Stability Analysis of a Scaled-Down Cold-Formed Steel Structure

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Abstract. The use of cold-formed steel CFS in building design has increased in the last period due to the numerous advantages it brings. However, there are relatively few research works on the behaviour of these types of structures to different loadings, especially compared to the hot-rolled steel structures. Unlike the hot-rolled steel elements, the CFS elements are prone to buckling failure. This paper investigates the stability of a scaled-down CFS structure considering the Eurocode EC and the American International Building Code IBC. First a comparison is done between the two codes in what concerns the stability analysis based on the theoretical aspects. Then, the results from the finite element program are presented, for both EC and IBC, and compared.

1. Introduction
Cold-formed steel CFS elements are used all around the world for building both residential and commercial structures. Having a high strength to weight ratio, they are easier to manufacture, to handle and to assemble on site, leading to a significant decrease of the structure’s total cost. Taking into account their mechanical properties, the CFS structures have a different behaviour than the hot-rolled steel HRS ones, and they must be analysed according to their behaviour. Due to their slenderess ratio and, generally, thin walled open cross-sections, CFS elements fail by buckling before entering the plastic domain [1]. They can experience local, distortional and global buckling or even a combination of the mentioned phenomena which can lead to the failure of the entire structure. [2, 3, 4]

Taking into account the world-wide use of the CFS elements and the different national regulations, differences may appear in the structural design. Structural design using CFS is a laborious process especially due to the presence of local and / or global buckling modes, sometimes the two phenomena interacting with one another [5] This paper compares the stability analysis results of a CFS structure, for two of the most important building codes, namely the Eurocode (EC) and the American International Building Code (IBC). The comparison was carried out on a numerical model of a scaled down CFS structure.

2. Comparison between Eurocode EC and the International Building Code IBC
In this paper, only the differences which affect the stability analysis results will be presented. The regulations concerning the design of the CFS structures are found in EUROCODE 3: Design of Steel Structures, more specifically in SR EN 1993-1-3-2007: Design of Cold-Formed Steel Structures. The national annex considered in the analysis is the Romanian one. In the American norm, the code for designing the cold-formed steel structures is AISI S100-2016: North American Specification for the Design of Cold-Formed Steel Structural Members. The American code gives two design possibilities for the Ultimate Limit State ULS: Load and Resistance Factor Design (LRFD) and Allowable Strength Design (ASD).

The load combinations are an essential part of the stability analysis, the structure behaving in a different manner depending on the loading. Thus, the partial safety coefficients were taken into account in the analysis, and they are shown in Table 1, according to Table NA A1.2(B) from the Romanian national annex SR EN 1990-2004_NA-2006 [6] and to Chapter 2 from the American code ASCE/SEI 7-10 [7].
Table 1. Load combinations.

| EUROCODE          | ASCE/SEI 7-10  |
|-------------------|----------------|
| 1.35‧DL           | 1.40‧DL        |
| 1.35‧DL+1.50‧LL+1.05‧SL | 1.20‧DL+1.60‧LL+0.50‧SL |
| 1.35‧DL+1.50‧SL+1.05‧LL | 1.20‧DL+1.60‧SL+0.5‧LL |

where DL is the dead load, LL – live load and SL – snow load.

One major difference between the two standards is the sets of units which are used. The EC adopted the metric units, which are largely used in Europe, in comparison with the IBC which uses the imperial format. This leads to a slight difference in material properties and sectional dimensions. In what concerns the permanent and the imposed loads, both norms evaluate them in a similar manner.

For the purpose of comparison, it is mandatory to first correlate the symbols used in each norm with the definition. Table 2 presents a list of these symbols based on the EC and the American National Standard ANSI/AISC 360-16 [8], along with their corresponding definitions [9].

Table 2. List of symbols found in EC and ANSI/AISC 360-16.

| Definition                        | Symbol | EUROCODE | ANSI/AISC 360-16 |
|-----------------------------------|--------|----------|------------------|
| Minimum yield stress              | $f_y$  | $F_y$    |                  |
| Minimum tensile stress            | $f_u$  | $F_u$    |                  |
| Design resistance – bending moment| $M$    | $M$      |                  |
| Design resistance – axial force   | $N$    | $P$      |                  |
| Cross-sectional area              | $A$    | $A$      |                  |
| Elastic section modulus           | $W$    | $S$      |                  |
| Load factor                       | $\gamma$ | $\gamma$ |                  |

This paper analyses the stability of a CFS structure, buckling being one of the ways in which the structure could reach failure [10]. Therefore, the formulas regarding the elements’ buckling resistance must be analysed for both EC and ANSI/AISC 360-16 [8]. Any difference could lead to distinct stability analysis results.

According to SR EN 1993-1-1-2006 [11], the buckling resistance of a compressed element is computed depending on the class of the cross-section, as presented in Equations 1 and 2. The assigning of an element to a certain class is established depending on the way the element behaves when it reaches local buckling.

$$N_{b,Rd} = \frac{\chi A f_y}{\gamma_{M1}}$$ for Class 1, 2 and 3 cross-sections (1)

$$N_{b,Rd} = \frac{\chi A_{eff} f_y}{\gamma_{M1}}$$, for Class 4 cross-sections (2)

where $\chi$ is the buckling reduction factor, $A$ – the cross-sectional area, $A_{eff}$ – the effective area of the cross-section subjected to uniform compression, $\gamma_{M1}$ – partial safety coefficient.

The buckling reduction factor $\chi$ for elements centrically compressed is computed with Equation 3.
\[ \chi = \frac{1}{\phi + \sqrt{\phi^2 + \lambda^2}} \]  
\[ \chi \leq 1 \]  

in which

\[ \phi = 0.5 \cdot [1 + \alpha \cdot (\tilde{\lambda} - 0.2) + \tilde{\lambda}^2] \]  

\[ \tilde{\lambda} = \sqrt{\frac{A \cdot f_y}{N_{cr}}} \] for Class 1, 2 and 3 cross-sections  

\[ \tilde{\lambda} = \sqrt{\frac{A_{eff} \cdot f_y}{N_{cr}}} \] for Class 4 cross-sections  

\[ \alpha \] – the imperfection factor  

\[ N_{cr} \] – the critical buckling force, for the considered buckling mode.

The critical buckling force \( N_{cr} \) can be computed using Euler’s formula, where \( L_{cr} \) is the critical buckling length, \( E \) is the modulus of elasticity and \( I \) – the minimum moment of inertia of the cross-section.

\[ N_{cr} = \frac{\pi^2 E I}{L_{cr}^2} \]  

Unlike the EC, the American norm ANSI/AISC 360-16 [8] considers in the analysis the nominal compressive strength \( P_n \), computed, in this case, with the help of Equation 8.

\[ P_n = F_{cr} \cdot A_g \]  

The critical stress \( F_{cr} \) is computed with Equations 9 and 10 [8].

- When \( \frac{L_{cr}}{r} \leq 4.71 \cdot \sqrt{\frac{F_e}{F_y}} \), \( F_{cr} = \left( 0.658 \sqrt{\frac{F_e}{F_y}} \right) \cdot F_y \)  
- When \( \frac{L_{cr}}{r} > 4.71 \cdot \sqrt{\frac{F_e}{F_y}} \), \( F_{cr} = 0.877 \cdot F_e \)  

where \( A_g \) is the gross cross-sectional area of the element, \( E \) – the modulus of elasticity, \( r \) – the radius of gyration and \( F_e \) – the elastic buckling stress determined with Formula (11).

\[ F_e = \frac{\pi^2 E}{(\frac{L_{cr}}{r})^2} \]  

Equation 11 given by the American norm is an equivalent of the Euler’s critical buckling force, computed with Equation 7, but in terms of stresses. Therefore, although the two norms have different approaches in what concerns the buckling analysis, i.e. forces and stresses, the considered effects of the compressive force are similar [9].

3. Structural model

The structural elements were manufactured from a DX51D+Z steel sheet and, using a cold formed procedure, they were bent into a C89/41/12/1 profile. The connections of the elements were made using 2x Ø4x22 mm self-drilling self-tapping screws. Although the sectional dimensions are different in the United States and the materials used have slightly different values regarding the mechanical properties, the structure will not be changed and the analysis will be done on the initial structure having the above-mentioned geometrical dimensions.

The structural model dimensions are 2.30 x 2.70 x 2.60 m. The CFS scaled-down structure is presented in Figure 1. For the analysis, the structure was modelled in the SCIA Engineer v 18.1 software, where the connections were considered to be rigid. The structural model introduced in the finite element program is shown in Figure 2. The structure was subjected to a roof permanent load of 0.5 kN/m² and a snow load of 2 kN/m². Although the value of the snow load is different in the United States, for this
analysis it will be considered the same as in the EC case, the purpose being to observe the possible
difference in the obtained results based on the design philosophies of the two codes.

![Scaled-down CFS structural model](image1)

![CFS model in SCIA Engineer](image2)

Figure 1. Scaled-down CFS structural model

Figure 2. CFS model in SCIA Engineer

The analysis within SCIA Engineer is done according to the selected national code and national annexes. For the current analysis, two national codes are selected, namely the Eurocode EC and the International Building Code IBC. For the EC, the national annexes considered are the Romanian ones, which contain some additional information regarding EC parameters and establish national values. In what concerns the IBC, there are preset codes as follows:

- Code for loads: IBC 2012 (ASCE 7-10);
- Code for steel structures: AISC 360-16;
- Code for cold-formed steel structures: AISI S100-2016.

![Table 3. Stability combinations.](table3)

| Load case       | Eurocode coefficients | International Building Code coefficients |
|-----------------|-----------------------|------------------------------------------|
|                 | S1EC                  | S2EC          | S1IBC | S2IBC |
| Dead load       | 1.35                  | 1.35          | 1.20  | 1.40  |
| Roof dead load  | 1.35                  | 1.35          | 1.20  | 1.40  |
| Snow            | 1.50                  | -             | 1.60  | -     |

In SCIA Engineer software, in order to run the stability analysis, it is required firstly to define the stability combinations. These are based on the linear combinations assembled automatically by the program based on SR EN 1990-2004_NA-2006 [6] for the European norm and ASCE/SEI 7-10 [7] for the American one. Two stability combinations, i.e. S1 and S2, were considered for both EC and IBC, as presented in Table 3.

Also, each element is assigned to a buckling group, which defines the buckling constraints and the span settings.

4. Stability analysis results
4.1 Critical load factor
The first step in a stability analysis is to compute the critical load factor $\alpha_{cr}$, as presented in Equations 12 and 13. The principle of calculus is the same in both EC and IBC. Based on this value, it can be established if the deformations are significant enough to cause an increasing of the forces or a change in the global behavior of the structure. In case of large deformations, it is recommended to run a 2\textsuperscript{nd} order calculus and, if necessary, to include global and/or local imperfections, in order to obtain more accurate results. The limit values of $\alpha_{cr}$ up to which a 2\textsuperscript{nd} order analysis is required are presented in Equations 12 and 13, for elastic and plastic analysis, respectively [11].

$$\alpha_{cr}=\frac{F_{cr}}{F_{Ed}}<10, \text{ for elastic analysis}$$

(12)

$$\alpha_{cr}=\frac{F_{cr}}{F_{Ed}}<15, \text{ for plastic analysis}$$

(13)

where $F_{Ed}$ is the design load which acts upon the structure, $F_{cr}$ – elastic critical buckling load.

Table 4 presents comparatively the critical load factors $\alpha_{cr}$ of the considered structure obtained by using European and American codes, for the first three buckling modes.

| Buckling mode | Eurocode | International Building Code | Comparison (IBC relative to EC) |
|---------------|---------|-----------------------------|---------------------------------|
|               | S1EC    | S2EC | S1IBC | S2IBC | S1 | S2  |
| 1             | 218.58  | 914.94 | 213.41 | 882.27 | -2.37% | -3.57% |
| 2             | 232.52  | 947.57 | 227.28 | 913.73 | -2.26% | -3.57% |
| 3             | 245.56  | 1023.04 | 239.80 | 986.5 | -2.35% | -3.57% |

By comparing the critical load factors based on the EC with the ones based on the IBC, according to Table 4, it results slightly smaller values using IBC. The difference is given mainly by the partial safety coefficients used in the load combinations, the codes using different values, as shown in Tables 1 and 3. Therefore, for the first stability combination S1, which includes the snow load, the IBC values are smaller with approximately 2.30%, whereas for the second stability combination S2, made up only from permanent loads, the values are smaller with 3.57%.

Moreover, taking into account that the critical load factors $\alpha_{cr}$ are greater than 15, according to Equation 13 [11], it can be established that no 2\textsuperscript{nd} order calculus is required, the structure itself failing due to the failure of its members when the strength condition is no longer fulfilled.

4.2 Nodal displacements
Figure 3 shows the deformed shape of the structure corresponding to the stability combinations S1 and S2. The roof has been filtered out in order to have a better view upon the structural elements which are prone to buckling. Both EC and IBC have similar results in what concerns the deformed shape, therefore, only one situation is presented.
Figures 4-7 present the displacement of nodes in Ox and Oy direction, obtained from stability combinations S1 and S2 and based on EC and IBC. The values are computed only for the first buckling mode. The buckling shapes and the values are normalized so that the maximum nodal displacement component is 1 m. The analysis results shown that this maximum value was obtained for the rotation about Oz axis of column bordering the door opening.

**Figure 3.** Deformed CFS structure  
a) Stability combination S1  
b) Stability combination S2

**Figure 4.** Global maximum displacement of nodes on Ox direction - stability combination S1  
a) Eurocode; b) International Building Code

**Figure 5.** Global maximum displacement of nodes on Ox direction - stability combination S2  
a) Eurocode; b) International Building Code
By analyzing Figures 5-7, it can be observed that the global maximum displacement for Ox and Oy directions is obtained in the same node for both EC and IBC, and the values are close to one another. Table 5 presents a comparison between the global maximum displacements resulted considering EC and IBC, for S1 and S2, and for Ox and Oy directions.

### Table 5. Global maximum node displacements

| Direction | Eurocode [mm] | International Building Code [mm] | Comparison (IBC relative to EC) [%] |
|-----------|---------------|----------------------------------|-----------------------------------|
| S1EC      | S2EC          | S1IBC   | S2IBC   | S1    | S2    |
| Ox        | -79.3         | -85.6   | -79.1   | -85.6 | -0.25 | 0     |
|           | 41.0          | 45.3    | 40.8    | 45.3  | -0.49 | 0     |
| Oy        | -14.1         | -9.5    | -14.3   | -9.5  | 1.42  | 0     |
|           | 687.4         | 691.1   | 687.3   | 691.1 | -0.02 | 0     |
The difference between the maximum node displacements obtained according to EC and to IBC is very small, so it can be stated that the codes lead to approximately the same result. The large values for the maximum lateral displacements support the observations made in terms of the critical load factor that is the elements of the structure fail due to the values of stresses exceeding the strength of steel. The buckling, in the case of the investigated structure, occurs in the plastic range of the material behavior [4].

4.3 Steel slenderness
Taking into account that the two codes have similar approaches in what concerns the computation of the parameters related to buckling, as described in Chapter 2, the steel slenderness results are the same. Table 6 presents the global extreme values of the steel slenderness corresponding to the y-axis obtained in SCIA Engineer, where \( L_y \) is the system length for buckling around y-axis, \( k_y \) – the buckling ratio for buckling around y-axis, \( l_y \) – buckling length for buckling around y-axis and \( \lambda_y \) – the slenderness around y-axis.

| Member | \( L_y \) [m] | \( k_y \) [-] | \( l_y \) [m] | \( \lambda_y \) [-] |
|--------|--------------|-------------|--------------|---------------|
| B97    | 0.090        | 10.00       | 0.900        | 25.40         |
| B34    | 2.495        | 1.91        | 4.762        | 134.41        |
| B82    | 0.318        | 10.00       | 3.179        | 89.71         |
| B174   | 1.125        | 5.98        | 6.732        | **190.01**    |
| B35    | 2.495        | 1.47        | 3.674        | 103.71        |

Taking into account the restraint conditions, most elements have a buckling ratio \( k_y \) between 1 and 2. However, there are some elements, shown in Table 6, for which \( k_y \) is greater than 2. In this case are members B97 and B82, for which the resulted buckling ratio \( k_y \) is 10, this being the maximum allowed value, have a small length, namely 9 cm and 31.8 cm. Thus, despite of this high buckling ratio value, the slenderness is relatively small. The maximum slenderness ratio was obtained for member B174, which represents a bottom chord of one of the trusses.

4.4 Stability check
The stability check is done according to SR EN 1993-1-1 and SR EN 1993-1-3 for the European regulations and to AISI S100-16 [12] LRFD for the American regulations. For comparison, two elements will be considered, associated to the global extremes of the unity check obtained by running the stability check considering both codes. The unity check is defined by Equation 14, which checks that the design value of a force does not rise above the corresponding design resistance and, if more forces act upon the element, their combined effects do not overpass this resistance [11]. In case of the American regulations, the unity check is defined similarly, but in terms of stresses in AISI S100-16 [12].

\[
\frac{N_{Ed}}{N_{Rd}} + \frac{M_{y,Ed}}{M_{y,Rd}} + \frac{M_{z,Ed}}{M_{z,Rd}} \leq 1
\]

where \( N_{Ed} \) is the design axial compressive force, \( M_{y,Ed} \) – the design bending moment with respect to y-y axis, \( M_{z,Ed} \) – the design bending moment with respect to z-z axis, \( N_{Rd} \) – design resistance to axial force, \( M_{y,Rd} \) – design resistance to bending moment with respect to y-y axis, \( M_{z,Rd} \) – design resistance to bending moment with respect to z-z axis.

From the stability check according to EC, the global extreme value of the unity check corresponds to element B131, whereas according to IBC, it corresponds to element B149, as presented in Figure 8.
5. Conclusions

The paper presents the results in terms of critical buckling factor and lateral displacements obtained from the numerical analysis of a scaled-down cold-formed steel structure. The results are obtained by applying the buckling analysis as defined by the two major design codes in use nowadays: The Eurocode and The American International Building Code.

Based on the obtained results, the following preliminary conclusions can be drawn:

- The two design codes, although based on two different approaches, lead to similar results in terms of the effects of the compressive forces that may lead to the occurrence of buckling.
- The scaled down model used in the present research exhibited large values of the critical buckling coefficient which may lead to the conclusion that the failure of the steel section due to large stresses occurs before the loss of stability happens. This observation is later supported by the large lateral displacements of the nodes corresponding to the buckling modes along the two in-plane directions.
- The numerical model should be further calibrated by means of laboratory tests. The authors are aware that further research in the field of loss of stability of CFS member should be conducted before reliable conclusions can be reached.

6. References

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