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Influence of connection stiffness on static-dynamic behavior of steel frames

Emerson Alexandro Bolandim[1], Renato Silva Nicoletti[2*], Alexandre Rossi[3] and Alex Sander Clemente de Souza[2]

[1]Departamento de Engenharia de Estruturas, Universidade de São Paulo, São Carlos, São Paulo, Brasil. [2]Departamento de Engenharia Civil, Universidade Federal de São Carlos, Rod. Washington Luiz, s/n, 13565-905, Monjolinho, São Carlos, São Paulo, Brasil. [3]Faculdade de Engenharia Civil, Universidade Federal de Uberlândia, Uberlândia, Minas Gerais, Brasil. *Author for correspondence. E-mail: renato_nicoletti@hotmail.com

ABSTRACT. The growing problems of excessive vibration in steel structures due to human rhythmic activities and vibrating machinery have led to the need for increasingly rigorous dynamic analysis in order to verify excessive displacements and ensure the users' comfort. The present work aimed to carry out, via numerical simulation in ANSYS Mechanical APDL software, static, modal and harmonic analysis in steel frames with two floors and two bays in order to investigate the influence of the connection's stiffness on the behavior of the structure. The results obtained showed that the natural frequency value of the models increases as the stiffness of the beam-column connections is increased. However, due to the harmonic analysis performed, it was not possible to delineate a behavior between the stiffness of the connections and the amplification.

Keywords: Steel structures; semi-rigid connections; dynamic behavior; harmonic analysis; modal analysis.

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Introduction

Multi-storey steel buildings are increasingly gaining ground in the civil construction sector, as they are economical and safe solutions for the most varied types of projects and structures. In general, steel structures are slim, efficient and quick to erect. They use bracing systems, rigid frames, trusses, concrete cores or a combination of these to resist the effects of destabilizing horizontal forces and second order effects (Figure 1). The other structural elements in steel, columns and beams, called braced elements, are considered secondary elements, with their labeled connections, as they are not responsible for the stability of the structure (Fakury, Castro, & Caldas, 2016; Salmon, Johnson, & Malhas, 2009; Trahair, Bradford, Nethercot, & Gardner, 2008).

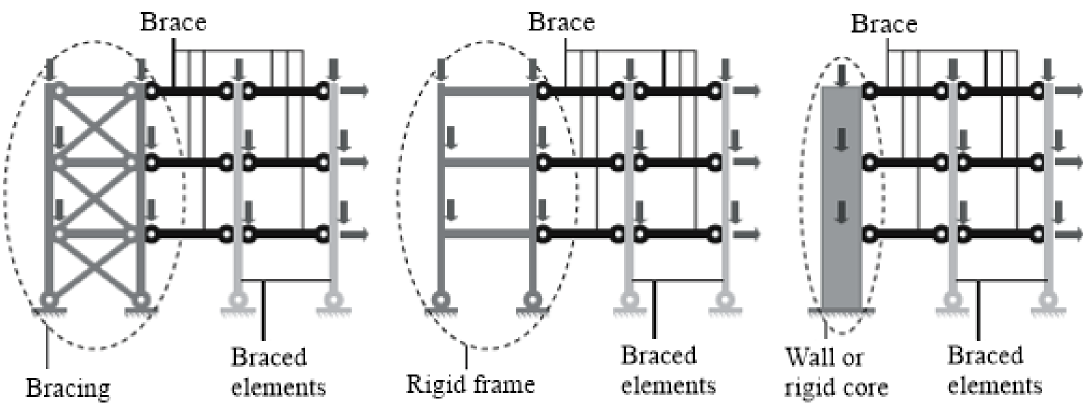

Figure 1. Bracing systems to the horizontal actions and 2nd order effects in steel structures. Adapted from Fakury et al. (2016).

Since the steel construction system is reticulated, it is necessary to use connections between the structural elements. Regarding connections between structural elements, Eurocode 3, Part 1.8 (European Committee for Standardization [CEN] EN 1993-1-1, 2005) and the AISC S360 (American Institute of Steel Construction [AISC], 2016) standard classify connections between steel elements as flexible, semi-rigid and rigid in structural analysis. In the flexible connection model, it is assumed that the connection does not transmit bending moment. In the rigid connection model, there is a total transfer of bending moment. The semi-rigid connection is an intermediate case among those mentioned. The three types of connection models described are classified based on two independent criteria expressed in terms of rotational stiffness and strength, respectively. (AISC, 2016; Bjorhovde, Colson, & Brozzetti, 1990; CEN EN 1993-1-1, 2005; Chen & Kishi, 1989; Chen, Kishi, & Komuro, 2011).

In fact, the connections between beams and columns are classified between two extreme cases: rigid and ideally flexible. Thus, numerical modeling of connections as semi-rigid is the most realistic approach. However, in engineering practice, the consideration of connections as flexible or not depends on their strength and stiffness, so when those connections are unable to carry significant bending moment, they are considered as flexible, allowing an almost free rotation. Likewise, some connections are considered rigid if their stiffness is large enough so that no significant discontinuities exist between adjacent elements. In this way, the consideration of ideally flexible or rigid connections considerably simplifies the design and analysis procedures of reinforced steel structures (Chen et al., 2011).

Furthermore, steel structures are slender and require careful structural analysis in order to ensure a stable, safe and efficient building throughout its lifetime. Regarding the dynamic behavior, to achieve a satisfactory vibration behavior of buildings and their structural elements in service conditions, it is necessary to consider the user's comfort; and the functioning of the structure or its structural elements (for example, cracks in walls, damage to the coating, rupture of facade elements, and others).

In order that the serviceability limit state of a structure or structural element is not exceeded when subjected to vibrations, the natural frequency of the structure or structural element must be maintained above appropriate values, which depend on the function of the building and the source of the vibration. If the natural frequency of the structure is less than a certain appropriate value, a more refined analysis of the structure's dynamic response is necessary, including consideration of damping.

Possible sources of vibration in this type of analysis include walking, synchronized movements of people (dancing for example), machines and wind actions. In practice, for small and medium-sized buildings, for commercial or residential purposes, the main issue is the dynamic response of the system to human excitation, mainly due to walking (footfall effect). (Costa-Neves, Silva, Lima, & Jordão, 2014; Silva, Andrade, & Lopes, 2014).

Specifically, there are some situations in which steel structures are subject to a level of dynamic loading that causes permanent deformations and localized structural damage, which can lead to the partial or total collapse of the system. Although the design of these structures in elastic approach avoids this behavior tends to be uneconomical, is a good resource from a dynamic point of view in evaluating possible problems related to unwanted vibrations, excessive displacements and still ensuring an acceptable safety for buildings and comfort for users.

In the literature, there are several works studying dynamic effects in steel structures. In general, such research can be divided into the study of building vibrations globally (Housner & Brady, 1963; Satake & Yokota, 1996; Kohler; Davis, & Safak, 2005; Kazantzi & Vamvatsikos, 2020) and in the study of its parts, such as floors and connections (Chan, 1994; Chan & Ho, 1994; Chui & Chan, 1997; Tremblay, 2005; Culau, Quintas, & Gomes, 2018; Ribeiro, Gomes, Ferreira, & Calenzani, 2018). In particular, regarding this work, semi-rigid connections are widely studied, but there are few studies on the effect of semi-rigidity on structural behavior and especially on vibrations.

Considering the importance of understanding the dynamic behavior of the structure, this paper performs, via numerical simulation in ANSYS Mechanical APDL software, static, modal and harmonic analysis in two-story steel frames in order to investigate the influence of the connection’s stiffness in the structure behavior.

Material and methods

Types of dynamic analysis

Modal analysis

Modal analysis determines the natural frequencies and vibration modes of a structure. The analysis uses entire mass and stiffness of the structure determining the various frequencies at which the structure will naturally resonate. Both parameters (frequency and vibration mode) are important in the design of a structure under dynamic loading conditions and are necessary for a harmonic, transient and spectral analysis.

Harmonic analysis

Harmonic analysis evaluates the steady state behavior of a structure subject to the action of cyclic loads. It calculates the structure's response to a load (or loads, since they have the same frequency) that has a sinusoidal (harmonic) time history, such as an unbalanced electric motor. Figure 2 illustrates this phenomenon, where $F=F(t)$ is the force as a function of time that represents the harmonic excitation; F_0 is the amplitude of statically applied force; ω the force excitation frequency; t the time; φ the phase angle measured in relation to the time reference; u the displacement as a function of time; and u_0 the initial displacement.

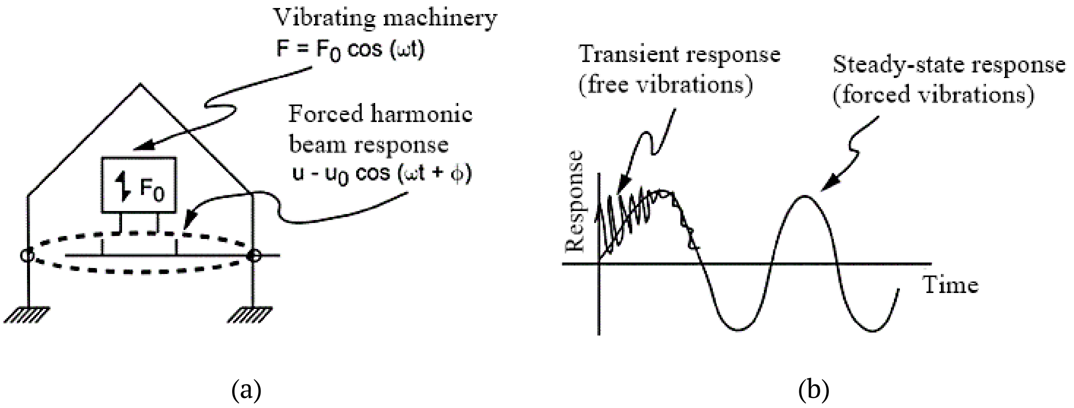

Figure 2. Typical simple harmonic system (a) F_0 and ω known, u_0 and φ unknown (b) Transient and stationary response of a structural system. ANSYS 18.2.

Since the analyzed structure has different frequencies depending on the different vibration modes (determined through a modal analysis), the harmonic analysis indicates the frequencies of the structure that will be amplified with the frequency of the harmonic load. With such information, it is possible to compare, through graphs (usually of displacements), the response peaks and then analyze the displacements and stresses for the peak frequencies.

Transient analysis

Transient analysis, also known as time-history, determine the dynamic response of a structure under time-dependent loads (no periodic frequency is considered in this type of analysis). This type of analysis determines the displacements, stresses, strains, and time-varying forces in a structure, as it responds to any combination of static, transient, and harmonic loads. The loading time scale is such that inertia or damping effects are considered important. Equation (1) represents the system of equations to be solved in this type of analysis.

$$[M]\{\ddot{u}\}+[C]\{\dot{u}\}+[K]\{u\}=\{F(t)\} \tag{1}$$

Where $[M]$ is the mass matrix; $[C]$ is the damping matrix; $[K]$ is the stiffness matrix; $\{\ddot{u}\}$ is the nodal acceleration vector; $\{\dot{u}\}$ is the nodal velocity vector; $\{u\}$ is the nodal displacement vector; and $\{F(t)\}$ is the forces vector. It is noteworthy that the mass, damping and stiffness matrices do not vary with time. The only parameter that varies with time is the nodal displacement.

Spectral analysis

Spectral analysis is used to study the response of the structure under the action of loads with known frequency ‘spectra’ (structural dynamic loading conditions, for example).

Computational numerical model

In order to investigate the influence of connections on the dynamic behavior of the structure, a steel frame with five different stiffness values in its connections was modeled using the finite element software ANSYS Mechanical APDL. Figure 3 presents a scheme of the structure whose frame is inserted.

The beams and columns were pre-dimensioned and consist of hot-rolled “I” profiles, with sections W410x38.8, W410x53.0, HP250x62.0 and HP250x85.0 in ASTM A-572 Gr.50 steel, according to the table of profiles from Gerdau Açominas. The position of each profile can be seen in Figure 4.

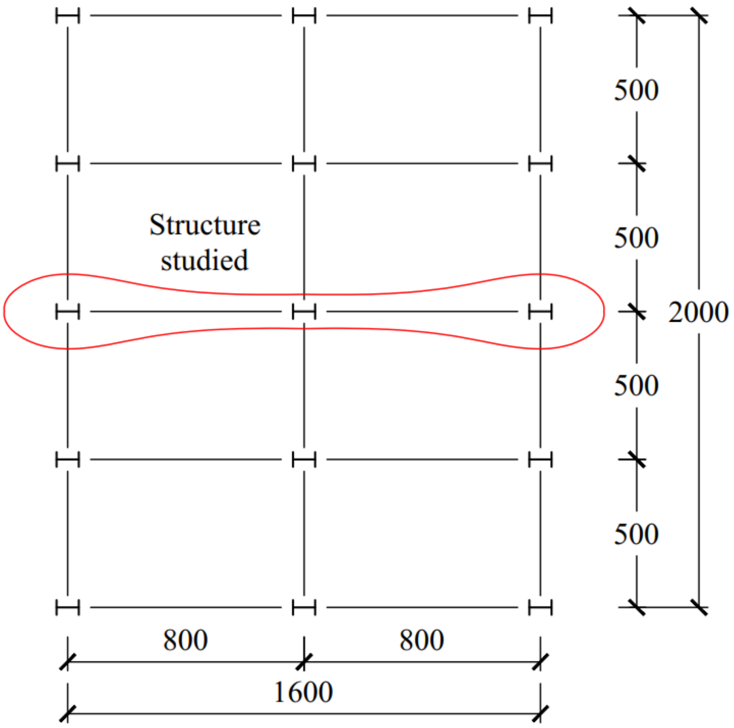

Figure 3. Structural scheme with identification of the studied frame. Dimensions in cm.

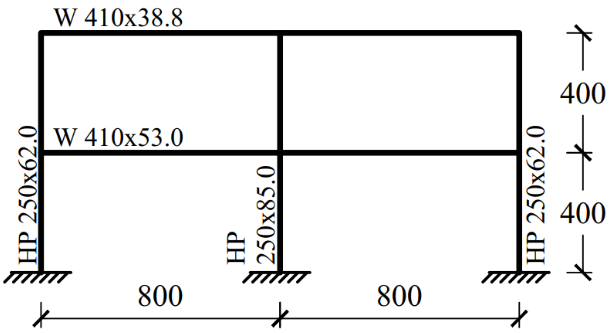

Figure 4. Structure studied with profile identification. Dimensions in cm.

Regarding the rigidity of beam-column connections, NBR 8800 (Associação Brasileira de Normas Técnicas [ABNT], 2008), in its item 6.1.2, defines that the connection can be considered flexible if it meets the Equation (2) condition, and rigid if it meets the Equation (3) condition, where S_i is the connection stiffness; E the modulus of elasticity of the material of the beams and columns; and I_v and L_v the moment of inertia of the cross section in the structure's plan and the beam spam linked to the connection, respectively.

$$S_i \leq \frac{0{,}5 \cdot E \cdot I_v}{L_v} \tag{2}$$

$$S_i \geqslant \frac{25 \cdot E \cdot I_v}{L_v} \tag{3}$$

Considering E = 200,000 MPa, Table 1 presents the limit stiffness for which the connections of the 1st and 2nd floor of the frame in Figure 4 are considered, respectively, flexible and rigid. Figure 5 illustrates the position of such connections.

Table 1. Stiffness limits to classify frame connections as flexible and rigid.

Floor	Connections stiffness [kN m^{-1}]	
	Flexible connection	Rigid connection
1° (Tipo 1)	< 2341.75	> 117087.50
2° (Tipo 2)	< 1597.13	79586.25

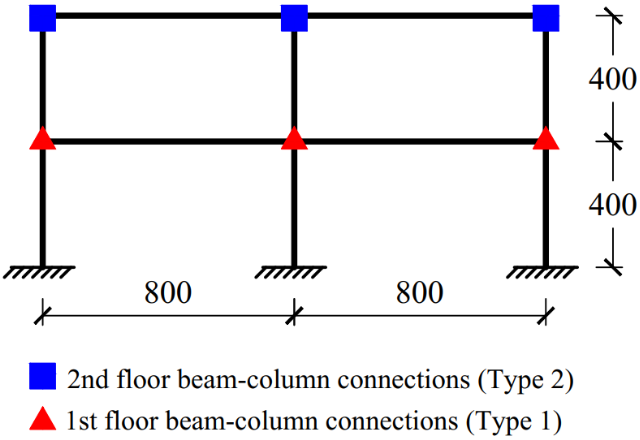

Figure 5. Position of beam-column connections whose rigidity will be varied. Dimensions in cm.

In view of this, as already mentioned, in order to investigate the influence of the stiffness of the connections in static, modal and harmonic analyses, five models were studied, with uniform variation in the stiffness of the beam-column connections, within the Table 1 limits. The stiffness considered in the models is presented in Table 2.

Table 2. Rigidity of the beam-column connections of the studied models.

Model	Connections stiffness [kN m^{-1}]		Connection classification
	1st floor	2nd floor	
1	2341.75	1597.13	Flexible/Semi-rigid*
2	31028.19	21161.91	Semi-rigid
3	59714.63	40726.69	Semi-rigid
4	88401.06	60291.47	Semi-rigid
5	117087.50	79856.25	Semi-rigid/Rigid**

*Connections classified as flexible and/or semi-rigid. **Connections classified as semi-rigid and/or rigid.

With the variations shown in Table 2, it was possible to perform analysis covering a flexible connection case, three semi-rigid connections, and a rigid connection.

Static analysis

Static analysis was performed in order to determine the displacements in the structure. Specifically, vertical and horizontal analysis were performed. For this reason, a load of 20 kN acting in the middle of the span of one of the beams was considered to analyze the dynamic behavior of the structure against vertical stresses. In turn, in order to analyze the gantry subjected to lateral actions, the wind acting on the gantry was calculated considering the arrangement in Figure 3. In this step, was considered: the basic wind speed V_0 = 40 m/s; topographic factor S_1 = 1,00; statistical factor S_3 = 1.00; and high turbulence wind. It is noteworthy that high turbulence wind was used, because, for the height of the analyzed structure, it would hardly exceed twice the average height of the buildings in the interior. Thus, admitting high turbulence wind is in accordance with item 6.5.3 of NBR 6123 (Associação Brasileira de Normas Técnicas [ABNT], 1988). Figures 6 and 7 show the loads considered for the vertical and horizontal analysis, respectively.

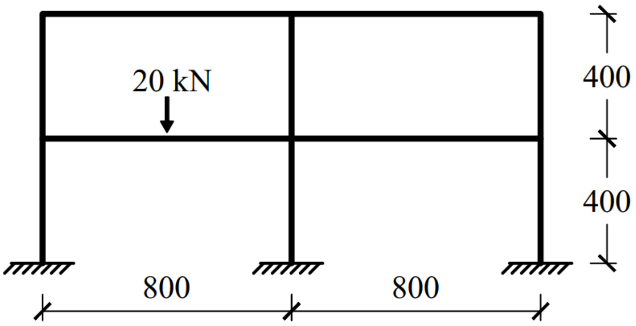

Figure 6. Loads considered for vertical structural analysis. Dimensions in cm.

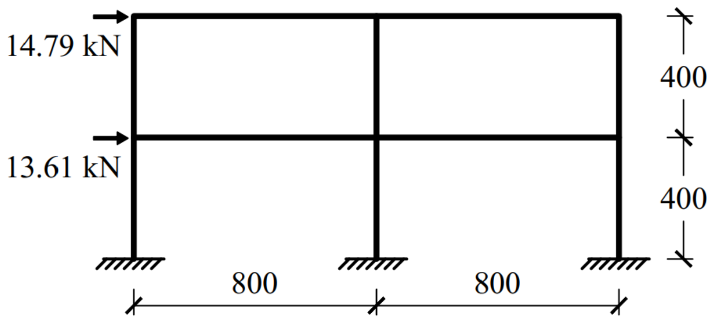

Figure 7. Loads considered for the horizontal structural analysis. Dimensions in cm.

Modal Analysis

The modal analysis was conducted in order to determine the free vibration modes of the structure and their respective frequencies. In this context, it is important to define that such frequencies indicate the rate of free oscillation of the structure after the force that caused its movement has ceased; while vibration modes are how the structure vibrates, related to each of its frequencies. Both the frequency and the vibration modes are calculated considering the already discretized and undamped structure through Equation (4).

$$\left(K - \omega_n^2 \cdot M\right) \cdot \{\Phi\} \tag{4}$$

Equation (4) consists of a typical problem of eigenvalues and eigenvectors, which *[K]* is the known stiffness matrix of the structure; *[M]* is the known structure mass matrix; ω_n^2 are the eigenvalues, squares of natural frequencies, positive real numbers to be calculated through numerical analysis; and $\{\Phi\}$ are the eigenvectors, vibration modes, vectors to be calculated through numerical analysis.

Harmonic analysis

In order to analyze the steady state behavior of the frames against cyclic loads, the harmonic analysis was performed in the vertical and horizontal directions, in which sinusoidal forces were considered according to Equation (5). The magnitude (F_0) and the position of the forces were assumed to be the same as in the static analysis (section 3.1).

$$F(t) = F_0 \cdot sen(\omega \cdot t + \varphi) \tag{5}$$

Where ω is the force excitation frequency, given in rad/s, t is the time, in seconds, and φ is the phase angle. The excitation frequencies were evaluated within a range that considered up to the thirtieth natural vibration frequency (ω_n) of the structure.

Finite elements used

The numerical modeling of steel beams and columns was performed using three-dimensional finite elements of the BEAM188 type (Figure 8), based on the Euler-Bernoulli formulation to determine the loads, displacements, natural frequencies and free and forced vibration modes of the system. This element has six degrees of freedom per node, being three translations (UX, UY, UZ) and three rotations (RotX, RotY, RotZ).

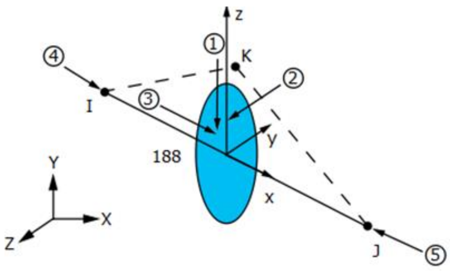

Figure 8. Beam188 Element. ANSYS 18.2.

To simulate the flexible connections, the numerical modeling was performed using the concept of nodal superposition, with one node belonging to the beam and the other belonging to the column, subsequently performing the coupling of the degrees of freedom of interest by means of sub-routines developed in ANSYS Mechanical APDL.

In turn, to simulate the connections semi-rigidity, the COMBIN14 spring element was used, which allows longitudinal or torsional applications in 1-D, 2-D or 3-D. The torsional option of the spring-cushion system is a purely rotational element with three degrees of freedom at each node: rotations about the x, y and z nodal axes (RotX, RotY, RotZ). Flexural or axial loads are not considered. The geometry, node positions, and coordinate system for this element are shown in Figure 9. The element is defined by two nodes, a spring constant (k) and damping coefficients $(cv)_1$ and $(cv)_2$. The values adopted for k are presented in Table 2. In turn, $(cv)_1$ and $(cv)_2$ are damping constants. In the present work, non-damped analyzes were performed. So $(cv)_1$ and $(cv)_2$ were equal to zero.

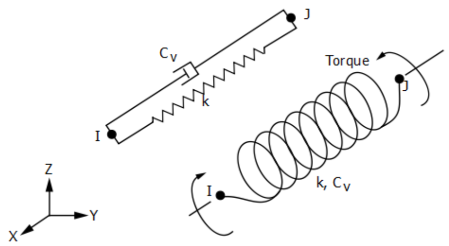

Figure 9. Combin14 element. ANSYS 18.2.

Results and discussion

Static analysis

Figures 10 and 11 show the vertical displacement and the bending moment diagram for Models 1 and 5, respectively.

From Figures 10 and 11, it is possible to notice that the effect of the connection's stiffness was properly modeled, since in the case of flexible connections (model 1), there was no transfer of bending moments from the beam to the column. However, in model 5, which has rigid connections, it is possible to observe bending moments being transferred from the beams to the columns.

Figures 12-a and 12-b show the maximum horizontal and vertical displacement, respectively, for the analyzed models.

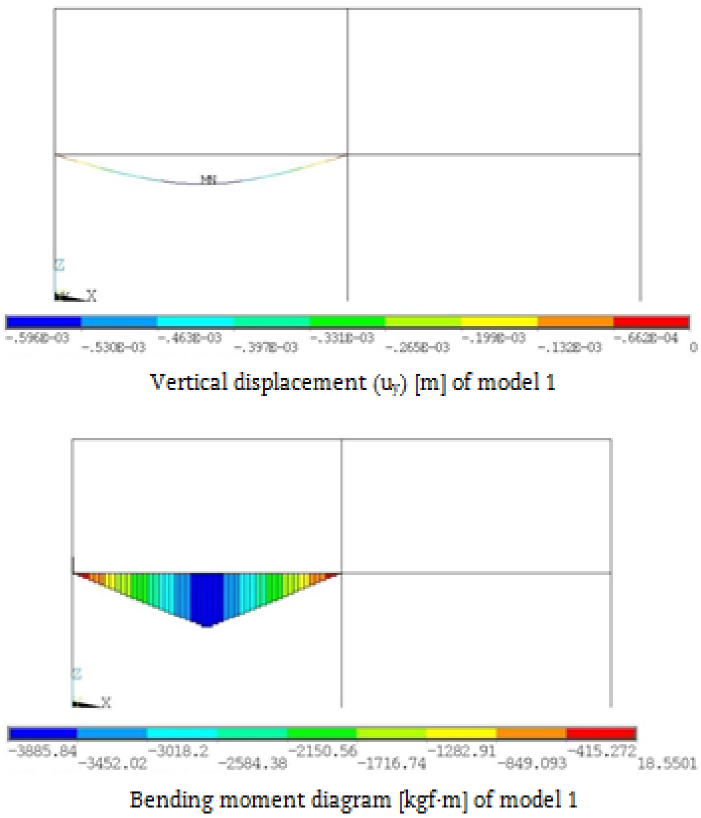

Figure 10. Vertical displacement and bending moment diagram of model 1.

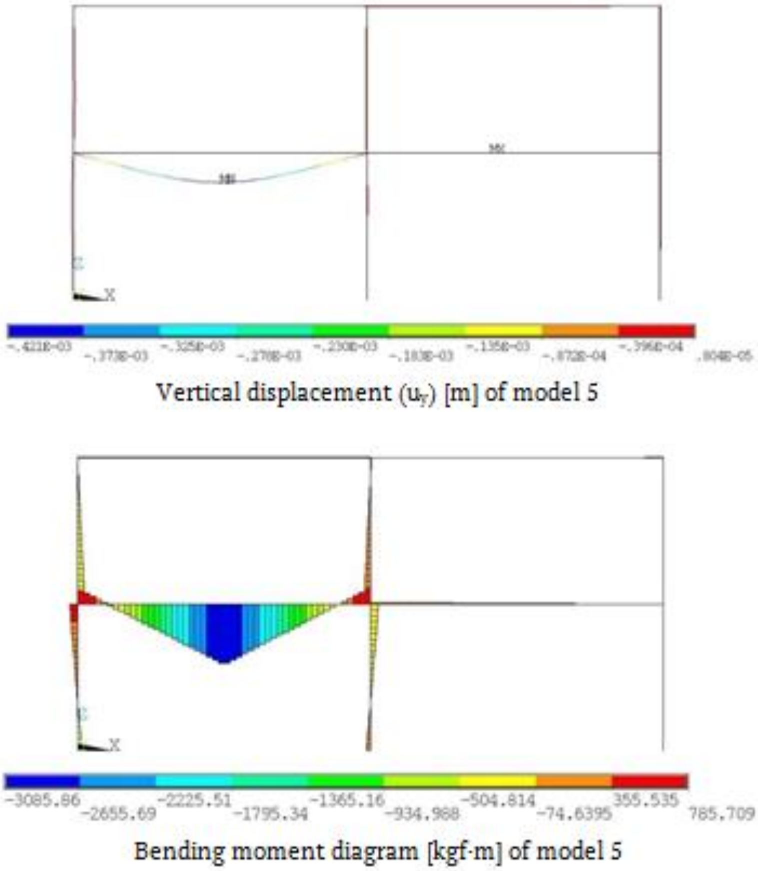

Figure 11. Vertical displacement and bending moment diagram of model 5.

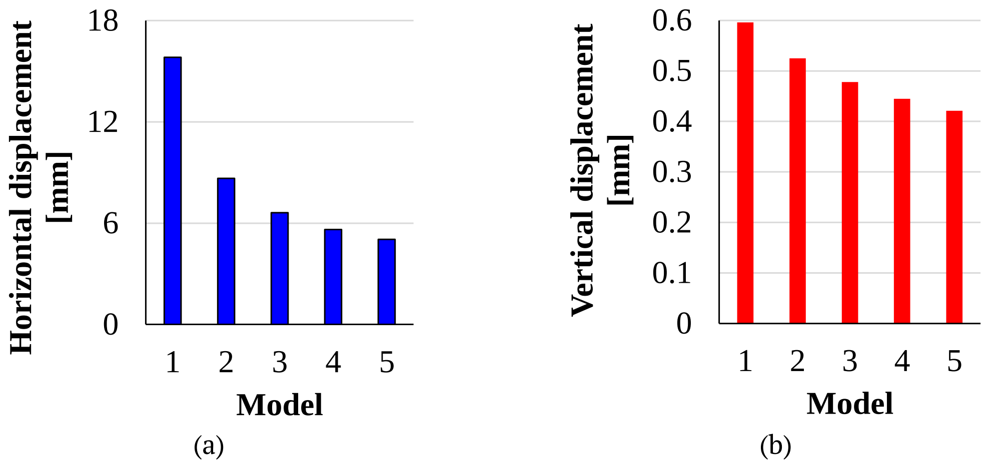

Figure 12. Maximum displacements (a) vertical and (b) horizontal in the analyzed models.

In general, and as expected, it is observed that the more rigid the connections, the smaller the displacements, given that when the connections rigidity increases, the rigidity of the frame also increases.

Modal analysis

Table 3 shows the frequencies of the first 30 vibrating modes of models 1 to 5. It also shows the standard deviation (SD) between the models for each vibration mode and the coefficient of variation (CV), calculated by the quotient between the deviation standard and the average.

In general, all vibration modes are bending in the plane of the frame. Considering only the vibration modes whose CV was above 1%, the frequencies of the models were compared and are shown in Figure 13.

From Figure 13, it is possible to see that the greater the connection stiffness, the greater is its natural frequency of vibration in all modes of vibration. Between model 1 and model 5, the greatest discrepancies occurred in the 1st and 22nd vibration modes, with variations of 74.30% and 20.72%, respectively. Figure 14 presents such vibration modes. It can be observed that, when changing the stiffness of the connections, the deformed configuration of the models changed to the vibration mode 22, corroborating the strong influence of the stiffness of the connections on the static-dynamic behavior

Table 3. Frequencies of the first 30 vibration modes of the models.

Vibration mode	Frequency [Hz]					SD	CV
	Model 1	Model 2	Model 3	Model 4	Model 5	[Hz]	[%]
1	2.930	3.935	4.481	4.843	5.107	0.863	20.27%
2	5.423	5.423	5.423	5.423	5.423	0.000	0.00%
3	5.423	5.423	5.423	5.423	5.423	0.000	0.00%
4	6.422	6.422	6.422	6.422	6.422	0.000	0.00%
5	6.422	6.422	6.422	6.422	6.422	0.000	0.00%
6	10.852	10.852	10.852	10.852	10.852	0.000	0.00%
7	10.852	10.852	10.852	10.852	10.852	0.000	0.00%
8	12.850	12.850	12.850	12.850	12.850	0.000	0.00%
9	12.850	12.850	12.850	12.850	12.850	0.000	0.00%
10	16.037	16.291	16.291	16.291	16.291	0.114	0.70%
11	16.291	16.292	16.292	16.292	16.292	0.000	0.00%
12	16.292	17.119	17.793	18.254	18.590	0.920	5.23%
13	19.291	19.291	19.291	19.291	19.291	0.000	0.00%
14	19.291	19.291	19.291	19.291	19.291	0.000	0.00%
15	19.497	20.002	20.002	20.002	20.002	0.226	1.13%
16	19.571	20.002	20.002	20.002	20.002	0.193	0.97%
17	20.002	20.018	20.018	20.018	20.018	0.007	0.04%
18	20.002	20.018	20.018	20.018	20.018	0.007	0.04%
19	20.018	21.357	21.747	21.747	21.747	0.749	3.51%
20	20.018	21.440	21.747	21.747	21.747	0.751	3.52%
21	20.214	21.670	22.425	23.119	23.621	1.336	6.02%
22	20.259	21.694	22.793	23.711	24.457	1.660	7.35%
23	21.747	21.747	22.853	23.868	24.700	1.304	5.67%
24	21.747	21.747	22.909	24.003	24.917	1.397	6.06%
25	25.750	25.750	25.750	25.750	25.750	0.000	0.00%
26	25.751	25.751	25.751	25.751	25.751	0.000	0.00%
27	26.676	26.676	26.676	26.676	26.676	0.000	0.00%
28	26.702	26.702	26.702	26.702	26.702	0.000	0.00%
29	27.224	27.224	27.224	27.224	27.224	0.000	0.00%
30	27.225	27.225	27.225	27.225	27.225	0.000	0.00%

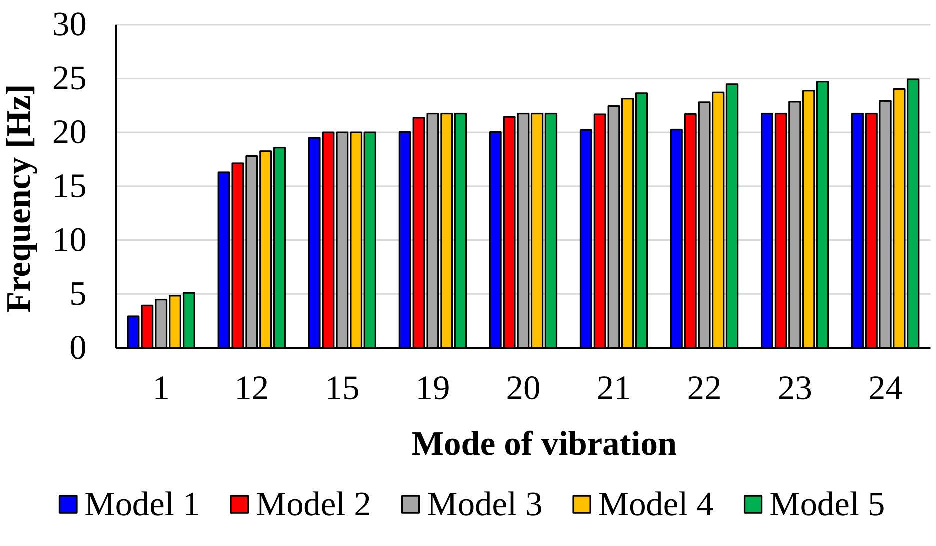

Figure 13. Comparison between the frequency of models for vibration modes with CV > 1%.

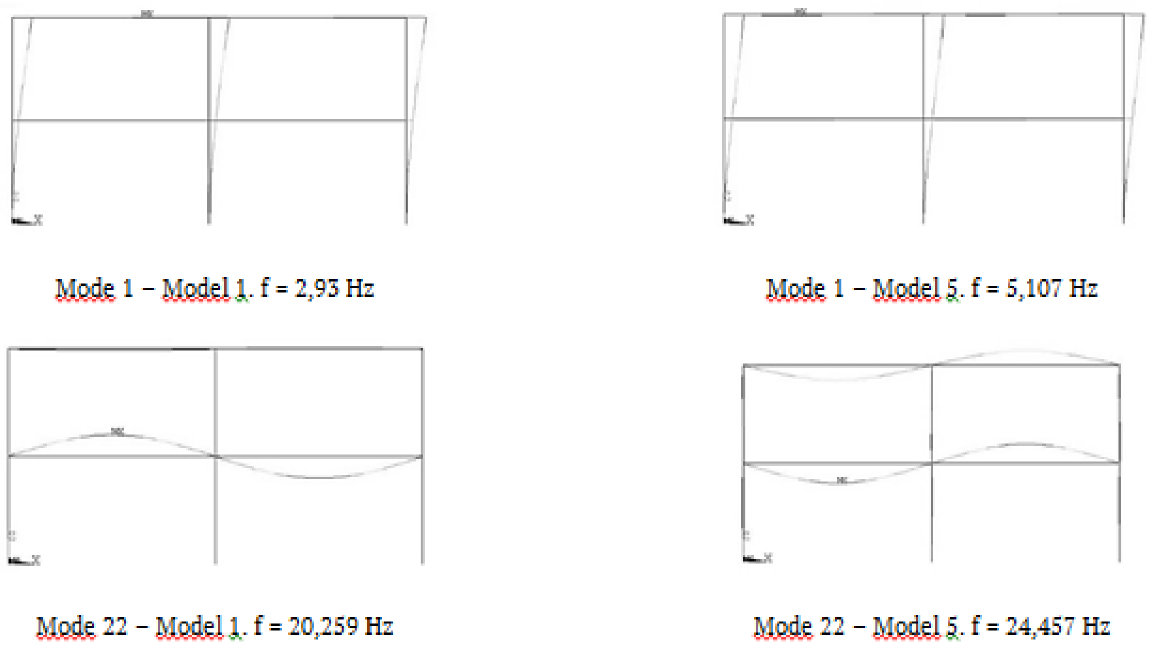

Figure 14. Deformed configuration of models 1 and 5 for vibration modes 1 and 22.

Harmonic analysis

Since the harmonic analysis determines which vibration modes of the structure are more significant in relation to steady state excitation, the nodes shown in Figure 15 were monitored in that analysis.

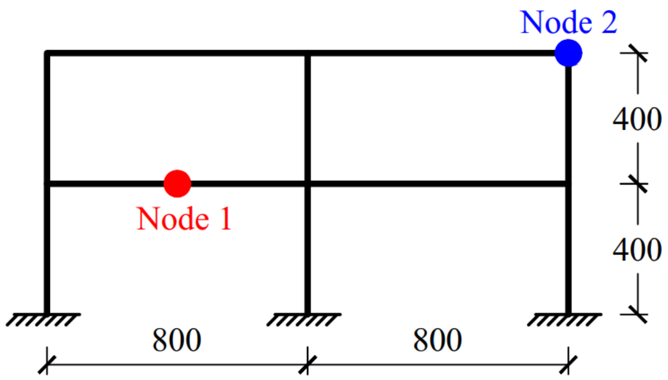

Figure 15. Monitored nodes in harmonic analysis for displacement evaluation. Dimensions in cm.

Because this response is provided as a function of the frequency spectrum of the nodal displacements, the response spectra were determined as a function of the position of the nodes of interest. For that, the modal superposition method was used. Considering the loads in Figure 6 and the procedures described in the section "Harmonic analysis", Figure 16 shows the vertical displacement of node 1 as a function of the studied frequency range, for the five models. Table 4 presents the displacement amplification, that is, the relationship between the maximum dynamic vertical displacement ($u_{y,dynamic}$) and the maximum static vertical displacement ($u_{y,static}$) of node 1, for all models.

From Figure 16 and Table 4, the following observations can be made about the analyzed models:

- There was an amplification of displacements of the order of 417 times the value of the static displacement;
- As the connection's stiffness increases, the resonant frequency (the one at which maximum displacement occurs) also increases;
- No clear correlation between connections stiffness and amplification was detected.

Similarly, considering the horizontal loads in Figure 7 and the procedures in the section "Harmonic analysis" of this work, Figure 17 shows the horizontal displacement of node 2 as a function of the studied frequency range, for the five models and Table 5 shows the horizontal displacement amplification.

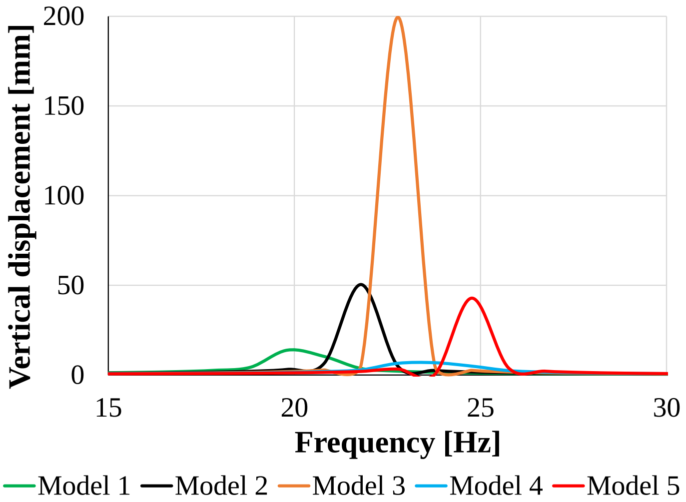

Figure 16. Vertical displacement amplitude of node 1 as a function of the studied frequency range.

Table 4. Amplification of node 1 vertical displacements.

Model	$u_{y,static}$ [mm]	$u_{y,dynamics}$ [mm]	Amplification	Frequency [Hz]
1	0.596	13.83	23.20	19.81
2	0.525	50.48	96.16	21.79
3	0.478	199.49	417.33	22.78
4	0.445	6.88	15.46	23.77
5	0.421	42.87	101.83	24.76

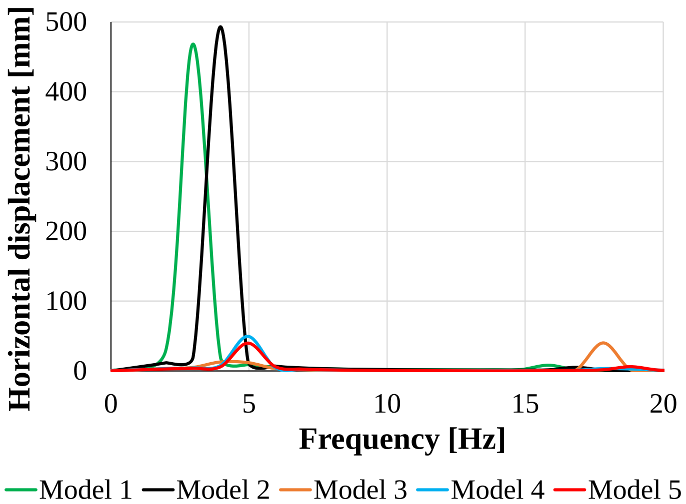

Figure 17. Vertical displacement amplitude of node 1 as a function of the studied frequency range.

Table 4. Amplification of node 2 horizontal displacements.

Model	$u_{x,static}$ [mm]	$u_{x,dynamics}$ [mm]	Amplification	Frequency [Hz]
1	15.829	468.17	29.58	19.81
2	8.658	493.07	56.95	21.79
3	6.616	40.16	6.07	22.78
4	5.629	49.60	8.81	23.77
5	5.04	39.72	7.88	24.76

In this case, from Figure 17 and Table 5, it can be seen that the maximum amplification of the static horizontal displacement occurred for model 2 and was 57 times, approximately. As mentioned for the vertical analysis, there was no clear correlation between the amplification and the stiffness of the beam-column connections, however, it was found that when making the frame more rigid, the resonance frequency also increased.

To be able to assess the amplification factor sensitivity against the stiffness of beam-column connections, it is necessary to analyze more models. Furthermore, it is important to study the behavior of frames with more (higher) floors, in which the action of lateral loading is more influential.

Conclusion

The present work aimed to evaluate, through numerical analysis by finite elements using ANSYS Mechanical APDL software, the static, modal and harmonic response of two-story steel frames in order to investigate the influence of the rigidity of the connections on the behavior of the structure. The numerical modeling used three-dimensional finite elements of the BEAM188 type, to represent the steel beams and columns, and elements of the COMBIN14 type, to represent the semi-rigid connections. In the studied numerical models, the connections behavior showed coherence, confirming that the applied method was efficient.

In static analyses, the more rigid the connections, the smaller the displacements, which is justified, since the overall rigidity of the structure increases, allowing it to better distribute internal efforts.

From the modal carried out, it was found that the predominant mode was bending in the frames plan, both for the beams and for the columns. Additionally, it was possible to verify that the fundamental mode of the structure, bending the columns in the plane, presented a growth in the natural frequency due to the increase in the stiffness of the connection. Considering that the Brazilian standard NBR 8800 (ABNT, 2008) recommends a minimum natural frequency of 3 Hz for floor structures, the stiffness of the connections is an important parameter in this context. In the analysis carried out, for example, only model 1 (with the most flexible connection among all) presented a natural frequency for the first vibration mode below 3 Hz.

However, from the harmonic analysis performed, it was not possible to delineate a behavior between the stiffness of the connections and the dynamic amplification, although it has occurred. To investigate this behavior in more detail, it is necessary to carry out additional analysis, covering a greater range of variation in the stiffness of the connections, also contemplating flexible and rigid connections. In addition, it is believed that carrying out this analysis by varying the number of floors, that is, the height of the frames, is fundamental, above all, for analysis with the action of horizontal forces, such as wind, because in tall buildings these actions exert greater dynamic influence. Furthermore, the present work did not consider the contribution of the concrete slab in the analyzes carried out. However, it exerts considerable importance in the behavior of connections, especially composite connections, increasing its rigidity. In view of that, the reproduction of the present study evaluating the effect of the concrete slab would certainly lead to interesting results.

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