International Journal for Computational Civil and Structural Engineering, 20(2) 141–154(2024)

DOI:10.22337/2587-9618-2024-20-2-141-154

ANALYTICAL MODEL FOR BUCKLING LENGTH COEFFICIENT DETERMINATION IN STEEL TRUSSES: A LATERAL TORSIONAL BUCKLING APPROACH

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Abstract: This paper presents a contribution to the study of the stability of steel trusses. The paper proposes an analytical model for the determination of the buckling length coefficient of the truss N by the lateral torsional buckling approach. The model is derived by considering the equilibrium of the truss under the action of the applied loads and the buckling load. The buckling load is determined by considering the lateral buckling of the equivalent beam and the interaction between the members. The model is validated by comparing the predicted buckling loads with the experimental results of several steel trusses. The results show that the model can predict the buckling loads with good accuracy. The proposed model is a valuable tool for the design of steel trusses. The model can be used to determine the buckling length coefficient of the truss and to ensure that the truss is safe and reliable.

Keywords: Buckling, Truss, LTB, Equivalent Web, Ritz, Steel, Stability

АНАЛИТИЧЕСКАЯ МОДЕЛЬ ДЛЯ ОПРЕДЕЛЕНИЯ КОЭФФИЦИЕНТА ДЛИНЫ СМЯТИЯ В СТАЛЬНЫХ ФЕРМАХ: ПОДХОД К ОПРЕДЕЛЕНИЮ БОКОВОГО КРУТИЛЬНОГО СМЯТИЯ

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Аннотация: Данная работа представляет собой вклад в исследование устойчивости стальных ферм. Предложена аналитическая модель для определения коэффициента длины смятия N-фермы по методу бокового кручения. Модель построена с учетом равновесия фермы в условиях приложенных нагрузок и нагрузки смятия. Нагрузка смятия определяется с учетом бокового смятия эквивалентной балки и взаимодействия между элементами. Модель проверена путем сравнения прогнозируемых нагрузок на смятие с экспериментальными результатами для нескольких стальных ферм. Результаты показывают, что модель может предсказывать нагрузки на смятие с хорошей точностью. Предложенная модель является ценным инструментом для проектирования стальных ферм. Модель может быть использована для определения коэффициента длины смятия фермы и обеспечения ее безопасности и надежности**.**

Ключевые слова: Смятие, ферма, LTB, эквивалентное полотно, Ритц, сталь, устойчивость

INTRODUCTION

In steel frame buildings, trusses can provide better performance in terms of strength and stiffness than I-section beams. Trusses are made up of a series of interconnected members that form triangles. The triangles distribute the load evenly across the structure, making it more resistant to buckling and other forms of failure.

I-section beams, on the other hand, are made up of a solid web and two flanges. The web is responsible for carrying the shear force, while the flanges carry the bending moment. I-section beams are strong and stiff, but they are also more susceptible to buckling than trusses.

The equilibrium of a truss subjected to transverse loads can be obtained by developing exclusively axial forces in the bars that

constitute it. This means that the bars are only subjected to tension or compression, and not to bending. This makes trusses more efficient than I-section beams, as they can carry more load with less material.

However, trusses are susceptible to two kinds of instability: local buckling and overall transverse buckling. Local buckling occurs when the individual members of a truss buckle under compression. Overall transverse buckling occurs when the entire truss buckles under the combined action of compression and bending.

To prevent local buckling, the members of a truss must be designed to have a sufficient slenderness ratio. The slenderness ratio is a measure of the slenderness of a member, and it is calculated by dividing the member's length by its radius of gyration. To prevent overall transverse buckling, the truss must be designed to have a sufficient bracing system. The bracing system is a network of members that are used to prevent the truss from buckling under the combined action of compression and bending.

Despite the potential for instability, trusses are a very efficient and versatile structural system. They are often used in steel frame buildings because they can provide better performance in terms of strength, stiffness, and weight than I-section beams.

The local buckling of compressed bars occurs at a buckling length that is generally equal to the length of the bar. The buckling length can be less if the nodes are rigid. The buckling length depends on the type of connection and whether the bar belongs to a chord or a diagonal. The buckling length can be up to a factor of 1.0 times the length of the bar[1]or even less[2, 3];

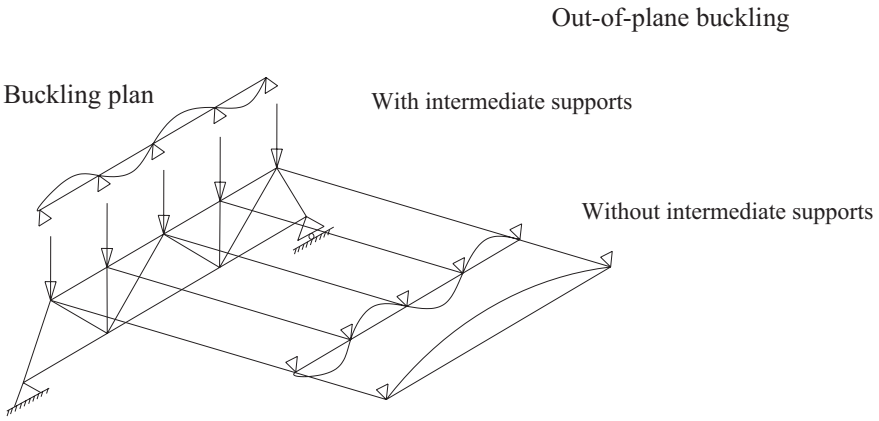


Figure 1. Buckling of a compressed chord

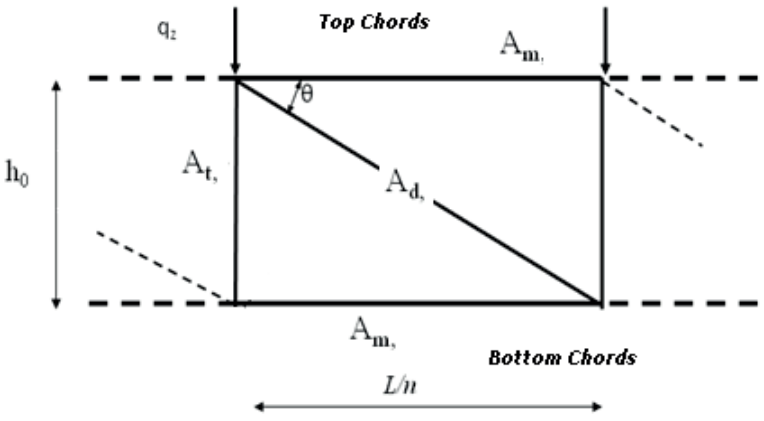


Figure 2. A panel of N-truss models for analyses

The overall transverse buckling, or Lateral torsional buckling (abbreviated as LTB), is induced by the transverse instability of a compressed chord involving several bars and carrying with it the rest of the truss. This can occur when the chord isn't braced laterally. Increasing chord component, therefore, has a transverse length of buckling greater than or less than its individual length.

This paper presents a contribution to the study of the stability of steel trusses. The paper proposes an analytical model for the determination of the buckling length coefficient of the truss N by the lateral torsional buckling approach.

The model is derived by considering the equilibrium of the truss under the action of the applied loads and the buckling load. The buckling load is determined by considering the lateral buckling of the equivalent beam and the interaction between the members.

The stability of steel structures has been the subject of many studies[4-11], starting with Horne [12], who equated truss stability to compressed member buckling, while modelling the truss as a bar with elastic supports to taking into account the contribution of braces and members. Morino & al [13] has produced a literature review of truss instabilities by assimilating the system to a lattice column. More recent analytical [14-16], numerical [3, 17] or experimental [2, 8, 9, 18, 19] works have been interested in the buckling stability of the trusses as well as in the buckling length coefficient of the compressed member, for different configurations. The design codes[1, 20], consider checking the lateral buckling stability of a truss beam by considering the buckling of the compressed chord; the results found [2, 3, 17], show that this solution is too safe. Moreover, consideration of the location of the load and consideration of the force of shear in the web of the beam remains limited [14].

Biegus and Wojczyszyn [3] evaluate the out-of-plane loss of stability of chords of one-way trusses. The values of the out-of-plane buckling length coefficients of the chords have been presented. The model of the compression chord as a straight member isolated from the one-way truss associated with the space model of lateral buckling of the truss has been analysed.

The problem has been studied parametrically by varying the number of web plates. For the models studied, comparative analyses of the out-of-plane buckling length coefficients have been carried out regarding the guidelines of selected design codes.

This article presents a general analytical approach to obtain the expression of the buckling length of a chord of N-truss by the LTB approach. The use of the notion of the equivalent web in the development of a critical moment formula of N trusses will allow us to analyze the effect of the position of the load in the calculation of the critical moment of LTB of an N truss considering the equivalent web. After that, an expression of the buckling length coefficient μ_z by the LTB approach with the equivalent web has been proposed. The results will be compared and discussed with those found in the literature.

1. METHODS

1.1. Basic assumptions

According to the Timoshenko theory [21], the model is based on the following assumptions:

The truss is composed of straight, slender members.

The members are connected by rigid joints.

The loads are applied at the joints.

The buckling is assumed to be elastic.

1.2. Critical moment expression

In the case of I-section beams, the usual differential equation of the elastic curve assumes that the curvature is proportional to the bending moment. In truss beams, the deformations due to shear stress cannot be neglected. The equivalent web of a truss beam is the cross-sectional area "A_a" of the web which would have the same deformation under the

influence of shear stress. It is only a convenient device for the calculation of deformations due to shear stress.

The value of "A_a" is determined so that the transverse deformation of a panel of length l_m of a reticulated member, whose distance between the neutral fibbers of the members is h_0, as shown in Figure 1 and which is subjected to a shear stress V_z, for N truss, is written under equation 1:

$$A_a = \frac{2{,}6\, A_d \dfrac{l_m}{h_0}}{\dfrac{A_d}{A_t} + \dfrac{l_d^3}{h_0^3}} = \frac{2{,}6\, A_d \cot g\, \theta}{\dfrac{A_d}{A_t} + \dfrac{1}{\sin^2 \theta}} \qquad (1)$$

Eurocode 3 [1] defines the critical moment of a double-symmetric beam subjected to a uniform load as given in equation (2).

$$M_{cr} = C_1 EI_z \frac{\pi^2}{L^2}\left[-C_2 e_z + \sqrt{\left[C_2 e_z\right]^2 + \left\{\frac{I_\omega}{I_z} + \frac{GJ}{EI_z}\frac{L^2}{\pi^2}\right\}}\right] \qquad (2)$$

Furthermore, the following parameters designate:

L length of steel beam;

E modulus of elasticity;

I_z second moment of the area to z axes;

I_ω warping constant;

J torsional constant;

G shear modulus;

e_z the eccentricity between the shear centre and load;

$C_1; C_2$ factors depending upon the loading and restraint conditions.

To take into account the effect of the equivalent web, we develop the expression of the critical moment from the total potential of a bi-symmetrical truss beam; This total potential is developed for an I-section beam from the kinematics by Belaid et al [7].

The potential of a Truss Beam, considered as an I beam with an equivalent web is given by: equation 3

$$\Pi_b = \frac{1}{2}\int_L \begin{bmatrix} EI_z (v'')^2 + GJ(\varphi_x')^2 + EI_\omega (\varphi''_x)^2 \\ +2(M_y - \rho q_z)(v''\varphi_x) \end{bmatrix} dx + \int_L \frac{1}{2}(z_Q - z_c) q_z \varphi^2{}_x\, dx \qquad (3)$$

With $\rho = \dfrac{EI_y}{GA_a}, q_z = -\dfrac{dV_z}{dx}, M_y(x) = \left[\left(\dfrac{q_z}{2} x\left[L - x\right]\right)\right]$ (4.a-4.b4.c)

Two main methods are used in the literature for the calculation of critical loads, namely the Ritz and Galerkin methods. With both methods, the displacement modes are first approached by functions that are compatible with the boundary conditions of the beam. While knowing that the resolution by the Galerkin method was solved by Ammari et al. [14]

It is important to note that solving an equation by approximate methods requires proposing a solution to that equation that satisfies the boundary conditions. The adopted solution represents the shape of the torsional deformation, which is taken as a trigonometric series. However, the solution of the buckling mode is known to be sinusoidal [7, 21].

Therefore, for the load cases studied in this paper, the solution will be of the first-order sinusoidal. This solution, which is an approximate solution, has been assumed by many authors[6, 7, 14, 21, 22].

Approximated functions are adopted for the solutions of the lateral displacement and the torsional rotation and are given as follows:

$$v = v_0 \sin\frac{\pi x}{L} \qquad \varphi_x = \varphi_0 \sin\frac{\pi x}{L} \qquad (5a\text{-}5b)$$

These functions fit ideally the boundary condition of the beam $v(0) = v''(0) = 0$ and: $\varphi_x(0) = \varphi_x''(0) = 0$

According to the theory of the minimum of total potential energy [6], one gets: $\dfrac{\partial \Pi}{\partial \varphi_0} = \dfrac{\partial \Pi}{\partial v_0} = 0$

The system of an equivalent simply supported steel beam loaded by uniform was developed,

Therefore, by setting the determinant equal to zero we obtain the expressions of the critical loads as given in(6)

$$q_{cr} = \frac{-EI_z(ez)\frac{L}{2} \pm \sqrt{\left[EI_z(ez)\frac{L}{2}\right]^2 + 4\left[\frac{(\pi^2+3)}{24}L - \rho\frac{\pi^2}{2L}\right]^2 \frac{2L^3}{\pi^4} EI_z \frac{\pi^2}{2L}\left\{EI_\omega \frac{\pi^2}{L^2} + GJ\right\}}}{2\left[\frac{(\pi^2+3)}{24}L - \rho\frac{\pi^2}{2L}\right]^2 \frac{2L^3}{\pi^4}} \quad (6)$$

Multiplication of the critical load by L²/8 and adaptation to the Eurocode, the critical moment become:

$$M = C_1^v EI_z \frac{\pi^2}{L^2}\left[-C_2^v e_z + \sqrt{\left[C_2^v e_z\right]^2 + \left\{\frac{I_\omega}{I_z} + \frac{GJ}{EI_z}\frac{L^2}{\pi^2}\right\}}\right] \quad (7)$$

Where C_1^v and C_2^v given by Eq. (8-a and 8-b) are factors depending upon the loading (uniform load for this case) taken into account the shear deformation of the equivalent web.

$$C_1^v = \frac{\pi^2}{16\left[\frac{(\pi^2+3)}{24} - \rho\frac{\pi^2}{2L^2}\right]}, C_2^v = \frac{1}{\left[\frac{(\pi^2+3)}{6} - 2\rho\frac{\pi^2}{L^2}\right]} \quad (8a; 8b)$$

2. RESULTS AND DISCUSSION

2.1. Parametric study

In the case of a symmetrical truss beam, the Top and Bottom Chords are the same, so the geometrical characteristics of the truss beam are calculated as follows:

$$I_y \approx 2 \times A_m \times \left(\frac{h_0}{2}\right)^2; \quad (9.a)$$

$$I_z = 2 \times I_{zm} \quad (9.b)$$

$$J = 2 \times J_m + J_{tr}$$

with $$J_{tr} = \frac{2{,}6\, K_{tr}\, I_d \cot g\,\theta}{\frac{I_d}{I_t} + \frac{1}{\sin^3\theta}} \quad (9.c)$$

$$I_\omega = \frac{I_z \times h^2}{4} \quad (9.d)$$

$$L = nh_0 tg\theta \quad (9.e)$$

Where:

A reference cross-sectional area (unit);

$A_m = m \times A$ cross-sectional area of a chord;

$A_t = t \times A$ cross-sectional area of a strut;

$A_d = d \times A$ cross-sectional area of a diagonal;

h_0 distance between the centres of gravity of the two chords;

I_{zm} second moment of area (for z axes of the truss beam) of a chord;

J_m torsional constant of a chord;

J_{tr} torsional constant moment of the truss beam;

K_{tr} coefficient of the connection conditions of the & trusses on the chords;

I_t et I_d second moment of area of struts and diagonals respectively;

n number of panels constituting the beam;

θ the degree of inclination of the diagonal about the chord.

Replacing equation (9) in equation (7) allows us to write the critical moment as a function of the characteristics of the components of the truss in equation (10)

2.2. Effect of the equivalent web section on coefficients C_1^v and C_2^v

While knowing that C_1 depends on the moment distribution and C_2 involves the position of the load and in the absence of the effect of the shearing force ($\rho = 0$), the coefficients defined in equation (8) whose values are 1.150 and 0.466 respectively are close and similar to those of Eurocode 3.

The use of Galerkin's method makes it possible to have in the case of ($\rho = 0$) the same values as those given by the codes[14].

When we compare C_1^v and C_1 of a truss beam excluding the effect of the shear force with that of a simply supported beam loaded by uniform load, the difference is not very significant, this value was obtained for I section beam solved by the Ritz method [6, 7] The development carried out in this work assumes that for a descending load on the Top chord we have ($e_z = +h_0 / 2$) and for an ascending load on the Bottom chord we have ($e_z = - h_0 / 2$). The relationship between the section height and the length of a beam is given by (9.e) and shown in Figure 1.

The coefficients C_1^v and C_2^v vary according to the ratio ρ / L^2. This ratio depends on the cross-sections of the chords, diagonals and trusts. So it depends on the ratios "m, t, d" respectively, as well as the number of truss panels "n".

Figures 3 a -b show the variation of the coefficients C_1^v and C_2^v respectively according to the number of panels "n" , for chord and diagonal section ratios (m/d =, 1, 1,5 and 2).

$$M_{cr} = 2C_1^v EI_{zm} \frac{\pi^2}{n^2 h_0^2 tg^2\theta} \left[-C_2^v \alpha \frac{h_0}{2} + \sqrt{\left[C_2^v \alpha \frac{h_0}{2} \right]^2 + \left\{ \frac{h_0^2}{4} + \frac{\left(J_m + \frac{J_{tr}}{2} \right)}{2.6 \times I_{zm}} \frac{n^2 h_0^2 tg^2\theta}{\pi^2} \right\}} \right] \quad (10)$$

α : Takes into account the position of the load.

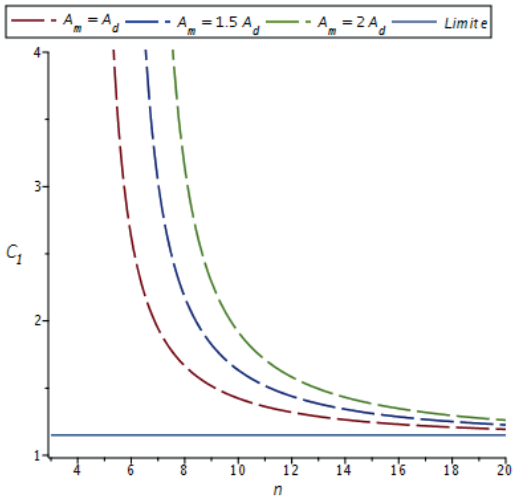


Figure 3-a : variation of C_1^v according to the number of panels "n"

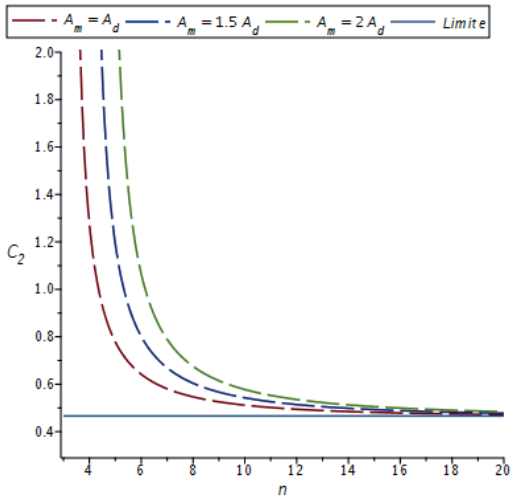


Figure 3-b: variation of C_2^v according to the number of panels "n

We observe that for the number of panels greater than 15, for any section ratio "m/d", the values of C_1^v and C_2^v tend towards the values of 1.150 and 0.466 respectively, which are obtained without taking into account the effect of the shear force or those of a free beam under uniform load. This can be explained by the fact that when the beam is slender, the shear effect

on the beam has a very small influence unlike the case of short beams, where we observe that the coefficients C_1^v et C_2^v take higher values and are inversely proportional to the number of panels. It is also important to note that the cross-section of the chords has a significant and proportional influence on the coefficients and thus on the critical moment, regardless of the ratio of the cross-sections.

2.3. Buckling length of the chord by the Lateral Torsional Buckling approach

The critical force N_{crz} of the chords for the out-of-plane buckling has been obtained by dividing the critical moment by h_0:

$$N_{crz} = M_{cr} / h_0 \tag{11}$$

The coefficients of buckling lengths have been determined using equation [1, 3, 17]:

$$\mu_z = \frac{\pi}{L}\sqrt{\frac{EI_{zm}}{N_{crz}}} \tag{12}$$

EI_{zm} : Represents bending stiffness of the chord in the plane of buckling.

By substituting (Eq. 10) and (Eq. 11) into (Eq. 12), then the coefficients of buckling lengths can be expressed as:

$$\frac{1}{\mu_z} = \sqrt{C_1^v\left[-C_2^v\alpha + \sqrt{\left[C_2^v\alpha\right]^2 + \left\{1 + 4\frac{G}{E}K\frac{n^2}{\pi^2}\right\}}\right]} \tag{13}$$

With: $K = \left(J_m + \frac{J_{tr}}{2}\right) / I_{zm}$ (14)

as mentioned above, the coefficient K takes into account the ratio of torsional and bending stiffness

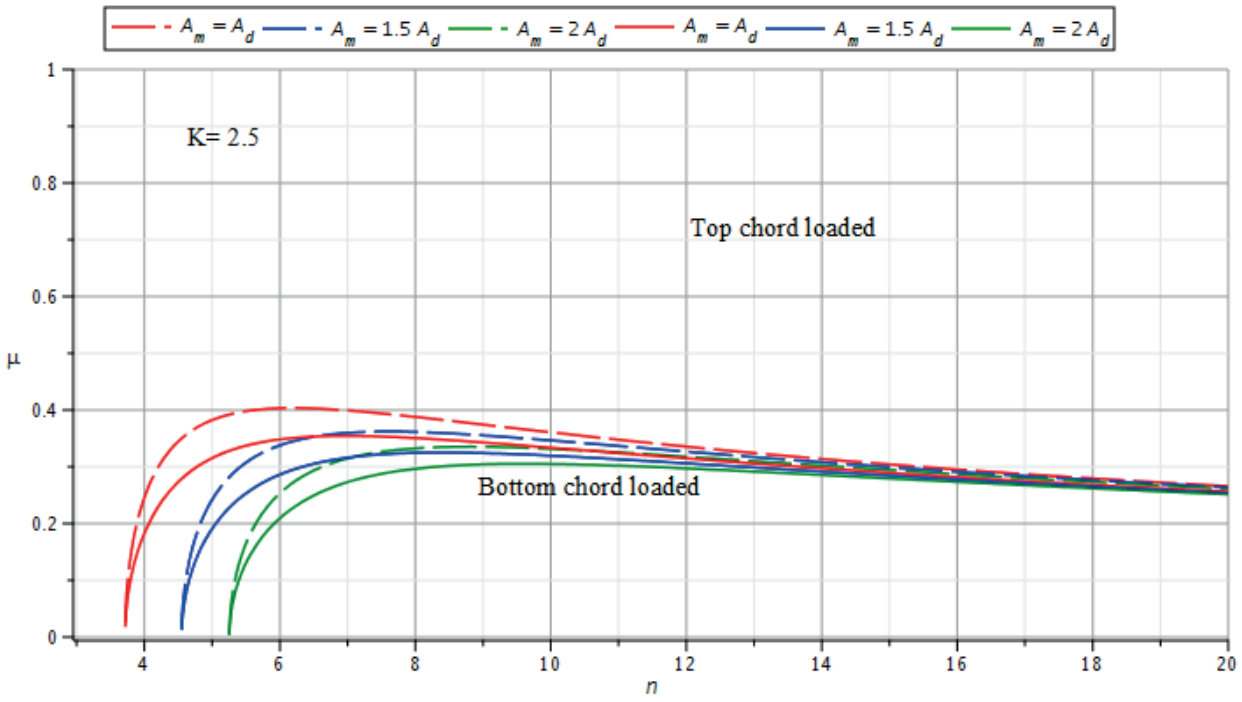


Figure 4. Coefficients of buckling lengths according to the number of panels K =2.5

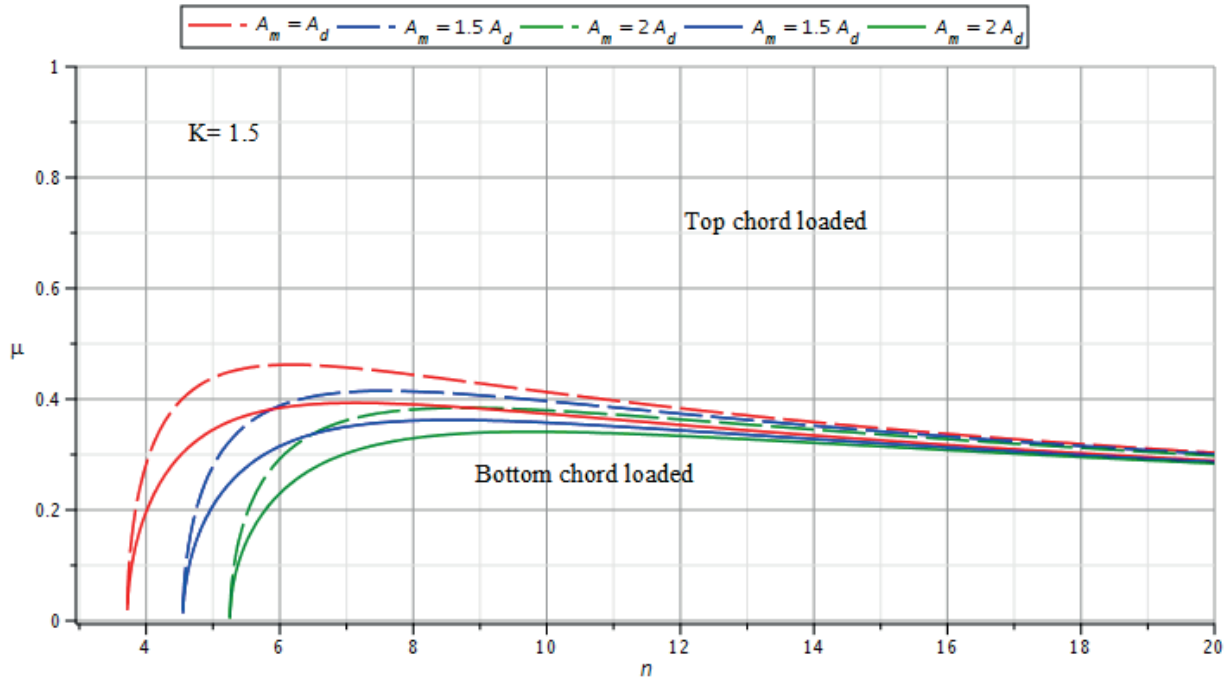


Figure 5. Coefficients of buckling lengths according to the number of panels K =1.5

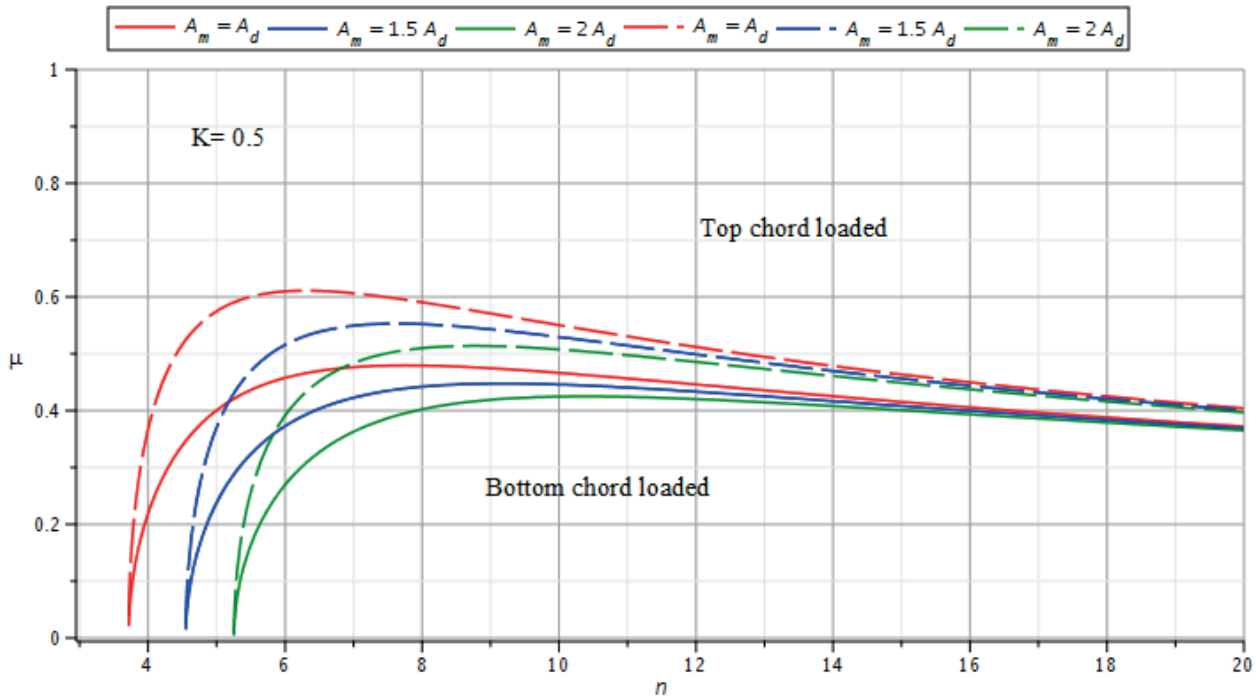


Figure 6. Coefficients of buckling lengths according to the number of panels K =0.5

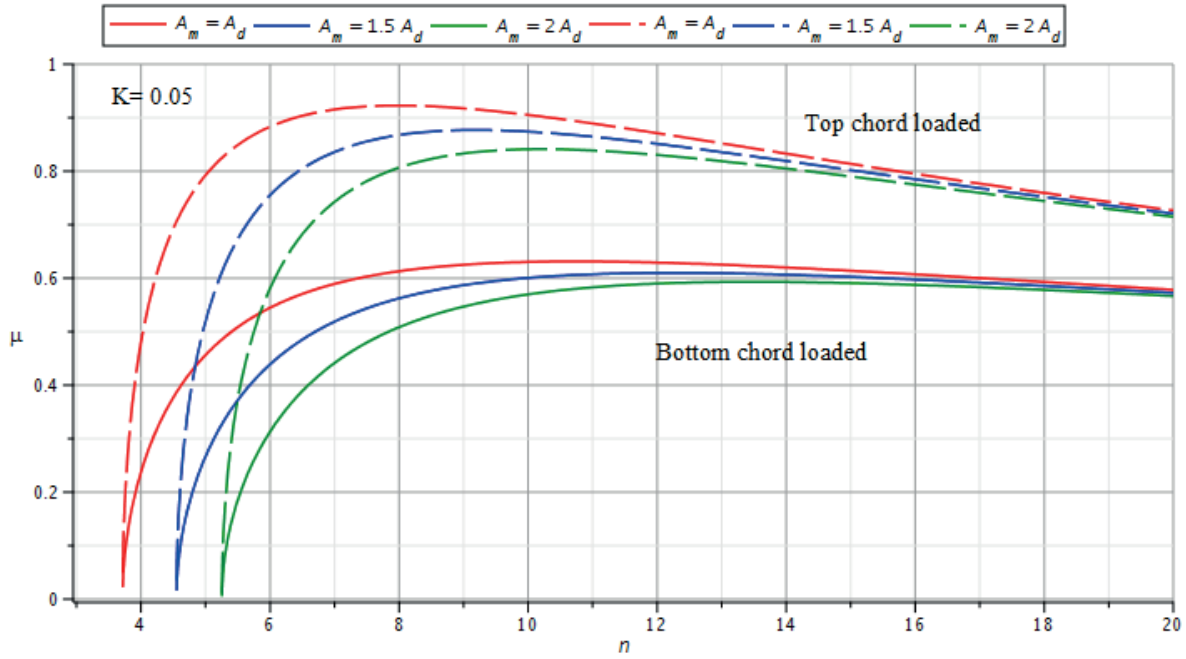


Figure 7. Coefficients of buckling lengths according to the number of panels K =0.05

This ratio depends on the support conditions which are represented by (Eq 9-c)

For $\theta = 45°$ and $A_d = A_t$, Figures 4, 5, 6 and 7 show the values of the coefficient μ_z as a function of the number of panels constituting the truss for chord and diagonal section ratios (m/d =, 1, 1,5 and 2), this for two cases of load, namely: load applied at the Top chord (TC); load applied at the Bottom chord (BC),

The values of K, are chosen to reflect the respective cases of a beam composed by :

- tubular section with a rigid connection K=2.5;
- hinged tubular section K= 1.5;
- open sections with rigid connection K=0.5;
- hinged open rolled sections K= 0.05;

We observe, for all the cases studied, that the coefficient μ_z varies significantly with the number of panels, and this occurs in two phases: For a number of panels less than 6, the coefficient shows an ascending variation. This can be explained by the fact that the beam is not highly slender, and thus, the effect of shear rigidity (equivalent web) favours the risk of local buckling of the components (diagonal and strut).

For a number of panels greater than 6, the effect of shear rigidity diminishes, favouring the risk of global buckling of the beam.

Nevertheless, for all the cases studied, the coefficient remains below the regulatory limits. The use of tubular sections, as observed in Figures 4 and 5, shows that the variation of the coefficient μ_z is not significantly affected by the variation in the sections composing the truss or the effect of load position, unlike the use of rolled sections (Figures 6-7). The maximum value that the coefficient μ_z can reach does not exceed 0.5 for the most unfavourable case, namely, a hinged connection of the components (K=1.5), the section of the web being the same as that of the uprights and diagonal, and the load being applied to the upper chord (destabilizing effect of the load). This leads us to confirm the remarks made by Biegus and Wojczyszyn **[3]**that, in this case, the calculation codes overestimate the values of μ_z.

The use of rolled sections (Figures 6-7) shows that the variation of the coefficient μ_z is

affected by the variation in the sections composing the truss or the effect of load position, unlike the use of tubular sections. The maximum value that the coefficient μ_z can reach is 0.9 for the most unfavourable case, namely, a hinged connection of the components (K=0.05), the section of the web being the same as that of the uprights and diagonal, and the load being applied to the upper chord (destabilizing effect of the load). The value obtained by our approach is the one used by Eurocode 3 [1].
In comparison with the limits given by Eurocode [1], we can see that the buckling length coefficient varies considerably depending on the number of panels, the cross-sections constituting the truss as well as the position of the load. The values are given by Eurocode [1], remain too safe when the Bottom Chord is loaded.

2.4. Comparison and example

The purpose of this section is to compare the coefficient μ_z developed for the uniform load defined by Equation (13) with those reported in the literature and which is presented in Section 1. likewise, Table 1 compares the coefficient μ_z
In this table, columns from 1 to 8 represent respectively number of panel(1), limit given by NF EN 1993-1-1/NA [23](2), coefficients of buckling lengths obtained by the present work (3) and (5) , as well as those of Biegus and Wojczyszyn **[3]** (4) and (6) , and, comparison between the code and the present work are given on the columns (7) and (8) for the case of loads applied at the Top chord and Bottom chord respectively.
Although the model of beams developed by Biegus and Wojczyszyn **[3]**, and the model developed in the present work are different, as well as the method, the results are very similar for the case of loads applied at the upper chord. Table 1 shows the values obtained for K=1.5 using equation (13) and those obtained by Biegus and Wojczyszyn **[3]** in the case of a truss composed by RHS.
The buckling lengths assumed according to Eurocode for the out-of-plane instability of the truss may be overestimated for the cases when the buckled segment covers several panels of the system. This effect leads to the underestimation of the critical load and in sequence to uneconomical consumption of material.

Table 1. Comparison of coefficients of buckling lengths μ_z
coefficients of buckling lengths μ_z

number of panels	*EC3*	Top chord		Bottom chord		Comparison Δ=EC-Eq/EC	
1	*2*	3	4	5	6	7	8
"n"	[1]	Eq,(13)	[3]	Eq,(13)	[3]	α = +1	α = -1
4	0,9	0,7	0,71	0,57	0,64	22%	37%
5	0,9	0,63	0,65	0,53	0,6	30%	41%
6	0,9	0,57	0,6	0,49	0,56	37%	46%
7	0,9	0,53	0,56	0,47	0,52	41%	48%
8	0,9	0,49	0,53	0,44	0,5	46%	51%
9	0,9	0,46	0,5	0,41	0,47	49%	54%

The examination of the results has shown the significant effect of the Equivalent web, this model constitutes a more accurate basis of computation and material savings in comparison code. The buckling length coefficients of the studied cases have given μ_z = 0.70–0.57, which

increases the critical load about 22% and 37% (for n = 4), and 49% - 54% (for n = 9) referred to μ_z = 0.9 , for the load applied at Top and Bottom chord respectively

However, it is important to note that in all the cases studied, the coefficient μ_z remains below the regulatory limits, indicating that the truss structures analyzed meet the required stability criteria.

We also compared the influence of different types of sections on the coefficient μ_z. The use of tubular sections showed less significant variation in the coefficient compared to rolled sections, regardless of changes in section composition or load position. This highlights the advantage of using tubular sections in terms of stability performance.

Furthermore, our results aligned with previous findings by Biegus and Wojczyszyn, supporting the observation that existing calculation codes tend to overestimate the values of μ_z for the specific case analyzed.

CONCLUSION

This work has allowed establishing an analytical expression of the critical buckling moment for an N-truss, taking into account the influence of the load through the use of coefficients C_1^v and C_2^v . By using the LTB approach with "the equivalent web", an expression of the buckling length coefficient μ_z has been proposed. In comparison with the limits given by Eurocode [1], we can see that the buckling length coefficient varies considerably depending on the number of panels, the cross-sections constituting the truss as well as the position of the load. The values given by Eurocode [1] (1 and 0.9 respectively), remain too safe when the Bottom chord is loaded.

In conclusion, our study focused on the variation of the coefficient μ_z in truss structures. We observed that the coefficient exhibits two distinct phases depending on the number of panels. For a lower number of panels, the coefficient increases due to the beam's lower slenderness, which enhances the risk of local buckling. Conversely, for a higher number of panels, the effect of shear rigidity diminishes, leading to an increased risk of global buckling.

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Furthermore, our results aligned with previous findings by Biegus and Wojczyszyn, supporting the observation that existing calculation codes tend to overestimate the values of μ_z for the specific case analyzed.

Overall, our study provides valuable insights into the behaviour of truss structures and emphasizes the importance of considering the number of panels and section types when assessing their stability. These findings contribute to the ongoing refinement of design codes, such as Eurocode 3, for accurate and efficient structural analysis

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