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Numerical study of local buckling stability for HEA rolled profiles

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Abstract

When the structural elements are subjected to compressive load, their webs and flanges are totally or partially in-plane compressed and they can locally buckle. Local buckling of webs and flanges can reduce their compressive and flexural strength. To avoid local buckling failure, we can limit the plate slenderness value which depends on its critical buckling stress.
A numerical buckling simulation of H profile is developed using finite elements method. This numerical model permits to calculate web buckling stress and flange buckling stress, according to the boundary conditions of these plates. The numerical results will be used to propose a classification of rolled profiles. The proposed classification will be compared to that made according to the steel design codes.

1.Introduction

The rolled or welded profiles are formed by an assembly of thin plates; each of them is delimited by another one which is orthogonal or by a free edge. A plate which is delimited by two orthogonal plates is called "internal wall", and a plate which is delimited by another orthogonal plate and by a free edge is called "outstanding wall". For an H profile, web is an "internal wall", and flanges are "outstanding walls".

As the plates of the steel profiles are relatively thin compared to their width, when they are subjected to in-plane compression, they can locally buckle. Local buckling of webs and flanges can reduce compressive strength and flexural strength.

Buckling of plates is a subject that continues to gain importance in steel construction research. Taleb et all. interest in the local buckling of steel plates [1, 2, 3, 4]. They have developed an analytical (2015) [1] and numerical study [2] (2013) about the stability of the webs steel profiles under compression. Those studies have shown that the dimensions of the steel rolled profiles flanges are sufficient to consider the webs as supported plates in four sides. They have developed a numerical program (2020) [3], which permits to estimate the buckling stress of webs steel profiles under shear force.

Komur and Sonmez (2015) [5] have studied the buckling behavior of perforated rectangular plates under different edge loads by studying the effects of the plate length, the location of the edge loading and the diameter of the hole on its critical buckling load.

Kitarovic et al. (2015) [6] have developed a numerical program in order to calculate the critical buckling shear stress, and propose a formula for calculating this stress for stiffened thin plates.

Premature failure caused by the effects of local buckling may be avoid by limiting the ratio width/thickness of the cross section walls. This is the basis of the steel profiles classification approach, adopted by the steel design codes (EC3 2005 [7], CCM97 1997 [8]), 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 non-dimensional slenderness which is defined by the formula of the equation (1):

$$\overline{\lambda_p} = \sqrt{\frac{f_y}{\sigma_{cr}}} \quad (1)$$

where f_y is the yield strength of the wall material and σ_{cr} is the critical buckling stress of the wall. The critical buckling stress for a thin plate with in plane dimensions axb and thickness t, subjected to totally or partially in-plane compression along its width is given in the literature (TIMOSHENKO et all. 1961[9] and 1959 [10]) by the formula of the equation (2):

$$\sigma_{cr} = \frac{k_\sigma\, \pi^2 E}{12(1-\upsilon^2)}\left(\frac{t}{b}\right)^2 \quad (2)$$

According to steel design codes (EC3 2005 [7], CCM97 1997 [8]), the elastic buckling critical stress is calculated for a compressed internal wall assuming that the latter is a simply supported plate on its contour. The buckling coefficient k_σ is considered identical to that of compressed simply supported plate which is equal to 4. For a compressed outstanding wall, the buckling coefficient k_σ adopted in the steel design codes (EC3 2005 [7], CCM97 1997 [8]) is identical to that of compressed plate simply supported on three sides which is equal to 0,43.

In this study, a numerical model for buckling behavior of steel profile is developed using finite elements analysis. The developed numerical program permits to estimate the values of profile walls critical buckling stresses according to the boundary conditions that effectively occur at the sides of this profile walls.

As a first step, the buckling of a simply supported plate on its sides is modeled, under compression, in order to validate the numerical results with those of the literature (TIMOSHENKO et all. 1961[9]). Based on the same program, the rolled profile walls buckling are simulated in order to calculate their critical stresses. The corresponding slenderness are compared with those adopted by the steel design codes (EC3 2005 [7], CCM97 1997 [8]), which allow reviewing the classification of the cross-sections for these profiles.

2. Buckling simulation of a simply supported plate on its contour

The first model consists in modeling one case whose the theoretical results are known: a plate which is simply supported on its four sides. Assumed that the plate is rectangular, formed by an isotropic material, homogeneous and elastic. It is subjected to a linear compression force acting along its width. The aim of this model is the calculation of the critical buckling stress.

The numerical program is executed by the software CASTEM.

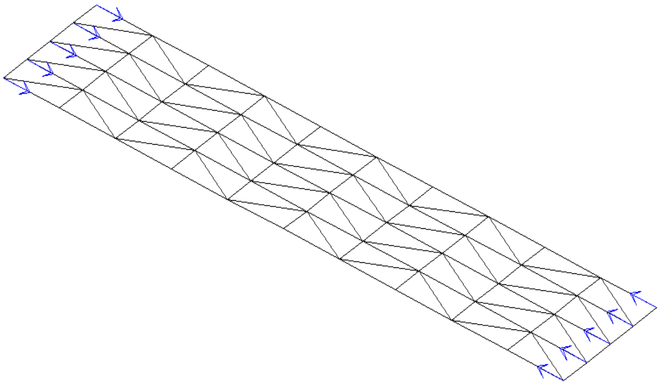

Figure 1. Plate modeled by CASTEM

The example of a plate with 4m of length and 0.01m of thickness is considered. The width "b" is varied. The numerical values of the critical buckling stress are compared with the theoretical ones, calculated using the formula of the Equation (2).

Table 1 Differences between the critical buckling stresses estimated by CASTEM and by the theoretical formula.

b (m)	σ_{cr}(MPa) numerical	σ_{cr}(MPa) theoretical	Differences %
0,25	1185	1215	2,5
0,35	606	620	2,2
0,4	465	475	2,1
0,45	367	375	2,1
0,5	300	304	2,1
0,55	246	251	2
0,65	176	179	2

The values of Table 1 show that the numerical results are very approached to the theoretical ones.

3. Numerical model of a steel profile

To model "H" steel profile, the plate of the preceding paragraph will be used. In the longitudinal direction, the supports will be replaced by two plates with thickness t_f and width b_f.

These plates, which represent the flanges of the steel profile, are perpendicular to the initial plate, which represent the web. A steel profile is modeled as shown in the Figure 2.

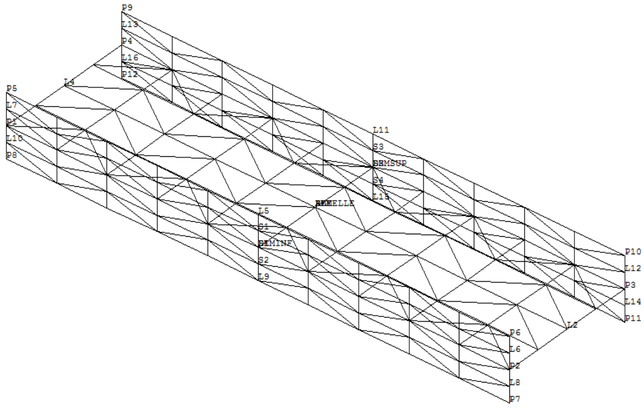

Figure 2. H steel profiles modeled by CASTEM.

The application of the compressive load in one wall permits to estimate its critical buckling stress.

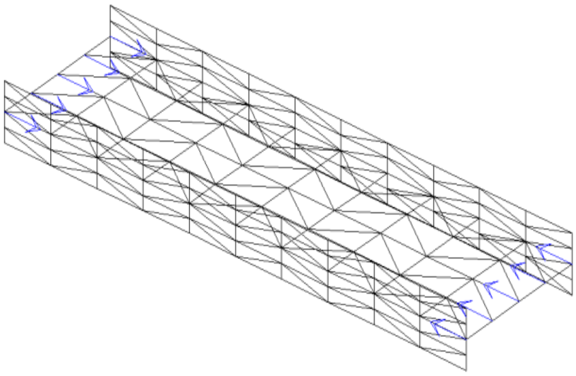

Figure 3. Steel profiles under compression

4. HEA rolled profiles under compression

According to the classification of the cross-sections given by the EC3 (2005) [7] and CCM97 (1997) [8], the class of the HEA rolled profiles, under compression, depends on the class of their webs (Euronorm [11]). Classifying webs with the limits specified in these codes designs (EC3 (2005) [7], CCM97 (1997)[8]) means considering the buckling coefficient equal to that of simply supported plate on four sides k_σ=4, whereas this coefficient depends on the dimensions of the flanges (Taleb et all. (2015) [1],(2016) [4]).

4.1. Estimation of the buckling coefficient and calculation of the non-dimensional slenderness

Using the numerical model, the value of the critical buckling stress σ_{cr} is estimated. Thereafter, the buckling coefficient k_σ can be deduced from the formula of the Equation(2) by :

$$k_\sigma = \frac{12(1-\upsilon^2)}{\pi^2 E} \times \sigma_{cr} \times \left(\frac{b}{t}\right)^2 \quad (3)$$

The values of the webs' critical buckling stresses and the corresponding buckling coefficients of the HEA webs rolled profiles are given in the Table 2.

Table 2 The critical buckling stresses and the corresponding buckling coefficients according to the numerical model for the HEA webs.

Profil	σ_{cr}(MPa)	k_σ
HEA100	6575,24	4,35
HEA120	4390,22	5,07
HEA140	3603,72	5,31
HEA160	3347,81	5,3
HEA180	2435,87	5,31

HEA200	2347,65	5,26
HEA220	2110,48	5,24
HEA240	2080,97	5,24
HEA260	1795,41	5,27
HEA280	1657,39	5,24
HEA300	1662,86	5,25
HEA320	1598,95	5,27
HEA340	1525,88	5,26
HEA360	1464,07	5,25
HEA400	1351,49	5,23
HEA450	1112,91	5,25
HEA500	945,43	5,26
HEA550	810,85	5,25
HEA600	710,16	5,23
HEA650	632,43	5,21
HEA700	609,54	5,17
HEA800	483,95	5,15
HEA900	419,78	5,12
HEA1000	349,67	5,1

Knowing the numerical value of the buckling coefficient, the non-dimensional slenderness can be calculated through the formula given in the EC3 (2005) [7], and the CCM97 (1997)[8] as in the Equation (4).

$$\overline{\lambda_p} = \frac{b/t}{28{,}4\,\varepsilon\sqrt{k_\sigma}} \qquad (4)$$

The non-dimensional slenderness values considered in the design codes (EC3 2005, CCM97 1997) can be obtained by replacing the buckling coefficient by k_σ= 4 in the formula of the Equation(4).

The design codes values of the non-dimensional slenderness and those numerically estimated for the steel grades S235, S275 and S355 are given in the Table 3.

Table 3 The non-dimensional slenderness according to the EC3(2005)[7] and according to the numerical approach for the HEA webs.

		according to EC3(2005) and CCM97(1997)			according to the numerical approach		
Profil	λ=d/tw	S235	S275	S355	S235	S275	S355
HEA100	11,2	0,2	0,21	0,24	0,19	0,21	0,23
HEA120	14,8	0,26	0,28	0,32	0,23	0,25	0,29
HEA140	16,73	0,29	0,32	0,36	0,26	0,28	0,32
HEA160	17,33	0,31	0,33	0,38	0,27	0,29	0,33
HEA180	20,33	0,36	0,39	0,44	0,31	0,34	0,38
HEA200	20,62	0,36	0,39	0,45	0,32	0,34	0,39
HEA220	21,71	0,38	0,42	0,47	0,33	0,36	0,41
HEA240	21,87	0,38	0,42	0,48	0,34	0,37	0,42
HEA260	23,6	0,42	0,45	0,51	0,36	0,39	0,45
HEA280	24,5	0,43	0,47	0,53	0,38	0,41	0,47
HEA300	24,47	0,43	0,47	0,53	0,38	0,41	0,46
HEA320	25	0,44	0,48	0,54	0,38	0,42	0,47
HEA340	25,58	0,45	0,49	0,56	0,39	0,43	0,48
HEA360	26,1	0,46	0,5	0,57	0,4	0,44	0,49
HEA400	27,09	0,48	0,52	0,59	0,42	0,45	0,52
HEA450	29,91	0,53	0,57	0,65	0,46	0,5	0,57
HEA500	32,5	0,57	0,62	0,71	0,5	0,54	0,62
HEA550	35,04	0,62	0,67	0,76	0,54	0,59	0,67
HEA600	37,38	0,66	0,72	0,81	0,58	0,63	0,71
HEA650	39,56	0,7	0,76	0,86	0,61	0,66	0,75
HEA700	40,14	0,71	0,77	0,87	0,62	0,68	0,77
HEA800	44,93	0,79	0,86	0,98	0,7	0,76	0,86
HEA900	48,13	0,85	0,92	1,05	0,75	0,81	0,92
HEA1000	52,61	0,93	1,01	1,14	0,82	0,89	1,01

Based on the classification of the EC3 (2005) [7], and the CCM97 (1997)[8] about the local buckling, the internal walls slenderness, under compression, are limited for each class of resistance from 1 to 4. By considering theses limits, the limits of the non-dimensional slenderness are obtained. Under compression, an internal wall is in the class 1 if $\overline{\lambda_p} < 0{,}58$, in the class 2 if $\overline{\lambda_p} < 0{,}67$ and in the class 3 if $\overline{\lambda_p} < 0{,}74$. For a non-dimensional slenderness greater than that of class 3, the internal wall is considered in the class 4.

4.2. Reclassification of HEA rolled profiles under compression

Knowing the webs non-dimensional slenderness of HEA rolled profiles (given in the Table 3), and based on the limits given above, a classification of these profiles under compression can be made. The class of resistance is given for the the EC3 (2005) [7]and according to the numerical approach in the Table 4.

Table 4 Classification of the HEA rolled profiles under compression according to the EC3 (2005) [7] and to the numerical approach.

	Classification according to EC3 (2005) and CCM97 (1997)			Classification according to the numerical approach		
Profil	S235	S275	S355	S235	S275	S355
HEA100	1	1	1	1	1	1
HEA120	1	1	1	1	1	1
HEA140	1	1	1	1	1	1
HEA160	1	1	1	1	1	1
HEA180	1	1	1	1	1	1
HEA200	1	1	1	1	1	1
HEA220	1	1	1	1	1	1
HEA240	1	1	1	1	1	1
HEA260	1	1	1	1	1	1
HEA280	1	1	1	1	1	1
HEA300	1	1	1	1	1	1
HEA320	1	1	1	1	1	1
HEA340	1	1	1	1	1	1
HEA360	1	1	1	1	1	1
HEA400	1	1	2	1	1	1
HEA450	1	1	2	1	1	1
HEA500	1	2	3	1	1	2
HEA550	2	3	4	1	2	2
HEA600	2	3	4	1	2	3
HEA650	3	4	4	2	2	4
HEA700	3	4	4	2	3	4
HEA800	4	4	4	3	4	4
HEA900	4	4	4	4	4	4
HEA1000	4	4	4	4	4	4

- Notes:

The profiles HEA400 and HEA450 have changed the class of resistance from the class 2 to the class 1 for the grade of steel S355.

The profile HEA500 has changed the class of resistance from the class 2 to the class 1 for the grade of the steel S275, and from the class 3 to the class 2 for the grade of steel S355.

The profiles HEA550 and HEA600 have changed the class of resistance from the class 2 to the class 1 for the grade of steel S235, and from the class 3 to the class 2 for the grade of steel S275. For the steel grade S355, the profile HEA550 has changed the class from 4 to the class 2, and the profile HEA600 has changed the class from 4 to the class 3.

The profiles HEA650 and HEA700 have changed the class of resistance from the class 3 to the class 2 for the grade of steel S235. For the steel grade S275, the profile HEA650 has changed the class from the class 4 to the class 2, and the profile HEA700 has changed the class from the class 4 to the class 3.

The profile HEA800 has changed the class of resistance from the class 4 to the class 3 for the grade of steel S235.

6. Conclusion

A large number of tests allowed us to develop numerical programs which have a good approximation with our research, and the results are very approached to the theoretical ones.

This study permits us to remark one problem in the consideration of a specific value of the buckling coefficient for the steel profiles according to the EC3 (2005) [7], and the CCM97 (1997)[8]. The value of the buckling coefficient of the steel profiles walls depend on the dimensions of the walls which support them.

The numerical simulation of HEA profiles web's behavior under a uniform compression allowed us to notice that the numerical buckling coefficients are often higher than that of a plate which is supported on its four sides (k_σ= 4).

The use of the numerical buckling coefficient value in the calculation of the non-dimensional slenderness of the rolled steel profiles webs under compression allowed reviewing their current classification. Some profiles which were considered in the class 4 according to the EC3 (2005) [7] and the CCM97 (1997)[8], have become in a better class, which allows to base their resistance on the area of the cross section instead the area of the effective cross section.

Declaration of Conflict of Interests

The authors declare that there is no conflict of interest. They have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

References

[1.] Taleb, C., Ammari, F., Adman, R., Analytical study of buckling profile web stability. Structural Engineering and Mechanics (2015).

[2.] Taleb, C., Ammari, F., Etude numérique de la stabilité au voilement local des âmes des profilés laminés sous compression centrée, 21ème Congrès Français de Mécanique (2013).

[3.] Taleb, C., Ammari, F., ADMAN, R., DJOUAHER, F., Modélisation numérique du voilement par cisaillement des âmes des profilés métalliques par le logiciel Cast3M, Club Cast3M 2020(2020).

[4.] Taleb, C., Ammari, F., Etude de la stabilité des parois des profiles métalliques vis-à-vis du phénomène de voilement, Doctorat en science, FGC, USTHB(2016).

[5.] Komur, M.A., Sonmez, M., Elastic buckling behavior of rectangular plates with holes subjected to partial edge loading, Journal of Constructional Steel Research(2015).

[6.] Kitarovic, S., Andric, J., Piric, K., Rational magnification of the plate elastic shear buckling strength, Thin-Walled Structures (2015).

[7.] Eurocode 3: Design of steel structures, BS EN 1993-1-1(2005).

[8.] CCM97: Règles de conception et de calcul des structures en acier, Centre national de recherche appliquée en génie parasismique (1997).

[9.] Timoshenko, S.P., Gere, J.M., Theory of Elastic Stability 2E, MCGRAW-HILL (1961).

[10.] Timoshenko, S.P., Woinowsky-Krieger, S., Theory of plates and shells, Second Edition, McGRAW-HILL BOOK COMPANY (1959).

[11.] Euronorm 53-62: European wide flange beams HE A, HE B and HE M 100-1000.