin by instability, when applied compressive stress reaches a critical value (σ_{cr}), called critical buckling stress. Its expression is given by several researchers Timoshenko (1963) and Estanave (1900) as the following formula

$$\sigma_{cr} = \frac{k_\sigma \pi^2 E}{12(1-\upsilon^2)}\left(\frac{t_w}{d}\right)^2 \qquad with \qquad \sigma_{cr} = \frac{N_{cr}}{t_w} = k_\sigma \frac{\pi^2 D}{d^2 t_w} \tag{2}$$

Where k_σ the buckling coefficient is given by

$$k_\sigma = \left(\frac{md}{a} + \frac{a}{md}\right)^2 \tag{3}$$

For aspect ratios a/d> 1, the buckling coefficient (k_σ) tends to the value 4.

2.2 Web resting on two elastic supports:

In the following study, the web of the same structural steel element, is considered as a simply supported plate on the two edges perpendicular to the load (transversal direction) and on the flanges which are considered as elastic supports (longitudinal direction).

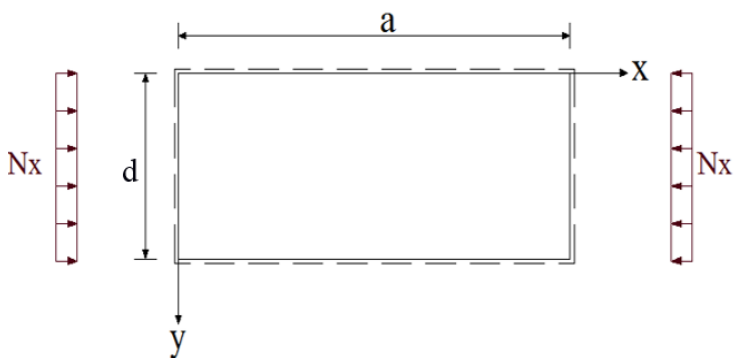

Fig. 1 Web's profile considered as a plate supported on four sides and subjected to a unidirectional compressive loading

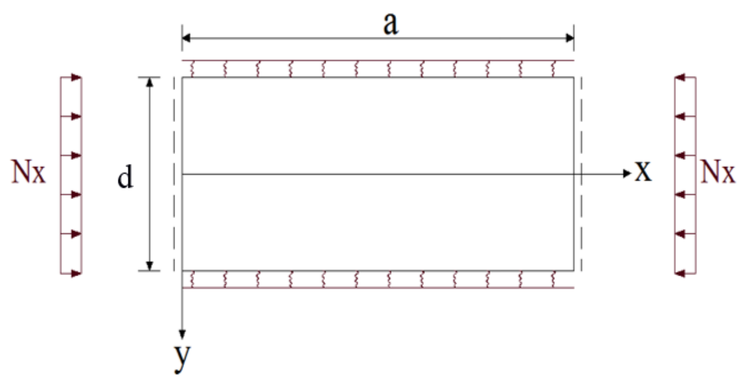

Fig. 2 Web of steel profile considered as a simply supported plate in (*y*-*y*) and elastically supported in (*x*-*x*)

The differential equation of the plate is derived from the Saint-Venant's equation (Timoshenko 1963). The only force acting is a compressive force (-Nx)

$$\frac{\partial^4 W}{\partial x^4} + 2\frac{\partial^4 W}{\partial x^2 \partial y^2} + \frac{\partial^4 W}{\partial y^4} = \frac{1}{D}\left(-N_x \frac{\partial^2 W}{\partial x^2}\right) \tag{4}$$

The solution for rectangular plates simply supported on two opposite sides is given by Maurice Levy (Estanave 1900, Courbon 1980) by

$$W(x,y) = \sum_{m=1}^{\infty} f_m(y) \sin\frac{m\pi x}{a} \tag{5}$$

The boundary conditions of a plate which is simply supported on both sides Fig. 2 are: displacement and bending moments are zero on the supported sides.

For *x*=0:

$$W(0,y) = 0 \qquad \textit{satisfied condition}$$

$$\frac{\partial^2 W(0,y)}{\partial x^2} + v\frac{\partial^2 W(0,y)}{\partial y^2} = 0 \qquad \textit{satisfied condition}$$

For *x*=a:

$$W(a,y) = 0 \qquad \text{satisfied condition}$$

$$\frac{\partial^2 W(a,y)}{\partial x^2} + \upsilon \frac{\partial^2 W(a,y)}{\partial y^2} = 0 \qquad \text{satisfied condition}$$

Let’s calculate the 4th order partial derivatives of the solution (5) and replace in the differential Eq. (4), which allows transforming the partial differential equation to an ordinary differential equation

$$f^{(4)}(y) - 2\left(\frac{m\pi}{a}\right)^2 f^{(2)}(y) + \left[\left(\frac{m\pi}{a}\right)^4 - \frac{N_x}{D}\left(\frac{m\pi}{a}\right)^2\right] f(y) = 0 \tag{6}$$

The mathematical solution of this equation is given by

$$f(y) = Ce^{Ry} \tag{7}$$

Let’s calculate and replace the second and fourth derivatives *f(y)* in the Eq. (6), the following characteristic equation is obtained

$$R^4 - 2\left(\frac{m\pi}{a}\right)^2 R^2 + \left[\left(\frac{m\pi}{a}\right)^4 - \frac{N_x}{D}\left(\frac{m\pi}{a}\right)^2\right] = 0 \tag{8}$$

Its solutions are given as follows

$$R_1^2 = \left(\frac{m\pi}{a}\right)^2 + \left(\frac{m\pi}{a}\right)\sqrt{\frac{N_x}{D}}$$

$$R_2^2 = i^2\left[-\left(\frac{m\pi}{a}\right)^2 + \left(\frac{m\pi}{a}\right)\sqrt{\frac{N_x}{D}}\right] \tag{9}$$

The second solution R_2^2 is defined only if

$$\frac{N_x}{D} > \left(\frac{m\pi}{a}\right)^2 \tag{10}$$

We proposed to determine the dimensions of the flanges from which the sides $y=d/2$ and $y=-d/2$ are considered, as simply supported and so that the buckling coefficient in this case is that of a plate which is simply supported on its contour. Considering expressions (2) and (3), the inequality (10) becomes

$$\frac{\pi^2}{d^2}\left(\frac{md}{a} + \frac{a}{md}\right)^2 > \left(\frac{m\pi}{a}\right)^2 \tag{11}$$

This condition is satisfied, and then we maintain the solution given in (9).
Substituting the expression (2) in the solution (9)

$$R_1 = \pm\sqrt{\left(\frac{m\pi}{a}\right)^2 + \left(\frac{m\pi^2}{ad}\right)\sqrt{k_\sigma}} = \pm\alpha$$

$$R_2 = \pm i\sqrt{-\left(\frac{m\pi}{a}\right)^2 + \left(\frac{m\pi^2}{ad}\right)\sqrt{k_\sigma}} = \pm i\beta \qquad with \qquad k_\sigma \frac{\pi^2}{d^2} > \left(\frac{m\pi}{a}\right)^2 \tag{12}$$

Replace the expressions of R_1 and R_2 in the solution of the ordinary differential Eq. (7). The general solution (5) of the plate differential equation becomes

$$W(x,y) = (C_1 e^{\alpha y} + C_2 e^{-\alpha y} + C_3 \cos\beta y \ + C_4 \sin\beta y)\sin\frac{m\pi x}{a} \tag{13}$$

To determine the constants of this solution, we use the boundary conditions as given by Timoshenko (1963). Rotational freedom conditions require that

$$\frac{\partial^2 W}{\partial y^2} + v\frac{\partial^2 W}{\partial x^2} = 0 \qquad for\ \ y = \pm\frac{d}{2} \tag{14}$$

By calling EI_f the flange flexural rigidity, the differential equation of the elastic line is given by

$$\begin{aligned} EI_f\frac{\partial^4 W}{\partial x^4} &= D\left[\frac{\partial^3 W}{\partial y^3} + (2-v)\frac{\partial^3 W}{\partial x^2 \partial y}\right] - A_f\sigma_x\frac{\partial^2 W}{\partial x^2} \qquad for\ \ y = \frac{d}{2} \\ EI_f\frac{\partial^4 W}{\partial x^4} &= -D\left[\frac{\partial^3 W}{\partial y^3} + (2-v)\frac{\partial^3 W}{\partial x^2 \partial y}\right] - A_f\sigma_x\frac{\partial^2 W}{\partial x^2} \quad for\ \ y = \frac{-d}{2} \end{aligned} \tag{15}$$

By substituting the solution (13) into these boundary conditions, we obtain a system of four equations with four unknowns. Equaling its determinant to zero, we obtain a complex expression which allows calculating the values of the critical buckling stresses.

3. Analytical developments exploitation:

Developments carried out above do not allow to simply determine the critical stresses values, for this reason, we explore the expression obtained to determine the characteristics of elastic beams, which make the plate having the same critical stress for a simply supported plate on its four sides, i.e., for (*a/d*> 1) then (k_σ=4).
For simplification reasons, the following change of variables is posed

$$\gamma = \frac{a}{d} \qquad \delta = \frac{t_f}{t_w} \qquad \lambda_f = \frac{b_f}{t_f} = \frac{b_f}{\delta t_w} \qquad \lambda_w = \frac{d}{t_w} \tag{16}$$

The geometrical characteristics of elastic supports (the flanges), and stress applied to the elastic supports become

$$\begin{aligned} I_f &= \frac{t_f b_f^3}{12} = \frac{\lambda_f^3\delta^4 t_w^4}{12} \qquad A_f = b_f t_f = \lambda_f\delta^2 t_w^2 \\ \sigma_x &= \frac{k_\sigma\pi^2 E}{12(1-\upsilon^2)}\left(\frac{t_w}{d}\right)^2 = \frac{E}{12(1-\upsilon^2)}\frac{k_\sigma\pi^2}{\lambda_w^2} \end{aligned} \tag{17}$$

This problem is introduced as a matrix form, and taking into account (15) and (16), then we set

$$\frac{\alpha\, d}{2} = \mathrm{G1} = \frac{\pi}{2}\sqrt{\frac{m^2}{\gamma^2} + \frac{m\sqrt{k_\sigma}}{\gamma}}$$

$$\frac{\beta\, d}{2} = G2 = \frac{\pi}{2}\sqrt{-\frac{m^2}{\gamma^2} + \frac{m\sqrt{k_\sigma}}{\gamma}}$$

$$S1 = \alpha^2 - \nu\,\frac{m^2\pi^2}{a^2} = \frac{\pi^2}{\lambda_w^2 t_w^2}\left[(1-\nu)\frac{m^2}{\gamma^2} + \frac{m\sqrt{k_\sigma}}{\gamma}\right]$$

$$S2 = \beta^2 + \nu\,\frac{m^2\pi^2}{a^2} = \frac{\pi^2}{\lambda_w^2 t_w^2}\left[(-1+\nu)\frac{m^2}{\gamma^2} + \frac{m\sqrt{k_\sigma}}{\gamma}\right]$$

$$S3 = EI_f\frac{m^4\pi^4}{a^4} - A_f\sigma_x\frac{m^2\pi^2}{a^2} = \frac{E\pi^4}{12}\frac{\lambda_f}{\lambda_w^4}\left[\frac{m^4\delta^4\lambda_f^2}{\gamma^4} - \frac{k_\sigma m^2}{1-\nu^2}\times\frac{\delta^2}{\gamma^2}\right]$$

$$S4 = D\alpha^3 - D(2-\nu)\frac{m^2\pi^2}{a^2}\alpha = \frac{E\pi^3}{12\,(1-\nu^2)\lambda_w^3}\left[\sqrt{\left(\frac{m^2}{\gamma^2} + \frac{m\sqrt{k_\sigma}}{\gamma}\right)^3} - (2-\nu)\frac{m^2}{\gamma^2}\sqrt{\frac{m^2}{\gamma^2} + \frac{m\sqrt{k_\sigma}}{\gamma}}\right]$$

$$S5 = D\beta^3 + D(2-\nu)\frac{m^2\pi^2}{a^2}\beta = \frac{E\pi^3}{12\,(1-\nu^2)\lambda_w^3}\left[\sqrt{\left(-\frac{m^2}{\gamma^2} + \frac{m\sqrt{k_\sigma}}{\gamma}\right)^3} + (2-\nu)\frac{m^2}{\gamma^2}\sqrt{-\frac{m^2}{\gamma^2} + \frac{m\sqrt{k_\sigma}}{\gamma}}\right]$$

(18)

In a matrix form, the equation's system is given as follows

$$\begin{pmatrix} S1 * e^{G1} & S1 * e^{-G1} & -S2 * \mathrm{Cos}[G2] & -S2 * \mathrm{Sin}[G2] \\ S1 * e^{-G1} & S1 * e^{G1} & -S2 * \mathrm{Cos}[G2] & S2 * \mathrm{Sin}[G2] \\ e^{G1} * (S3 - S4) & e^{-G1} * (S3 + S4) & S3 * \mathrm{Cos}[G2] - S5 * \mathrm{Sin}[G2] & S5 * \mathrm{Cos}[G2] + S3 * \mathrm{Sin}[G2] \\ e^{-G1} * (S3 + S4) & e^{G1} * (S3 - S4) & S3 * \mathrm{Cos}[G2] - S5 * \mathrm{Sin}[G2] & -S5 * \mathrm{Cos}[G2] - S3 * \mathrm{Sin}[G2] \end{pmatrix} \times \begin{pmatrix} C_1 \\ C_2 \\ C_3 \\ C_4 \end{pmatrix} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \quad (19)$$

By equaling to zero the determinant of this system, we obtain an equation as a function of (δ), (ν), (k_σ), (γ), (m), (λ_w) and (λ_f).

In the case of a plate supported on four sides, and aspect ratios *a/d* greater than 1 (common case in steel construction) the buckling coefficient (k_σ) is equal to 4, and is given by expression (3). By using the notation (16), we obtain

$$\left(\frac{m}{\gamma} + \frac{\gamma}{m}\right)^2 = 4 \;\rightarrow m = \gamma \quad (20)$$

For all values of (m) and (γ) which satisfy the Eq. (20) the steel profile internal wall can be considered as a plate supported on four sides (k_σ=4).

By replacing parameters: ν=0.3 (steel), k_σ=4 and $\gamma=m$ in the determinant of the equations system (19), we obtain an equation as a function of (λ_w), (λ_f) and (δ).

3.1 Parametric study

In order to develop numerical results, we proceed to vary the web slenderness and calculate the corresponding flanges slenderness. The expressions exploitation of $\lambda_f=f(\lambda_w)$ for each values of (δ), allowed us to plot the following curves.

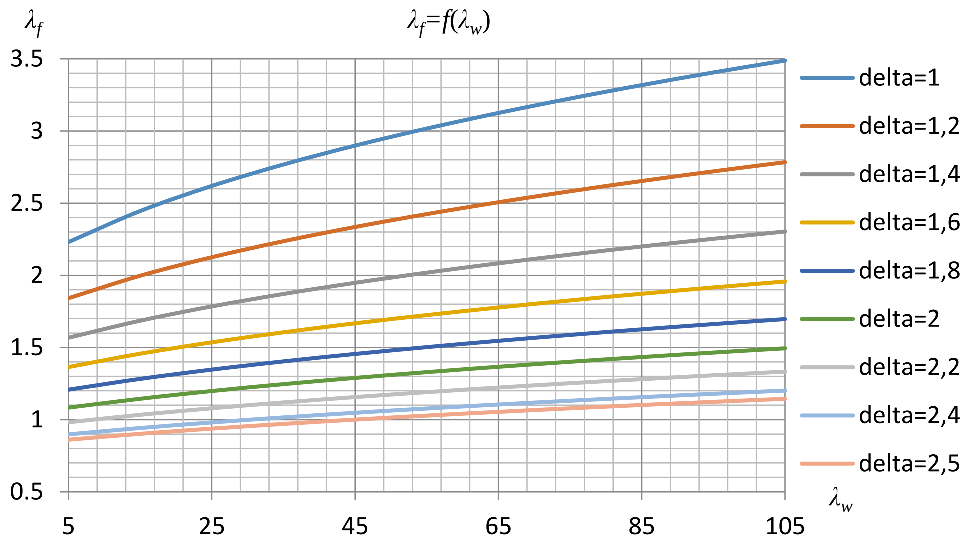

Fig. 4 Variation of flanges slenderness depending on the web slenderness $\lambda_f=f(\lambda_w)$

Remark:

From the graph of Fig. 4, we can calculate the minimum flanges slenderness to form efficient supports to the web. In this case, the assumption that the web is considered as a plate supported on four sides becomes acceptable.

4. Curves exploitation

To check that the flanges are efficient supports for the web of a steel profile, the following method is proposed:

We begin by calculating the ratio of flanges thickness on web thickness ($\delta=t_f/t_w$) and web width on its thickness ($\lambda_w=d/t_w$). Thereafter, to obtain the flanges slenderness value (λ_f), we project the (λ_w) value in the appropriate curve of the Fig. 4.

4.1 Practical example

Consider the case of the profile IPE220. For this profile and according to the standard EU 19-57 (Euronormes), its geometric characteristics are: d=178 mm; t_w=5.9 mm; b_f=110 mm; t_f=9,2 mm. We can then calculate: $\delta=t_f/t_w=1.56$; $\lambda_w=d/t_w=30.17$; $\lambda_f=b_f/t_f=11.96$. The minimum flanges slenderness is (λ_f=1.63).

Result:

In order to consider the web of an IPE220 profile as a simply supported plate on its contour, it is sufficient that the flanges slenderness be equal to (1.63), whereas in reality it is (11.96). Therefore this condition is largely satisfied.

4.2 Application on rolled profiles

The same work was carried out for the profiles IPE, HEA and HEB types. It is presented as a table showing minimum flanges widths and real flanges widths of rolled profiles.

From the results of this table we find that the assumption usually made which considers the web of a rolled profile (I or H) as a plate supported on four sides is widely checked.

4.3 Application on welded profiles

A simple and practical formulation is proposed for welded profiles. The aim of this approach is to provide the minimum flanges dimensions needed in order that the use of the classifications internal walls tables (Eurocode 3 1999), remains valid for the webs of these profiles. Smoothing curves to approximate the graphs of Fig. 4 in the form of a second degree equation gives

$$\lambda_f = A\lambda_w^2 + B\lambda_w + C \tag{21}$$

Table 1 Minimum flanges widths and real flanges widths for profiles IPE, HEA and HEB types

Profile	b_f (mm)	b_f min (mm)	Profile	b_f (mm)	b_f min (mm)	Profile	b_f (mm)	b_f min (mm)
IPE80	46	9	HEA100	100	11	HEB100	100	14
IPE100	55	10	HEA120	120	12	HEB120	120	15
IPE120	64	11	HEA140	140	13	HEB140	140	16
IPE140	73	12	HEA160	160	14	HEB160	160	18
IPE160	82	12	HEA180	180	14	HEB180	180	20
IPE180	91	13	HEA200	200	16	HEB200	200	21
IPE200	100	14	HEA220	220	17	HEB220	220	22
IPE220	110	15	HEA240	240	18	HEB240	240	23
IPE240	120	16	HEA260	260	18	HEB260	260	24
IPE270	135	17	HEA280	280	20	HEB280	280	25
IPE300	150	19	HEA300	300	21	HEB300	300	26
IPE330	160	20	HEA320	300	22	HEB320	300	27
IPE360	170	21	HEA340	300	23	HEB340	300	28
IPE400	180	23	HEA360	300	25	HEB360	300	30
IPE450	190	25	HEA400	300	27	HEB400	300	32
IPE500	200	27	HEA450	300	29	HEB450	300	34
IPE550	210	30	HEA500	300	30	HEB500	300	35
IPE600	220	32	HEA550	300	31	HEB550	300	37
/	/	/	HEA600	300	33	HEB600	300	38
/	/	/	HEA650	300	34	HEB650	300	40
/	/	/	HEA700	300	37	HEB700	300	43
/	/	/	HEA800	300	39	HEB800	300	45
/	/	/	HEA900	300	42	HEB900	300	48
/	/	/	HEA1000	300	44	HEB1000	300	50

Table 2 The coefficients values of the equation $\lambda_f = f(\lambda_w)$

	A×10^3	B	C
δ=1,0	-0,050	0,019	2,162
δ=1,2	-0,040	0,014	1,789
δ=1,4	-0,030	0,010	1,526
δ=1,6	-0,030	0,008	1,330
δ=1,8	-0,020	0,007	1,179
δ=2,0	-0,020	0,005	1,059
δ=2,2	-0,010	0,004	0,961
δ=2,2	-0,010	0,004	0,880
δ=2,5	-0,010	0,003	0,844

The values of the coefficients A, B and C for each value of (δ) are given in Table 2.

To check that the flanges are efficient supports for the web of a steel profile, we project directly the value of (λ_w) in the appropriate curve of the Fig. 4, or we replace (λ_w) in Eq. (21).

5. Conclusions

Through this work, we have attracted the attention of steel structures designers that a problem exists on the concept of "internal wall" as it is defined in the normative requirements for buckling instability (Eurocode 3 1999). The web of a profile does not behave as a supported plate on its four sides only if the flanges dimensions exceed a given values.

The exploitation of results found allowed us to check that for current rolled profiles, the flanges dimensions (and their rigidities) are largely sufficient to be able to assume that the webs of these profiles are simply supported on the flanges. This result serves to reassure the steel structures designers on the stability of current rolled profiles webs.

For welded profiles, the results of this study are used to calculate the minimum dimensions required for flanges.

We can use the results of this work, for profiles under compression, for the other cases of solicitations as pure bending and unsymmetrical bending.

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Nomenclature

f_y : Yield strength of materiel.
σ_{cr} : Critical buckling stress.
$\bar{\lambda}_p$: Reduced wall slenderness.
k_σ : Buckling coefficient.
E : Materiel elastic modulus.
v : Poisson ratio.
t_w : Thickness of the web.
d : Straight part of web steel profile.
N_{cr} : Critical value of compressive force.
N_x : Compressive forces per unit length in the x direction.
D : Plate bending rigidity per unit width.
m : Number of half waves in the longitudinal direction.
a : Length of the steel profile.

$W(x, y)$: Out of plane deflection.
$f(y)$: Deflexion in the x direction.
C_1, C_2, C_3, C_4 : The general solution constants of the plate differential equation.
R_1, R_2, α, β : Solutions of the characteristic equation.
I_f : The flange flexional inertia.
A_f : Area of the flange.
γ : Plate aspect ratio.
δ : The web thickness on flange thickness ratio.
λ_f : The flange slenderness.
λ_w : The web slenderness.
σ_x : Membrane stresses acting in the x direction.
S_1, S_2, S_3, S_4, S_5: Matrix components.
b_f : Width of the flanges steel profile.
t_f : Thickness of the flange.