Comparative application of two methods in the second order elastic analysis of the steel structures

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Abstract. In most situations, in the case of metallic structures, the loss of stability occurs before plastic elements enter the structure. The present paper aims to compare the response of a metallic structure obtained by an elastic analysis of the 2nd order developed with the two methods provided in SR-EN 1993-1-1: 1) global analysis that takes full account of geometrical and material imperfections and second-order effects; 2)-individual stability check of the equivalent bars using buckling lengths corresponding to the overall instability mode. Obviously the first method is the most complex because it takes into account both global and local imperfections. The second method is the simplest and most used method. In this case, the verification relationships take account of the imperfections of the elements. For global analysis, the global imperfections and local bar imperfections will be considered. These imperfections will be taken into account in the worst form and sense. The check for global analysis will be done using finite element meshing, while using the geometric nonlinearity option so that the stiffness matrix is calculated at each loading step. The individual check of the elements shall be performed according to SR EN1993-1-1 according to the type of request.

1. Introduction

By this comparative analysis it is obtained an imagine of a result of the verification established by the two methods provided by the normative, thus facilitating a choice of the solution chosen in the design. The high slenderness of the metal structures is important compared to the structures in reinforced concrete frames. Consideration of the effects of the second order is mandatory because the deformations have a significant influence on the stress.

This condition is expressed by:

$$\alpha_g = \frac{F_{cr}}{F_{Ed}}$$

where $$\alpha_g$$ - the coefficient of multiplication for the structure’s elastic instability load; $$F_{cr}$$ - lateral buckling critical elastic load; $$F_{Ed}$$ - the total vertical load applied to the structure.

SR EN 1993-1-1 presents three methods for taking into account the effects of the second order and the imperfections [5]:

- through the global analysis, the imperfections (geometric and material) and the effects of the second order are taken into account;
- the global analysis takes into account the global imperfections and the effects of the second order global (the force-displacement effect) and the stability checks at the element level take into account the local imperfections and the effects of the second order.
for the basic cases, through individual stability checks of the equivalent bars, using buckling lengths corresponding to the global instability mode.

In the present study, the results obtained from the first method, which is the most complex and, will be compared with the third method which is the most used in the design activity. Stability checks for individual elements for center compression, bending or axial force bending are currently performed in the design activity.

In order to analyze the structure of figure 1 with method 1, both global and local imperfections, curves of elements were taken into account. Figure 2 shows the global imperfections for the frames and the local ones for the columns. As with the columns, local imperfections were also used for the beams.

![Figure 1. Initial structure. Transversal framework studied.](image)

#### 2. Imperfection

The first type of imperfection used is the initial global deviation from the vertical axis, figure 2.

This was calculated with the relation [4]:

\[ \varphi = \varphi_0 \cdot \alpha_h \cdot \alpha_m, \]

in which \( \varphi_0 = 200^{-1} \), \( \alpha_h \) is the reduction coefficient applicable to the height \( h \) of the columns:

\[ \alpha_h = 2 \cdot h^{-0.5} = 2 \cdot 7^{-0.5} = 0.7559 \]

where \( h \) is the height of the structure in meters, \( \alpha_m \) is a reduction factor for the number of columns in a row:

\[ \alpha_m = [0.5 \cdot (1 + m^{-1})]^{0.5} = [0.5 \cdot (1 + 2^{-1})]^{0.5} = 0.866 \]

where \( m \) is the number of columns in a row and:

\[ \varphi = 200^{-1} \cdot 0.755 \cdot 0.866 = 3.2 \cdot 10^{-3} \]

For this \( \varphi \) calculated, taking into account the length of the columns, results in a deviation from the vertical of 2.2 cm presented in figure 2.

![Figure 2. Global imperfections for frameworks used in analysis.](image)

The second type of imperfections used in the analysis are the local imperfections in the arch \( e_o \) of the bends (initial curves) for bending buckling.
The value used for the columns is $\frac{e_0}{L} = 250^{-1}$ [4] and was used $e_0 = 2.2 \text{ cm}$ and for beams 4 cm. The table in the National Annex of SR EN 1993-1-1 (the calculation values of the imperfections in the arc) was used [2].

3. Case study

3.1 Linear analysis of individual buckling checks

The case study was performed on the transversally frame of a hall, figure 1, made in Constanta with a snow load of $0.8 \cdot 1.5 \text{ kPa} = 1.2 \text{ kPa}$, and a ground acceleration of $0.2 \text{ g}$. The wind load on the structure was also taken into account. It was considered a behavioral factor. The transversal framework is not constrained in the respective direction, thus, the specific provisions P100-1/2013 was used in the design activity. Thus, the design based on the over resistance $\Omega$ of Annex F (P100-1) was also taken into account [1].

The linear static calculation was performed with the SCIA Engineer program. Initially the calculation was performed spatially on a hall with four openings, the beam of 6m and a layout of panels of 10 panels / opening.

The columns have 7 m and the opening of the hall is 20.7 m. The beam resulted from the IPE450 stability checks and for the HEB300 columns.

Steel mark S355. The following efforts were used for checking the strength and stability of the beam:

- for the constant section: $N_{Ed} = 83.28 \text{ kN}; V_{Ed} = 138.36 \text{ kN}; M_{Ed} = 268.2 \text{ kNm}$
- for the haunch section: $N_{Ed} = 83.28 \text{ kN}; V_{Ed} = 138.36 \text{ kN}; M_{Ed} = 445.61 \text{ kNm}$

![Figure 3. Vertical displacement 175.7 mm for linear calculation.](image)

![Figure 4. Horizontal displacement 25.8 mm for linear calculation.](image)
Figure 5. Bending diagram, on the semi-structure, linear analysis (first case).

For the strength and stability checks at the columns in linear analysis, the following values were used for the M-N interaction: $N_{Ed} = 160.16 \text{ KN}; V_{Ed} = 67.6 \text{ KN}; M_{Ed} = 473.17 \text{ KNm}$.

In the case of the linear elastic calculation, the crossed scales are: a) classification in the class of the section; b) resistance checks; c) buckling checks. The calculation of the interaction factors $k_{yy}$, $k_{zy}$ was performed with method 1.

In the linear analysis, M-N interaction checks were performed for both the column and the beam in the haunch and without a haunch section.

In the case of the columns, the verification of the interaction formulas has the form [2], [3]:

$$
N_{ED} \cdot \gamma_{M1} \cdot (X_{y} \cdot N_{Rk})^{-1} + k_{yy} \cdot M_{Ed} \cdot \gamma_{M1} \cdot (X_{LT} \cdot M_{y, Rk})^{-1} = 0.86 < 1 \quad (5)
$$

$$
N_{Ed} \cdot \gamma_{M1} \cdot (X_{z} \cdot N_{Rk})^{-1} + k_{zy} \cdot M_{Ed} \cdot \gamma_{M1} \cdot (X_{LT} \cdot M_{y, Rk})^{-1} = 0.491 < 1 \quad (6)
$$

In the case of beams (without a haunch section) analytical relation (5) and (6) equal with: 0.468 respectively 0.26.

3.2 Geometrically nonlinear analysis
The following displacement values were obtained in the geometrically nonlinear analysis with global and local imperfections: $Dz = 182.9$ mm; $Dx = 26$ mm.

In the case of the column the values of the efforts are: $N_{Ed} = 159.86 \text{ KN}; V_{Ed} = 73.01 \text{ KN}; M_{Ed} = 484.7 \text{ KNm}$.

Figure 6. Vertical displacement diagram geometrically non-linear analysis with global and local imperfections in the arc $Dz = 182.9$ mm.
4. Conclusion

The use of modern calculation software that allows the activation of geometrical nonlinearity modules (Timoshenko) and initial imperfections simplify several calculations for the first method. The present study tries to show whether or not this stable analysis solution is justified.

The existence of global and local imperfections in the arc overlay with a non-linear geometric calculation does not bring major changes in terms of the efforts and displacements to which the design is made.

The realization of a classic design with resistance and stability checks for the individual elements is thus shown to be enough. I think it is more important that the optimization with the computing programs is doubled by the strength and stability checks performed manually. At the same time, the design of a seismically well structure, with correct knot connections introduced in the program and with the use of the super-resistance according to P100-1 are aspects that most often have a decisive influence on the structure’s response.

Finally, a non-linear geometrical calculation was made with the amplification of the snow loads, with increasing tracking according to the load (table 1). Obtaining a single matrix resulted in an increase of more than 4 snow loads. On the other hand, the design was achieved with a coefficient of interaction M-N of 0.491.

| Table 1. Displacements obtained for the amplification of the load. |
|---------------------------------------------------------------|
| Load coefficient | Dy [mm] | Dz [mm] |
| 1.0    | 27.4    | 182.9   |
| 1.5    | 29.1    | 252.2   |
| 2.0    | 30.7    | 323.1   |
| 2.5    | 32.3    | 395.8   |
| 3.0    | 38.5    | 470.5   |
| 3.5    | 44.8    | 547.8   |
| 4.0    | 50.7    | 628.0   |

References

[1] *** P100-1/2013 Design of structures for earthquake resistance, part 1
[2] *** SR EN 1993-1-1/2006 Eurocod 3 Design of steel structures. General rules and rules for buildings
[3] Dubina D and Ungureanu V 2010 Checking the stability of a steel element in accordance with SR EN 1993-1.1, Recommendations for calculating comments and application examples
[4] Dubina D and Dinu F 2010 The global structural design of the steel structures. Recommendations, comments and examples of application according to SR EN 1993-1-1 and SR EN 1998-1