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SECOND ORDER ANALYSIS OF THREE DIMENSIONAL STEEL INDUSTRIAL BUILDINGS

ANÁLISE DE SEGUNDA ORDEM DE EDIFICAÇÕES INDUSTRIAIS EM TRÊS DIMENSÕES

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ABSTRACT

Second order effects were historically included by the effective length method (K concept). All the studies about that methodology have been developed in frame plane, with regular rectangular frames. The new way to include those effects is the use of second order analysis, direct analysis method or alternative simplified options. This methodology was included in ANSI AISC 360 in the 13.0 version and in the 14.0 version. As before, the studies already developed for DAM analysis are in plane. In this paper, the K concept is revisited by numerical analysis, and extended to the 3D space. Using models of symmetric and non-symmetric industrial steel structures in plane, 3D stability analysis were developed, and the results were compared with plane behavior. Several conclusions and recommendations were exposed, resulting from analyzed models.
Keywords: second order analysis; steel structures; irregular 3D frames.

RESUMO

Os efeitos de segunda ordem foram inclusos historicamente pelo método do comprimento efetivo de flambagem (K). Todos as pesquisas sobre análise de segunda ordem têm sido desenvolvidas no plano e com pórticos regulares retangulares. As novas alternativas usam análise de segunda ordem pela análise direta e métodos simplificados. Essa metodologia foi incluída na norma Americana ANSI AISC 360 desde o ano 2005 e mantida na versão de 2010. Assim como com o conceito de comprimento efetivo, as pesquisas para análise direta de segunda ordem são quase todas em pórticos planos regulares. Neste artigo, o conceito de comprimento efetivo de flambagem é revisitado e estendido para o espaço 3D. Usando modelos não simétricos e tridimensionais de estruturas de aço industriais, análises de estabilidade elástica e de segunda ordem em 3D foram desenvolvidos e comparados com as pesquisas e seus resultados em pórticos planos. Destes estudos se obtém conclusões fundamentais para a análise tridimensional de estruturas de aço.
Palavras Chave: análise de segunda ordem; estruturas de aço; pórticos tridimensionais irregulares.

1 – INTRODUCTION

The evolution of structural engineering gives the opportunity to build more complex structures with irregular shapes and new structures with unknown behavior, as well as it brings up the necessity to study them. Complex shapes are not only used for buildings, they are also used for any kind of structures, as they could be industrial structures or bridges.

The 3D building projects with unusual geometry as a lack in research, specifically focusing on industrial structures with irregularities, from the geometry or the distributions of loads. This is mainly responsible for such bad conditions in design practice. Many questions concerning the behavior of these types of structures are still waiting for response. Some parts of stability analysis studies have not been deeply developed, and some design procedures should be necessary to evaluate new procedures for industrial steel structures.

A great effort in quantitative and qualitative research was spent in the last four decades about the nonlinear behavior of steel structures (CHEN and LUI, 1991; CHEN and TOMA, 1993).

Nonlinear seismic analyses of industrial steel structures with irregularities were studied with models with lower irregularities (CANELA, 2010), and showed that equivalent lateral force procedure should be studied for irregular structures, such as ours, in order to obtain more conclusions and maybe modify the code for the design. The whole procedure should also be revised in order to avoid or decrease large rotation on floors and torsion appearance in some members. Design process only considers earthquake in 0º and 90º direction. In a conventional way, where torsion is not conceived, it should be modified to consider it as long as it is created because of this irregularity.

1.1 Methods of Design

Until 2005, the only prescriptive or professional practical method was the effective length method, known as the K concept. It is based in stability analysis in plane, considering boundary conditions for isolated bars, or through stiffness of both edges of the bar. Also, in plane behavior, only applied to regular frame structures, i.e., same load applied in every column and column distance equivalent and other conditions that do not agree with real and common applications.

The effective length method generally provides a good design of framed structures. However, despite its popular use in the past and present as a basis for design, this approach has major limitations. Firstly, it does not provide an accurate indication of the factor against failure, because it does not consider the interaction of strength and stability between the member and structural system in a direct manner. It is a well-recognized fact that the actual failure mode of the structural system often does not have any resemblance whatsoever to the elastic buckling mode of the structural system that is the basis for the determination of the effective length factor K (KIM, PARK and SHOI, 2001).

In the same decades, several researches were carried about nonlinear analysis with the advance of computational capacity, turning more complex second order analysis feasible. The great change in AISC 360 (2005) was the inclusion of Direct Analysis Method (DAM) where K = 1.0, using imperfections or even substituting them by notional loads. Design method for stability that captures the effects of residual stresses and initial out-of-plumbness of frames by reducing stiffness and applying notional loads in a second order analysis (AISC 360, 2010). Again, simplified methods using linear analysis were introduced –First Order Method (FOM)– but only applied to frame plane structures. LeMessurier (1976 and 1977) exposed the simplified methods for the amplification method for pin jointed and rigid frame. In FOM, structural analysis in wich equilibrium conditions are formulated on the undeformed structure, this method neglected the second order effects (AISC 360, 2010).

There is not a single mention in AISC 360 (2005) or AISC 360 (2010) about indications on how to deal with sway in the space of non-symmetric frame structures.

A relevant problem of the conflict of FOM is when it is applied to pitched-roof frames; a problem with a solution (SILVESTRE and CAMOTIM, 2007): the first buckling mode is none sway mode, when the FOM is expected to have a sway buckling mode. In three dimensional with beams and columns, none disposed in orthogonal layout, the first or the first modes are non-sway, in general are torsional.

In Europe, the EC3-EN (2005) specify, for global analysis, how the effects of deformed geometry of structure should be taken in model to represent real structure. The standard defines a parameter α_{cr} (factor by which the design loading would have to be increased to cause elastic instability in a global mode). For elastic analysis, if the parameter is smaller than 10, it is necessary a second order analysis or the P-Δ effects must be taken in consideration.

$$\alpha_{cr} = \frac{F_{cr}}{F_{Ed}} \tag{1}$$

Where:
F_{cr} is the elastic critical buckling load for global instability, sway mode;
F_{Ed} is the design loading on the structure.

This last parameter has good behavior when the frame has main or first mode of buckling sway modes, but does not work in another kind of buckling mode.

1.2 State of the Art

A simplified model developed by Zubydan (2011) shows full behavior of steel plane frames with simplified model with stress distributions for a planar steel frame. However, the studied models were only in plane, without consider 3D behavior.

ThaI and Kim (2011) considers both geometric and material nonlinearities in a fiber beam-column element and using only one element per member in structure modeling with the stability functions derived from the exact stability solution of a beam-column subjected to axial force and bending when second order effects are investigated. The solution was compared with SAP2000 models results with focus in program developed where necessary more than one element per member for simulate the real columns behavior. The solution with non-linearity behavior is a result from a computer program developed by the authors, and shows the possibility to found the collapse operation load and operation load. (COSGUN and SAYIN, 2014) highlights the importance of the effect from geometrical variations on equations of equilibrium.

The influence of second order behavior in steel beam with new finite element model has been developed and presented for Castellazzi (2012). When element matrices incorporate the effects of eccentric and flexible connections without additional degrees of freedom, and shows the natural frequencies and mode shapes of steel framed structures. His results confirm that flexibility has the most effect on the lowest frequency, and when semi-rigid connections are considerably is necessary another experience, the study consider isolated elements for shown the influence of second order behavior. In the same direction, Black (2011) shows the coefficients for evaluating of second order behavior only in regular end planar steel frames, with horizontal load to simulate the behavior of structure.

1.3 Linear buckling analysis utility

Today, the majority of commercial or free software offers the option for a linear buckling analysis, but this qualitative alternative for the stability analysis is not used. As exposed in the previous item, with this analysis it is feasible to have parameter to take a decision on the use or not of the second order analysis, using the prescription of the EC3-EN (2005).

However, additional information can be result from the buckling modes. It is possible to improve the global stability trough the location of bracings, or even know how the structure will work, that is torsion, sway mode, or other alternatives. This is a matter of great importance because most of the modeling for second order analysis is done on drift displacement in two main directions and, if the main buckling mode is not of lateral behavior, what should be done?

2 – METHODOLOGY FOR 3D ANALYSIS

In this paper, the methodology to achieve 3D analysis is as follow:

(a) Buckling Analysis, verifying the modes of buckling behavior as input information to choose how to apply imperfections;
(b) Nonlinear analysis including imperfections, as notional loads or geometry change of nodes coordinates; with the objective to measure the amplification from static linear analysis to second order analysis;
(c) With the data produced in step (b) analyze the quality of results for global stability.

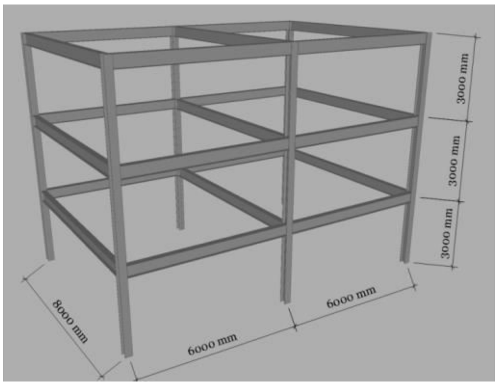
For the second order and buckling analysis, the commercial software used was SAP 2000 Version 16 (2013). Two models were studied, the first one with distribution of geometry and loads regular – regular frame. The second with geometry and loads distribution irregular.

3 – ILLUSTRATIVE EXAMPLES

In order to expose the main ideas behind the second order analysis, after the study of several models, two of them have been chosen: one framed and regular (RF), and a second one framed but irregular (IF) with notional loads applied as lateral load and the second step when loads are applied at all columns in each level (ERICKSEN, 2011).

The first structural model is represented in Figure 1, the beams are connected in columns with moment resistance connections, structure with three levels of 3.0 m each one, two 6.0 m between frames, and 8.0 m between columns of frame. The structures were analyzed as two-dimensional frames and three dimensional, with second order analysis (DAM), also using the First Order method, and the elastic buckling analysis.

Figure 1 – Regular frame model

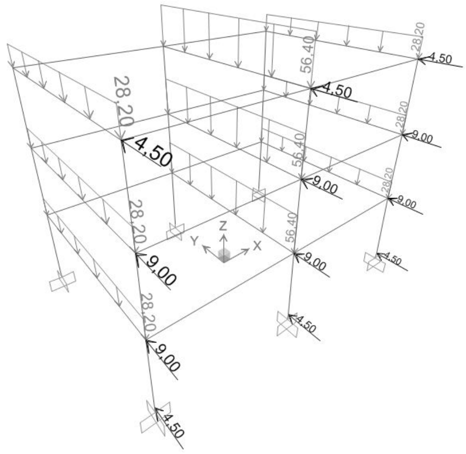


The RF show in Figure 1 is composite with welded steel profiles in columns WP 250·250·6.35·8.0 (mm) and beams WP 400·200·4.75·8.0 (mm). Selected steel is ASTM A572 Gr.50 for columns and beams (F_y = 345 MPa and E = 200 GPa).

In Figure 2, it is possible to see the model vertically loaded with lateral loads from wind. The results for the second order analysis (DAM) and FOM are presented in Table 1.

Nodal loads shows in Figure 2 are in kN and distributed load are shows in kN/m.

Figure 2 – Regular frame model with horizontal loads



It was observed that results from two-dimensional models are more conservative compared to the three dimensional analysis. In both cases, the First Order Method works well with some differences, not essentially. The critical factor 4.67 is below 10, meaning that the structure has some sources of nonlinearity and the first buckling mode is a sway mode.

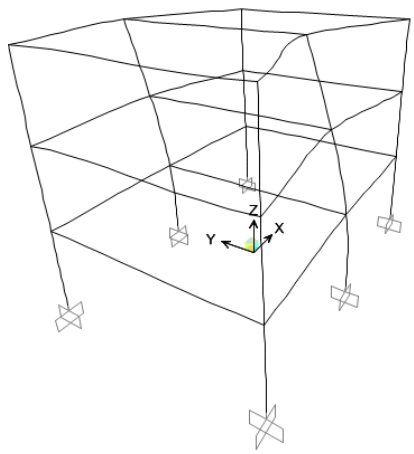
Table 1 – Results from RF model

Elevation (mm)	Model 2D		Model 3D	
	Δ_2*/Δ_1* (DAM)	FOM (B_2)	Δ_2/Δ_1 (DAM)	FOM (B_2)
3,000	1.08	1.04	1.06	1.07
6,000	1.08	1.04	1.09	1.07
9,000	1.09	1.03	1.08	1.06

*Δ_1: Drift for first order analysis (mm); * Δ_2: Drift for second order analysis with notional loads (mm);

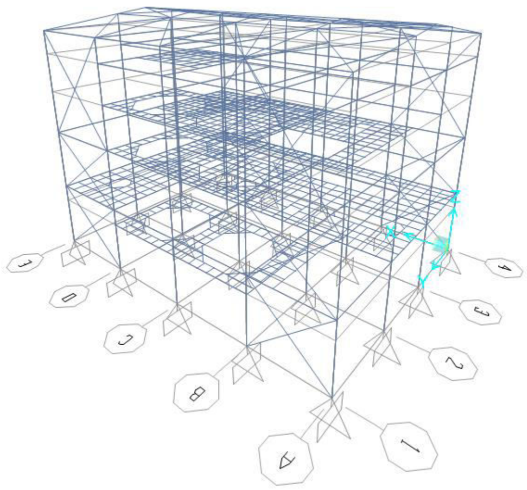
Figure 3 illustrate the First buckling mode in regular frame.

Figure 3 – First buckling mode with α_{cr} = 4.67



The second model is a real project of an industrial building, with irregular geometry in each elevation to locate equipment. In plane, the rectangular dimensions are 22.2 m wide and 29.9 m in length, column height of 20.8 m and outer ridge of the building, 22.7 m with 5 platform accesses for the use of the building, see Figure 4.

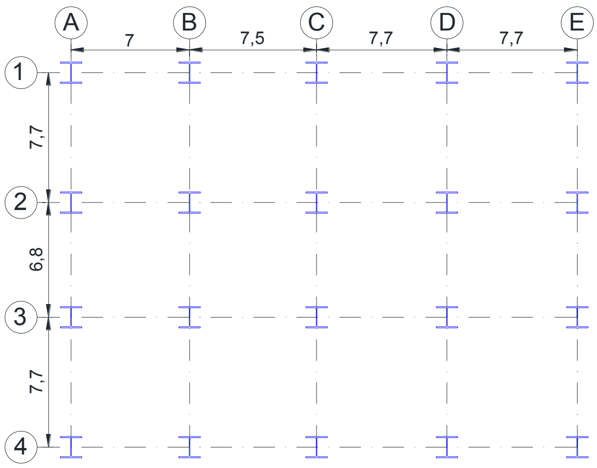
Figure 4 – Perspective of irregular building model



Is possible to see in Figure 5 the reference base system with references line of an industrial building, with irregular geometry.

Only in the thirteenth mode α (linear buckling) a sway mode appears, i.e., a mode with lateral displacement with the value of α = 12.49.

Figure 5 – Reference system of irregular building model (m)



Selected steel is ASTM A572 Gr.50 for columns and beams (F_y = 345 MPa and E = 200 GPa), for rolled profiles and welded profiles.

Figure 6 shows the sections used in axis 1, each level and location with specific equipment support.

Figure 6 – Elevation of axis 1 (profiles)

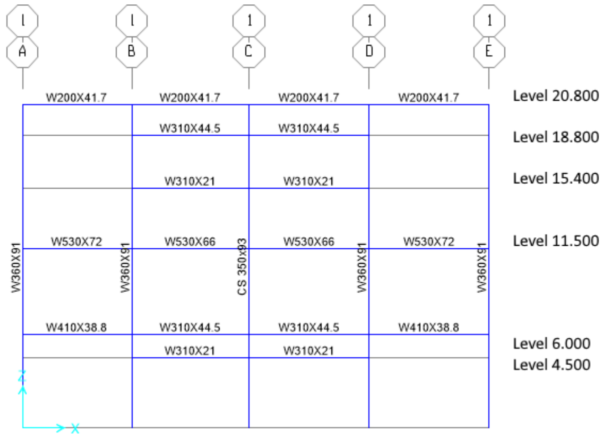


Figure 7 shows the profiles used in axis 3.

Figure 7 – Elevation of axis 3 (profiles)

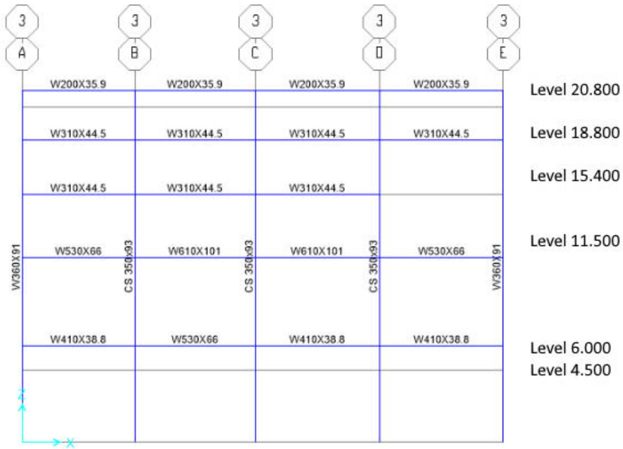


Figure 8 shows the profiles used in row A.

Figure 8 – Elevation of row A (profiles)

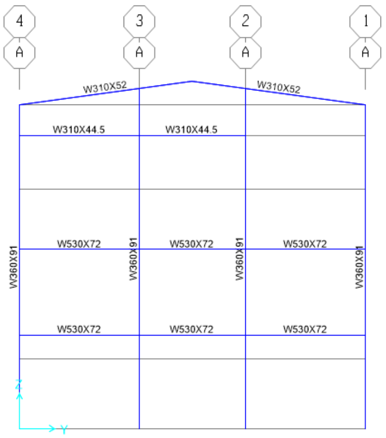


Figure 9 shows the profiles used in row D.

Figure 9 – Elevation of row D (profiles)

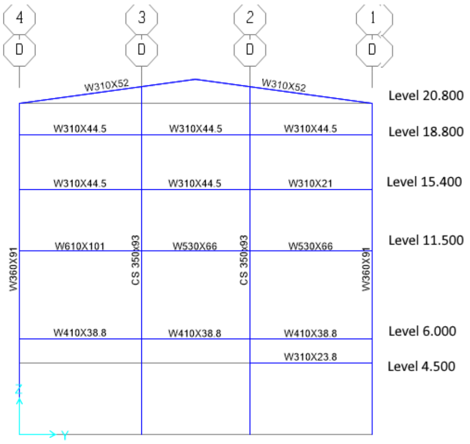
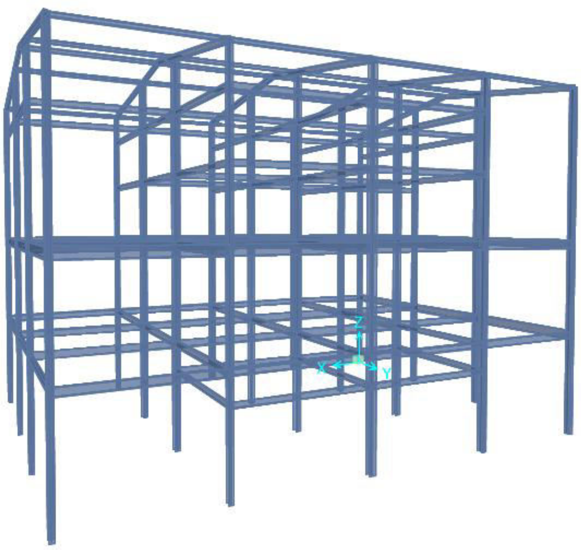


Figure 10 show the irregularities on structure, without symmetry on plane elevations, none also in the layout of columns and frame e the composition of the whole building structure.

Figure 10 – Elevation of row D (profiles)



According to EC3-EN (2005) it is not necessary to consider the second order effects, but if we use the concept of the relation of displacements of second to first order analysis, from AISC 360 (2010), is necessary to reduce stiffness and consider second order effects, according to the value of 2.45 (see Table 2, U_{x2}*/U_{x1}* for node 1311 is 2.45) at node 1308 at the elevation 18,800 mm, which is greater than 1.7 allowed by AISC 360 (2010), when the notional loads are applied as lateral loads.

Figure 11 shows notional loads applied only in one lateral of building for first and second order analysis.

Figure 11 – Notional loads applied as lateral loads (kN)

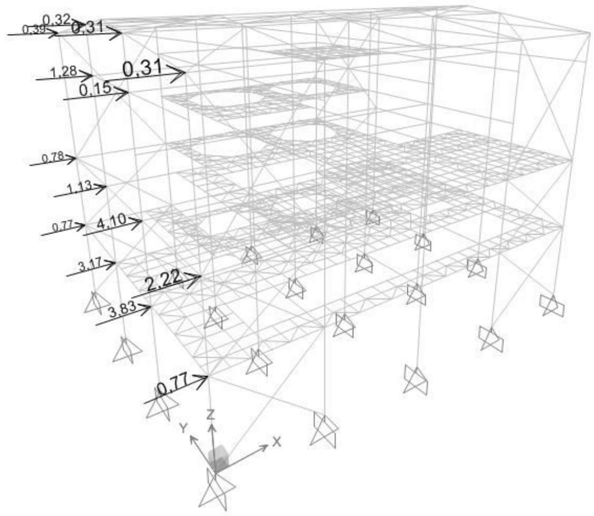


Figure 12 shows notional loads applied in all column nodes for first and second order analysis.

Figure 12 – Notional loads distributed over the nodes and over the levels (kN)

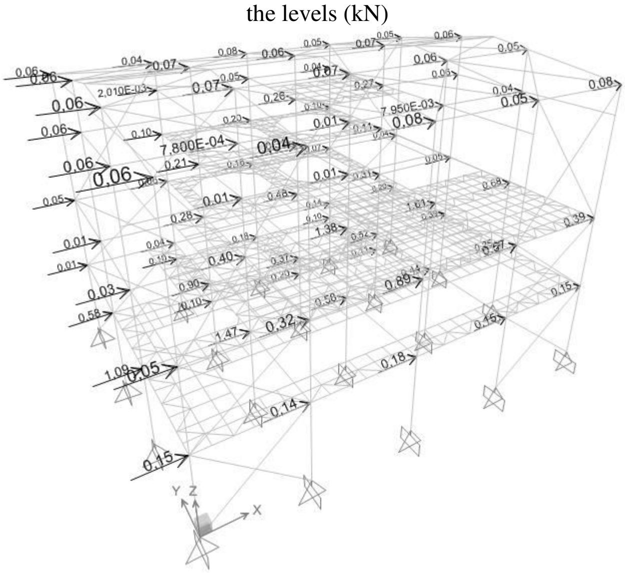
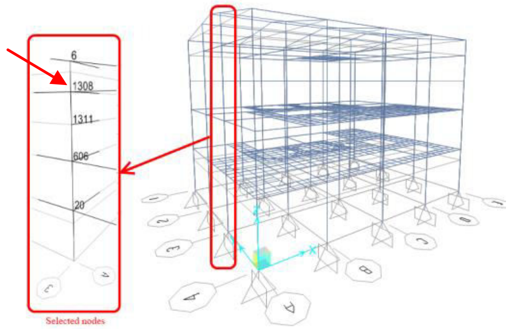


Table 2 shows results from relation between first and second order analysis (Δ2 (mm) /Δ1 (mm)), necessary to define the horizontal sensibility of structure. The nodes chosen for the study are the nodes of the intersection of line 3 column with row A, every node wit horizontal beam, see indication on Table 2.

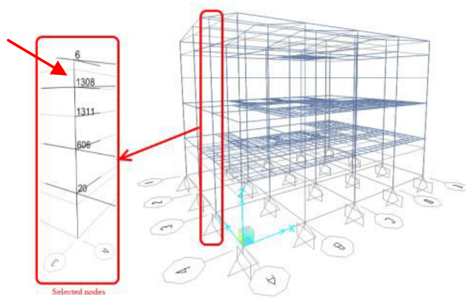
Table 2 – Displacements of first and second order analyses notional load applied as lateral load (Figure 5: U_{x2} is the second order absolute story displacement; U_{x2}* is the second order relative story displacement)



Node	Elev.	U_{x1}	U_{x2}	U_{x1}*	U_{x2}*	U_{x2}*/U_{x1}*
	mm	mm	Mm	mm	mm	
20	6,000	1.38	1.84	1.38	1.84	1.33
606	11,500	1.76	2.24	0.37	0.39	1.06
1311	15,400	1.74	2.20	−0.01	−0.03	2.45
1308	18,800	1.62	2.07	−0.12	−0.13	1.06
6	21,840	1.41	1.86	−0.20	−0.21	1.02

Table 3 shows results from relation first and second order analysis, necessary to define the horizontal sensibility of structure.

Table 3 – Displacements of first and second order analysis notional load applied to all columns in each level (see Figure 11: U_{x2} is the second order absolute story displacement; U_{x2}* is the second order relative story displacement)



Node	Elev.	U_{x1}	U_{x2}	U_{x1}*	U_{x2}*	U_{x2}*/U_{x1}*
	mm	mm	mm	mm	mm	
20	6,000	1.35	1.81	1.35	1.81	1.34
606	11,500	1.73	2.21	0.38	0.41	1.07
1311	15,400	1.76	2.23	0.03	0.01	0.39
1308	18,800	1.62	2.08	−0.14	−0.14	1.06
6	21,840	1.40	1.85	−0.23	−0.23	1.02

When loads applied on all the columns in each level (Figure 11), in the same node, the relation of second order to first order is less than one, with no meaning at all. In all the other levels, the Table 2 and 3 are similar.

In the model with irregularities, when existing a member with one free extreme, convergence was not achieved in the second order analysis. To overcome this situation, secondary members were retired from the model.

The only way to deal with unsymmetrical frames or with irregularities in their layout is using both second order analysis and linear buckling analysis, to know how the whole structure works.

CONCLUSIONS

The methods proposed in the standards for steel framed structures are based in regular framed models for drift behavior or lateral displacement (sway). When the analyzed building is irregular in geometry or load layout, the methods have some lacks. The alternative is using the second order analysis with elastic buckling to have qualified information about the behavior of the system.

The correct use of notional loads in all columns and not only in the external elevations, generate more approximate values of the calculated stresses in the analysis, allowing a project with conditions to meet the needs of building use.

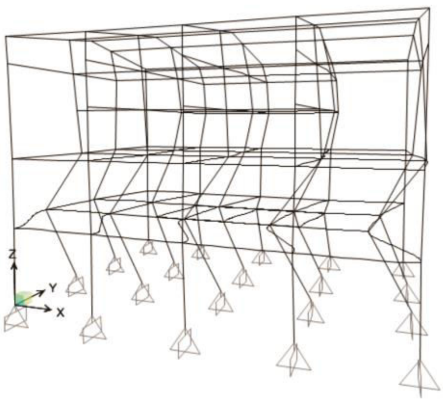
Real project does not have symmetry in planes XY or XZ or YZ, and the distances between columns are variable, then vertical loads acting in every column as different behavior and different contribution to the stability of the whole structure.

To deal with the project of three dimensional buildings, with irregular distribution of geometry or loads or both, is necessary first to develop linear stability analysis to see the behavior or the system, and decide how to dispose the imperfections as geometry displacements or equivalent forces.

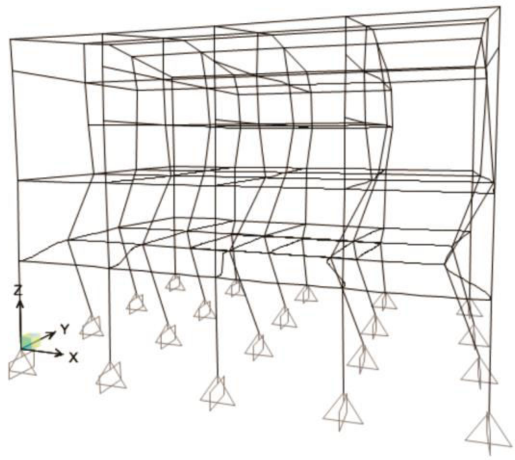
Is necessary to research in the size of the imperfections already defined in standards and its importance in the stability and strength of steel 3D building.

In same building showed in Figure 4, when we check the linear buckling modes, is possible see the differences modes, each one with your factor by which the design loading would have to be increased to cause elastic instability in a global mode, see Figure 13 the first four linear buckling modes.

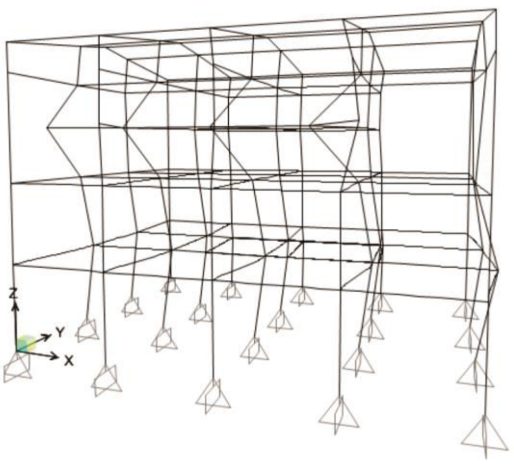
Figure 13 – First four buckling modes for IF model



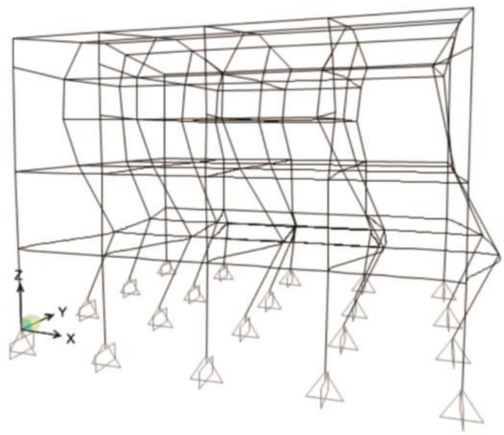
(a) $\alpha_1 = 6.59$



(b) $\alpha_2 = 7.60$



(c) $\alpha_3 = 8.36$



(d) $\alpha_4 = 8.54$

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