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OPTIMIZATION OF STIFFNESS AND DAMPING COEFFICIENTS OF CONNECTION DAMPERS TO REDUCE THE DYNAMIC RESPONSE OF TRANSMISSION LINE STEEL TOWERS SUBJECTED TO WIND ACTION

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ABSTRACT

Tall and slender latticed steel towers, such as power transmission line towers, are very susceptible to vibrations imposed mainly by wind action. Thus, changing the design layout or making use of vibration control devices is often necessary to reduce vibration amplitudes and avoid the collapse of the structure. In this work, an alternative to the conventional types of commercial dampers is the use of elements in the connections of the structure, such as rubber rings working like connection dampers, so they can dissipate the energy of the system reducing the dynamic response of the tower. Thus, this work proposes a methodology for the optimization of stiffness and damping coefficients of connection dampers in structures of latticed steel towers of Transmission Lines (TL) that are subjected to the dynamic effects of wind. An illustrative example is presented. First, the structure is evaluated considering perfectly rigid connections; then the stiffness and damping coefficient of the connections are optimized in order to minimize the vibration amplitudes of the tower. Finally, the natural frequencies, damping ratios and maximum horizontal displacements are compared for situations of perfectly rigid and semi-rigid connections. The results show that the optimization process results in a structure with a fundamental frequency of vibration similar to that of the original tower, however a significant reduction in the horizontal displacements can be observed, since an increase in damping occurs, thus proving the capacity of the proposed methodology.

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1. Introduction

Structures such as towers have been the focus of study of many researchers, due to the fact that these structures are very sensitive to vibration effects caused by the wind and earthquakes. With the continuous expansion of the electric power generation and transmission system in Brazil and in the world, and the construction of new towers for telephone and internet services, new challenges are imposed on the designers.

Reduction in manufacturing, assembly and maintenance costs and, above all, the safety that these structures must have, are some of the main reasons for the greatest care in design and execution. The ruin of one of these structures can cause damage, such as the interruption of electricity transmission or telecommunications services. Besides, if they are built close to habitations, they could also cause fatalities.

A simplification that is normally used in the design of a structure is considering the connections between the elements as perfectly rigid or perfectly flexible. Muscolino et al. [1] mention that, in a practical way, if the moment transferred between the elements is sufficiently small – thus it can be neglected – the connections can be considered perfectly flexible. On the other hand, if the moment cannot be ignored, the connections are considered rigid.

However, in a real structure, observing these two extreme cases is practically impossible, since all connections allow some level of rotation and have some rigidity. In this sense, it is correct to state that the connections have a semi-rigid behavior. Zlatkov et al. [2] claim that underestimating the semi-rigidity of the connections and treating them as pinned has a negative impact on the costs of executing the structure, while overestimating, and treating them as rigid, produces results that do not match the reality and go against safety.

Usually, the actions used in the dimensioning of a latticed steel tower are the structure's self-weight, the weight of the transmission cables and antennas and, mainly, the wind action. As the design of towers becomes more and more slender, the natural frequency of vibration of these structures becomes smaller, which makes them more susceptible to wind actions. However, considering the semi-rigid behavior of the connections of a structure can help its dynamic response. Thus, as the semi-rigid connections represent more accurately the behavior of the structure, this is a topic researched by several authors.

Ye and Xu [3] investigated the static and dynamic responses of a steel frame with semi-rigid connections, simulated by a rotational spring of zero length. The authors noted that the resonance effect did not occur. They concluded that this happened due to the large power of dissipation capacity. Another conclusion of the work was that the frame with semi-rigid connections presented a greater resistance to collapse.

By analyzing the dynamic behavior, natural frequencies and vibration modes of a frame in an earthquake, Masoodi and Moghaddam [4] noted that the influence of the semi-rigidity of the connections for the highest acceleration peak values is important because they cause large variations in maximum displacement for rotational flexibility factor of connections close to zero (no rotation restriction). The authors also observed that, as the rotational flexibility approaches to rigid, for each of the analyzed acceleration peaks, the variation of the maximum displacement values becomes insignificant.

Similar results were achieved by Raftoyiannis and Polyzois [5], who studied the dynamic characteristics of a pole composed of two parts. The parts are joined with a resin, and the connection formed is considered semi-rigid. The authors performed a modal analysis and an experimental study exciting the structure. The results obtained showed that, as the stiffness of the connection increases, the influence of semi-rigidity on the dynamic characteristics decreases. This is due to the fact that after a certain level of rigidity, the connection starts to reach an almost perfect rigidity behavior.

Basiński and Kowal [6] performed a forced harmonic dynamic analysis on beams composed of the union of two bars with a semi-rigid connection and analyzed the displacement amplitudes in the middle of the beam. The effect of varying the rotational stiffness in the connection on the damping of the structure was also analyzed. The author demonstrated that, by reducing the rotational stiffness of the connections, the magnitude of the displacement amplitude also decreased. However, the author states that the increase in the stiffness of the connection leads to a decrease in damping, ushering the beam to reach greater displacement amplitudes in the harmonic excitation. Basiński and Kowal [6] also mention that, as semi-rigid connections regulate displacement amplitudes, they can act as vibration dampers for cyclical actions, such as wind gusts.

Daryan et al. [7] analyzed numerically a steel frame with two types of semi-rigid connections (top and seat angle with and without web angle) against an earthquake record. It has been shown that the top and seat angle with web angle reduced the relative displacement between the floors, and increased the structure's energy absorption capacity. The authors also concluded that the connection of a top and seat angle with a web angle is a suitable alternative for strengthening vibration-sensitive structures.

Wang et al. [8] also studied the effect of the semi-rigidity of connections against earthquakes and conducted an experimental study on tubular steel frames filled with concrete with bolted connections. The tests demonstrated excellent power dissipation capability.

Hao-Xiang He et al. [9] presented an experimental study in which it is inserted parts of low-yield-point steel in the connections of a structure. The low-yield-point steel proved to be more suitable than other types of ordinary steel in energy dissipation. Also, the authors concluded that it dissipates energy by deforming (protecting the main structure) and, after the dynamic force stops acting, maintenance can be quickly performed.

In this sense, dampers are usually used to reduce problems related to excessive vibration. However, the use of industrial dissipation devices implies a higher cost in the work. As an alternative, some authors suggest the use of elements with high energy dissipation capacity in the structural connections in order to improve the dynamic response.

Zlatkov et al. [2] determined the matrices of beam elements with semi-rigid connections at both ends through the stationary potential energy function. Afterward, the authors conducted a numerical study applying an earthquake to a structure and varying the stiffness level of the connections. Results showed significant differences regarding the natural frequency of vibration, design forces and horizontal displacement. According to the authors, this demonstrates the importance of using the real stiffness of the connection in the calculation step for any engineering structure. Another configuration also used to consider semi-rigidity is to insert high flexibility elements in the connection or in some section of the structure. This alternative not only makes the connection semi-rigid, but also increases the structure's energy dissipation capacity.

Sekulovic et al. [10] studied the energy dissipation of a frame considering connections such as rotational springs and viscous rotational dampers. In that work, the stiffness matrix of the structure for this configuration was deduced and a parametric study was conducted. The authors demonstrated that frames with semi-rigid connections combined with connection dampers presented a more significant decay in displacement amplitude when compared to rigid connections.

Cacciola et al. [11] performed a deterministic and stochastic sensitivity analysis of the dynamic response of a frame with semi-rigid connections combined with connection dampers. Dampers are represented by elements whose behavior follows the Kevin-Voigt model. The authors noted that, in some situations, even with a small variation of the damping of the connection, the response can have large differences.

Attarnejad et al. [12] analytically evaluated the performance against a high-intensity earthquake of a structure with semi-rigid connections and energy dissipation elements. The authors observed that the greater the stiffness of the connection, the greater the initial overall stiffness of the structure. Furthermore, it was also observed that there is an optimal damping coefficient that can significantly reduce the dynamic response of frames with semi-rigid connections. The authors also point out that semi-rigid connections with connection dampers are significant in the design of structures resistant to earthquakes.

Yanxia Zhang et al. [13] and Meng-Yao Cheng et al. [14] presented an experimental study by adding columns with and without dampers in steel frames and observed that the structures with dampers perform better against dynamic action due to a better capacity of energy dissipation.

Köroğlu et al. [15] also studied connection dampers as an option to reduce the effects of seismic loads on the structural system. The results showed that, when loads and moments reached critical levels, the connection dampers acted and no damage was observed on the beam and column.

As already mentioned, the limitation of costs around the project requires that it has the best performance with the available resources. To achieve this, it employs the use of optimization, maximizing or minimizing a function, by applying computational simulations.

In order to use the device in the best possible way, that is, with lower cost and greater power dissipation capacity, many authors have studied the optimization of these devices. The use of optimization aims at finding the properties of devices; and/or the best positions for the device within the structure; and/or the ideal number of devices to obtain the best scenarios they can provide, thus reducing the effect of dynamic actions and prolonging the service life of the structure in which they are installed. Singh and Moreschi [16] point out that even a small increase in damping can be relevant, as slender structures have an extremely low damping value.

Generally, studies related to damping optimization are carried out on commercial dampers. For example, Si et al. [17] studied the optimization of the position and properties of a Tuned Mass Damper (TMD), in order to reduce the dynamic effects in an offshore wind power generation tower. The study showed that, by applying adequate properties and positioning the TMD above water, it is possible to reduce the effects caused by dynamic actions moderately, while improving the effectiveness of energy generation. Other examples of TMD optimization in different types of structures subjected to different types of dynamic excitation can be found in [18-27].

Mensah and Dueñas-Osorio [28] analyzed a Tuned Liquid Column Damper (TLCD) to provide greater reliability for wind power towers against excessive wind-induced vibrations. Results showed that the shock absorber used is a viable possibility, since it is cheaper and able to provide greater safety, as well as a useful life for these towers.

Zhang et al. [29] also analyzed vibrations in wind energy towers, and studied dampers to reduce the effects of vibration on turbine blades. The aim of the study was to optimize the damper mass, frequency, coefficient of friction and position. Results showed that the best conditions to achieve vibration reduction are to increase the damper mass and position it close to the free end of the structure.

Regarding friction dampers, the works by [30-38] can be highlighted, which propose different methodologies for optimizing these devices when applied to different structures subjected to different dynamic actions.

Ribakov and Reinhorn [39] presented a study based on the optimization of the damping of a structure by viscous dampers in a lever system. The authors conducted a numerical example and concluded that peak displacements were reduced to levels between 40% and 75%. Still, there was a notable gain in energy dissipation, thus indicating that the application of dampers can be an alternative for the recovery of structures sensitive to earthquakes.

As demonstrated in the works above, the use of optimization algorithms to assist in the design of external energy dissipation devices has been the focus of research by several authors, who propose modifications in the properties of dissipation devices in order to reduce the dynamic response of the structure to different excitations. However, there have been few studies on the optimization of the dynamic characteristics of structures with semi-rigid connections formed by less rigid elements, such as rubber rings, acting as an energy dissipation device (connection damper).

Thus, for the reasons mentioned so far, as well as for the lack of research on the subject, the present work proposes a methodology for the optimization of connection dampers inserted in the connections of a steel transmission line tower subjected to the dynamic action of synoptic winds (EPS winds), aiming to reduce the maximum displacement of the structure, that is, to improve its dynamic response.

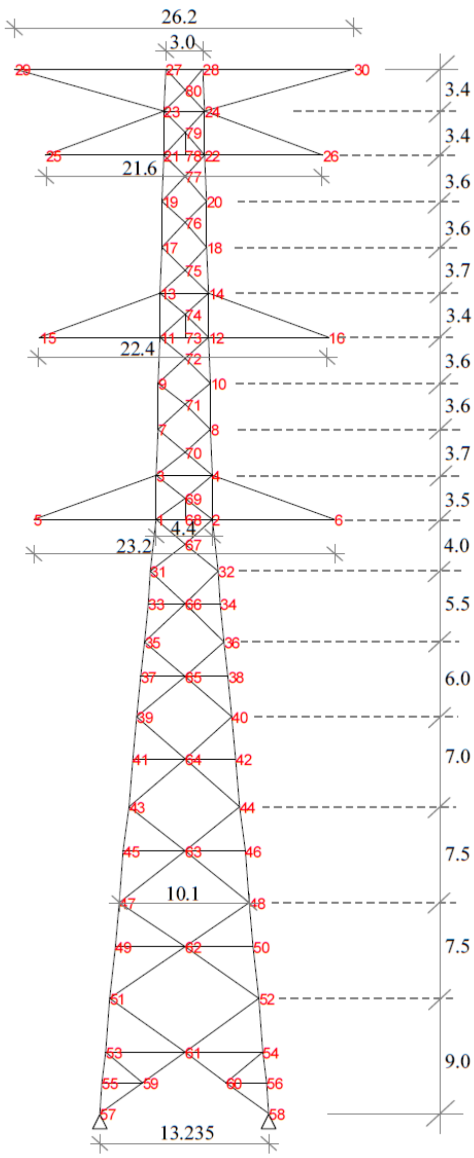


Fig. 1 Transmission tower - dimensions (in meters) and number of nodes. [41]

2. Definition of the structure

To illustrate the methodology proposed in this work, a real power transmission line tower (Fig. 1) damaged during a typhoon in Japan is studied. This structure has already been studied by Murotsu et al. [40] and Miguel and Fadel Miguel [41] in different contexts.

The structure is a silhouette of an 82-meter-high tower, with a base of 13.235 meters, and it is formed by angle profiles. The tower is modeled with 80 nodes and 163 2D frame elements. The representation of the tower with its nodes and dimensions is presented in Fig. 1.

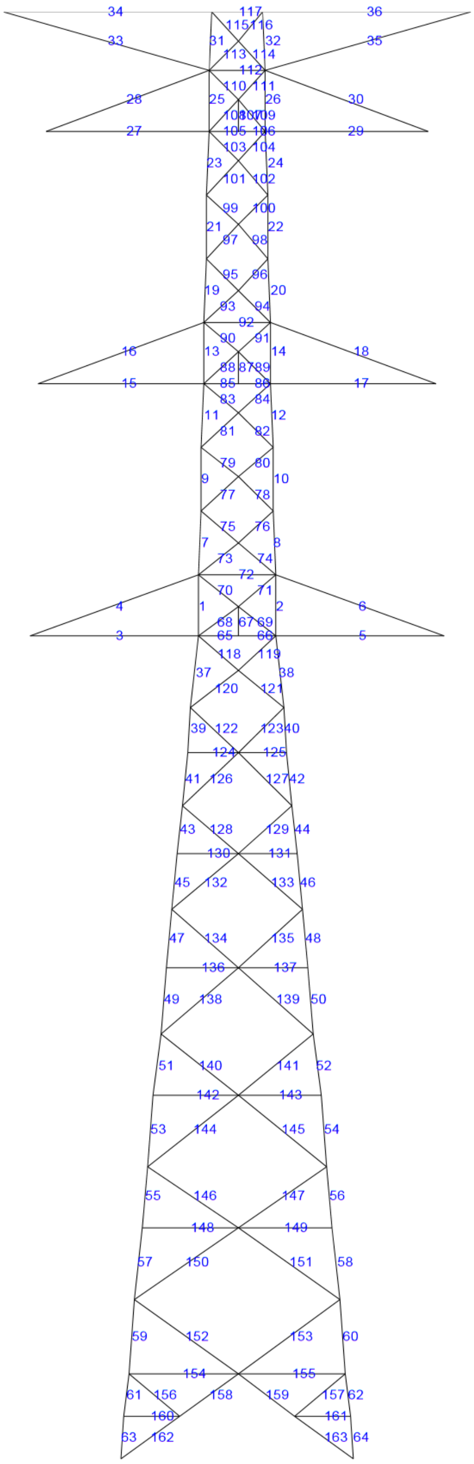


Fig. 2 Transmission tower - number of elements. [41]

In this study, it is assumed that: (i) the tower is located in the city of Chuí (RS, Brazil), unlike the original structure, located in Japan; (ii) the structure has a completely rigid base; (iii) the presence of cables is simulated by adding masses in some nodes of the structure. The corresponding mass value and the nodes where they are added were given by [40] and [41] and are presented in Table 1.

Table 1
Nodal masses added to the structure

Nodes	Mass (kg)
5, 6	1550
15, 16	1550
25, 26	1550
29, 30	130

The tower is formed by 11 different angle profiles, which have been adapted to Brazilian commercial angle profiles taken from [42], and the material used in the profiles of the structure is steel ASTM A36. The geometrical properties of the angle profiles are presented in Table 2 and the profile used in each element of the structure is presented in Table 3. The number of each element is presented in Fig. 2.

Table 2
Geometrical properties of the angle profiles

Number	Profile (mm)	Area (cm^2)	Moment of inertia (cm^4)
1	20.32 x 20.32 x 1.905	73.81	2901.1
2	20.32 x 20.32 x 1.588	62.9	2472.4
3	12.70 x 12.70 x 0.952	23.29	362
4	10.16 x 10.16 x 0.794	15.48	154
5	15.24 x 15.24 x 1.270	37.09	828
6	15.24 x 15.24 x 1.588	45.86	1007
7	15.24 x 15.24 x 1.905	54.44	1173
8	5.08 x 5.08 x 0.952	8.76	20
9	6.35 x 6.35 x 0.794	9.48	35
10	10.16 x 10.16 x 0.635	12.51	125
11	4.445 x 4.445 x 0.794	6.45	11.2

Table 3
Angle profile of each element used in the structure

Number	Element
1	1-8, 15-18, 27-30, 33-36
2	9-12, 65-67, 72, 85-87, 92, 105-107, 112, 117
3	13, 14, 19, 20
4	21-26, 31, 32
5	37-46
6	47-54
7	55-64
8	68-71, 73-84, 88-91, 93-104, 108-111, 113-116, 118-131, 136, 137, 142, 143, 148, 149, 156, 157
9	132-135, 138-141, 146, 147, 154, 155, 158, 159, 162, 163
10	144, 145, 150-153
11	160,161

The damping matrix is considered proportional to the mass and stiffness matrices, according to Eq. (1), and a damping ratio of 0.5% is considered for the first two modes.

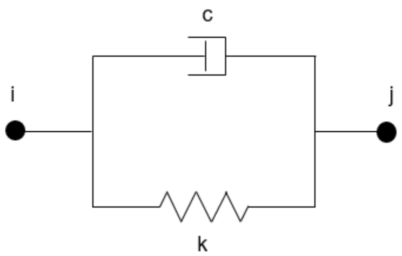
$$C_M = \alpha M_M + \beta K_M + CM \tag{1}$$

in which M_M , C_M and K_M are the mass, damping coefficient and stiffness matrices of the structure, respectively, CM is the global damping coefficient matrix of the connections dampers and α, β are constants.

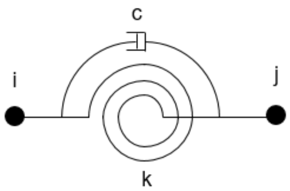
Two configurations of connections are studied in this paper. The first one considered that all bolted connections are perfectly rigid. The second one considered that all bolted connections with connection dampers have a semi-rigid behavior. The configurations are called rigid and semi-rigid, respectively. For both configurations, the bolted connections between elements that represent the main legs of the tower are considered rigid.

3. Definition of the connection damper

The connection damper is represented by an element with zero length, and it works as a connection between two 2D frame elements. As it is a 2D element, the connection damper has stiffness and damper coefficient in all three degrees of freedom and it can be exemplified as a system of spring and damper. Therefore, it can be depicted by two translational parts that represent the horizontal and vertical displacements, and one rotational part that represents the angular displacement. The depiction of the connection damper's parts is presented in Fig. 3.



(a) Translational part



(b) Rotational part

Fig. 3 Connection damper

As the length of the connection damper is zero, the end node of the first 2D frame element is superimposed (same position) with the start node of the second 2D frame element that makes up the connection. Furthermore, as the mass of this element is insignificant compared to the mass of the structure, its addition can be ignored.

Each element has stiffness in all three degrees of freedom. If it is necessary for the connection to have a perfectly rigid behavior, it assumes a very high value for the spring stiffness. If the connection has a perfectly flexible behavior, the spring assumes a stiffness equal to zero.

The stiffness matrix of the spring part of the element used in this study is defined by Eq. (2).

$$KM = \begin{bmatrix} KU_X & 0 & 0 & -KU_X & 0 & 0 \\ & KU_Y & 0 & 0 & -KU_Y & 0 \\ & & KROT_Z & 0 & 0 & -KROT_Z \\ & & & KU_X & 0 & 0 \\ & sym & & & KU_Y & 0 \\ & & & & & KROT_Z \end{bmatrix} \tag{2}$$

in which KU_X and KU_Y are the translational stiffness in relation to the X and Y axis, respectively. Also, $KROT_Z$ is the rotational stiffness in relation to the Z axis.

The stiffness values for the three degrees of freedom are already calculated and applied in relation to the global coordinates, so the coordinate transformation matrix is unnecessary.

The damping coefficient matrix of the damper part of the element used in this study is defined by Eq. (3).

$$CM = \begin{bmatrix} CU_X & 0 & 0 & -CU_X & 0 & 0 \\ & CU_Y & 0 & 0 & -CU_Y & 0 \\ & & CROT_Z & 0 & 0 & -CROT_Z \\ & & & CU_X & 0 & 0 \\ & sym & & & CU_Y & 0 \\ & & & & & CROT_Z \end{bmatrix} \tag{3}$$

in which CU_X and CU_Y are the translational damping coefficient in relation to the X and Y axis, respectively. Also, $CROT_Z$ is the rotational damping coefficient in relation to the Z axis.

Like the stiffness matrix, the damping coefficient matrix already has its coefficients in global coordinates, and the coordinate transformation matrix is unnecessary. Besides, if the connection is not responsible for any part of the damping ratio of the structure, it assumes that the damper part is null. On the other hand, if the connection assists the damping of the structure, the damper part is not null.

The stiffness and damping coefficient matrices are the matrix of the element CONBIN40, the spring element from ANSYS [43].

For the connection dampers to be considered to be made of the same material, the translational stiffness in both X and Y axes must be the same for any inclination of the structure bars. To do so, the values of KU_X and KU_Y must be equal. In relation to the damping coefficients, it is considered that the material has the same energy dissipation capacity for each of the degrees of freedom analyzed. Therefore, the values of CU_X , CU_Y and $CROT_Z$ are also identical.

When the connections of the structure are considered rigid, the CM matrix is considered null. In the second case, which considers the semi-rigid behavior of the connections, the CM matrix assumes the values of the damping coefficients of the connection dampers.

4. Definition of wind action

The structure is evaluated as if it were installed in Brazil, in the state of Rio Grande do Sul, in the city of Chuí, in a place with flat and open terrain, with few obstacles and isolated. The tower is planned to support transmission lines to the city, and its collapse does not affect people's safety, once it is located in a land with few obstacles.

The city of Chuí was chosen as the location for the installation of the structure because this is the region with one of the highest mean wind velocities in Brazil.

The wind action is subdivided into two parts: the mean static portion and the floating one. Before proceeding to the calculation of forces, it is interesting to determine both static and floating portions of wind velocity, then determine the definitive wind forces that are acting on the structure. Once the velocities are determined, the process of calculating the forces is identical for the static and floating portions.

To determine the mean static wind velocity acting on the structure of a latticed tower, the procedure described in NBR 6123/1988 [44] is used. To simulate the floating portion of the wind, the Davenport power spectrum is used and the Shinozuka and Jan method [45], also known as the spectral representation method, is used to simulate a random process through a series of cosines to produce the sample functions. This function is used as the basis for generating the spectral density sample.

To determine the floating components, the method proposed by Miguel et al. [46] is employed. The frequency range of the analysis is from 0.01Hz to 5Hz, with an interval Δf of 0.01Hz; and the time range is from 0s to 100s, with an interval Δt of 0.02s.

Furthermore, to calculate the floating wind velocity at different heights, it is necessary to determine the correlation length. Eq. (4) by [46] empirically relates the correlation length with the height of the studied structure. This equation is estimated through a linear regression obtained in relation to different heights and surface roughness presented in Fig. 4.

$$a = 0.93z + 29.3 \tag{4}$$

in which a is the correlation length and z is the height above the ground.

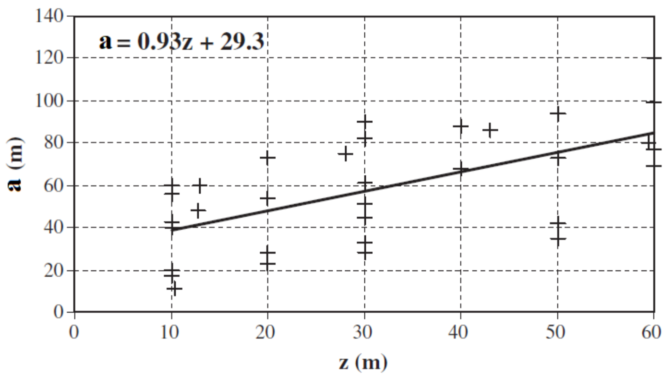


Fig. 4 Vertical correlation length [46]

Thus, it is possible to generate the floating components at points spaced apart by a correlation length and the intermediate components by means of interpolation according to Eq. (5).

$$V_f(z, f) = V_1(t) + \frac{V_2(t) - V_1(t)}{a} z \tag{5}$$

in which $V_1(t)$ and $V_2(t)$ are the wind velocity in height zero and a , respectively.

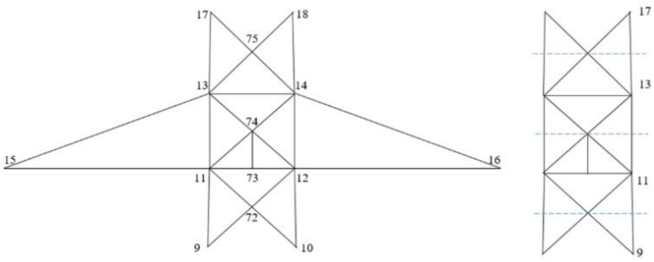
In possession of the velocity field for both mean static and floating parts, the total wind velocity is given by Eq.(6).

$$V_t(z,t) = V_S(z) + V_f(z,t) \tag{6}$$

in which $V_S(z)$ and $V_f(z,t)$ are the mean static wind velocity and the floating one, respectively.

As this study analyzes a 2D frame structure, an area might be supposed to generate the wind force profile. It is assumed that the lateral silhouette of the structure is the same as the front silhouette without the braces. Fig. 5 features a front and side view layout for some nodes.

After determining the influence area for all the different heights, it is possible to calculate the drag forces applied in the structure according to NBR 6123/1988 [44].



(a) Frontal view (b) Lateral view

Fig. 5 Example of a section’s frontal and lateral views

5. Optimization process

In this study, the objective function is to minimize the maximum horizontal displacement at the top of the structure. The design variables are the stiffness and damping constants of the elements inserted in the connections (connection dampers). To carry out the optimization proposed in this work, the Whale Optimization Algorithm (WOA) developed by Mirjalili and Lewis [47] is implemented.

The optimization study is carried out on the structure so that it has the smallest maximum horizontal displacement possible. For this, the stiffnesses KU_X , KU_Y and $KROT_Z$, and damping constants CU_X , CU_Y and $CROT_Z$ of the connection damper are modified for each optimization step. As previously mentioned, the stiffnesses KU_X and KU_Y are identical, as are the damping constants CU_X , CU_Y and $CROT_Z$. Therefore, optimization is performed by modifying three parameters.

A flowchart of the optimization process is presented in Fig. 6.

6. Proposed methodology

At first, the structure is studied without the connection dampers, and its connections are considered rigid. A modal analysis is performed and its natural frequencies of vibration are determined. Then, an EPS wind (a synoptic wind) is applied to the structure, and the horizontal displacements during the entire period of action and the damping ratios related to each of the structure's natural frequencies are determined. Fundamental natural frequency, damping ratio and maximum horizontal displacement data are stored for future comparison.

Subsequently, connection dampers are inserted into the structure's connections. At this moment, through the optimization process, the stiffness and damping coefficient of the connection dampers are optimized so that the maximum horizontal displacements are minimized. The natural vibration frequency, the damping ratio and the maximum horizontal optimized displacements are compared to that of the previous situation, in which the structure does not have the added damping from the connection dampers.

But it is important to note that not all the connections receive the connection dampers. The connections between the elements of the main legs are considered rigid in both configurations. All the other connections (diagonal bracing – main leg, diagonal bracing – horizontal bracing, diagonal bracing – diagonal bracing, horizontal bracing – main leg, horizontal bracing – horizontal bracing, cross arm – main leg and cross arm – cross arm) are considered semi-rigid with a connection damper between the elements.

The optimization of the connection dampers is carried out by using the WOA (metaheuristic algorithm programmed in MATLAB), and the natural frequencies and the vibration modes are obtained through an algorithm elaborated using the finite element method, solving the eigenproblem of the system. For the structure with perfectly rigid connections, the damping ratio used for the first two modes of vibration is 0.5%, as a function of the structure's natural damping, while for the structure with connection dampers, the damping ratio is calculated by the equations of uncoupled motion. The generation of the wind field velocities and forces, as well as the calculation of the structure's dynamic response, which allows the determination of the maximum horizontal displacement, are performed in computational routines developed in this work. The dynamic response of the structure is determined by using the Newmark method. All calculations and algorithms are developed in MATLAB language.

To impose a limit on the allowable displacement, the maximum horizontal displacement value during the entire application of wind action on the structure must not exceed 1% of the total height of the structure. The reference value applied in this work was used by [48] to define the maximum transverse or longitudinal displacement for the service limit state of a latticed metal tower of transmission lines.

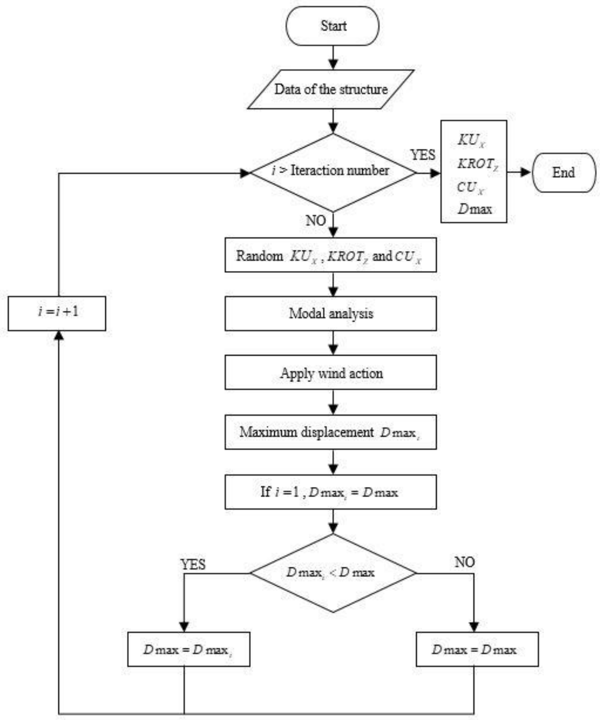


Fig. 6 Flowchart of the optimization process

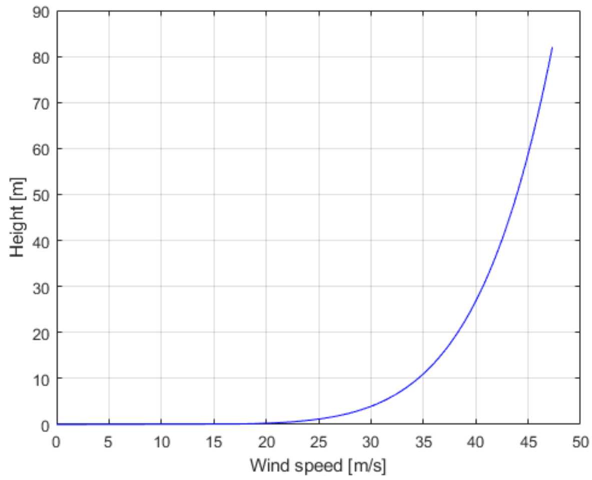


Fig. 7 Mean static wind velocity

7. Results and discussion

7.1. Wind force

The mean static wind velocity for all heights of the structure, determined by the procedure of NBR 6123/1988 [44], is presented in Fig. 7.

Knowing that the height of the structure is 82 meters and using Eq. (4), the correlation length becomes $a = 105.56m$.

As the correlation length is higher than the tower's height, only two signals generated by the spectral representation method are necessary to create the floating wind velocity field: one for the height of zero (ground), and the other for the height of the correlation length (105.56m). All floating wind velocities for the intermediate points are given by Eq. (5).

With the mean static and floating wind velocities portions, it is possible to determine the total wind velocity acting on each node of the tower. For instance, Fig. 8 shows the wind velocity at node 27 throughout the analysis period.

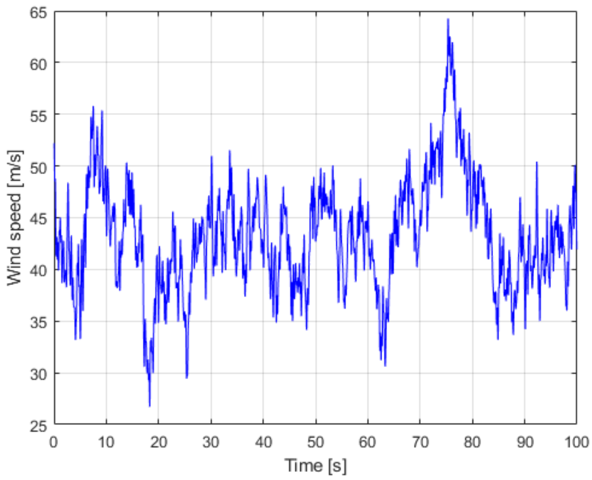


Fig. 8 Wind velocity at node 27

With the total wind velocity for all heights of the structure and the influence areas for all nodes, the total drag forces can be calculated according to NBR 6123/1988 [44]. For instance, Fig. 9 shows the drag force at node 27 throughout the analysis period.

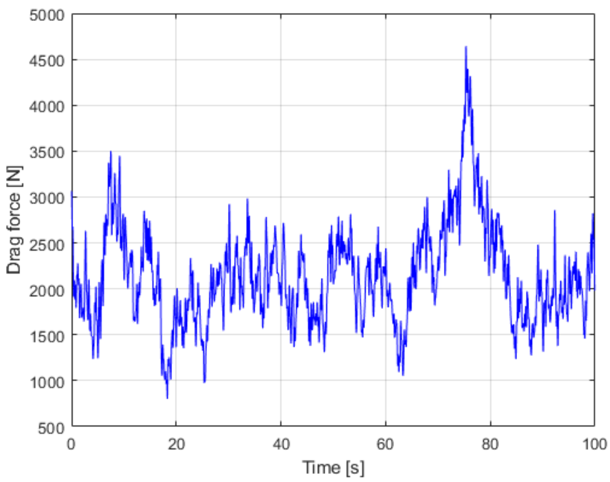


Fig. 9 Drag force at node 27

7.2. Structure with rigid connections

The modal analysis is conducted on the structure presented in Fig. 1, which was modeled in a program developed in MATLAB. As the developed program has a connection element in all points that bracing meets the main legs, for the perfect rigid connections without connection dampers, the stiffness in all degrees of freedom has a big value and the damping coefficient is null, as presented in Table 4.

Table 4
Stiffness and damping coefficient for rigid connections

Degree of freedom	Stiffness	Damping coefficient
U_X	$1E12\ N{\cdot}m^{-1}$	$0\ N{\cdot}s{\cdot}m^{-1}$
U_Y	$1E12\ N{\cdot}m^{-1}$	$0\ N{\cdot}s{\cdot}m^{-1}$
ROT_Z	$1E12\ N{\cdot}m{\cdot}rad^{-1}$	$0\ N{\cdot}s{\cdot}m{\cdot}rad^{-1}$

The value used in the stiffness for the three degrees of freedom is enough for the connections to be considered perfectly rigid. This value is validated by comparing the results obtained with the ANSYS program, simulating the same structure with rigid connections without connection elements.

The natural frequencies for the first three mode shapes, as well as the structure damping ratios, are presented in Table 5. The structure damping ratio, as already mentioned, is 0.5% for the first two vibration modes.

Table 5
Natural frequency and damping ratio for rigid connections

Vibration mode	Natural Frequency (Hz)	Damping ratio (%)
1	0.6187	0.5
2	2.0167	0.5
3	3.8458	0.79

Subsequently, the wind action is applied to the structure and, through the Newmark method, a dynamic analysis is performed. The wind force profile on the structure was determined in Section 7.1. Fig. 10 shows the horizontal displacements at the top of the structure (node 27).

As can be seen in Fig. 10, the maximum displacement at the top of the tower (node 27) with rigid connections is 0.9173m ($D\max = 0.9173m$).

7.3. Structure with semi-rigid connections

In this step, the connections are considered semi-rigid and, via the optimization process, the values of stiffness and damping coefficient of the connection dampers are defined. The objective function is to minimize the maximum horizontal displacement of the structure, changing the stiffness and damping coefficient of the connection dampers. The wind action applied to the structure was the same that was already applied to the configuration of rigid connections.

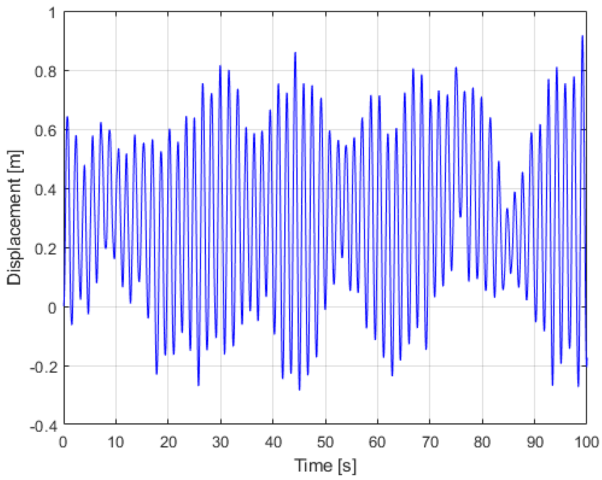


Fig. 10 Horizontal displacement at node 27 with rigid connections

The convergence curve of the optimization process is shown in Fig. 11, in which can be seen that the maximum displacement at the top of the tower (node 27) converges to its minimum value of 0.7056m ($D\max = 0.7056m$), in iteration number 67.

The results of the stiffness and damping coefficient of the connection dampers obtained in the optimization process are given in Table 6.

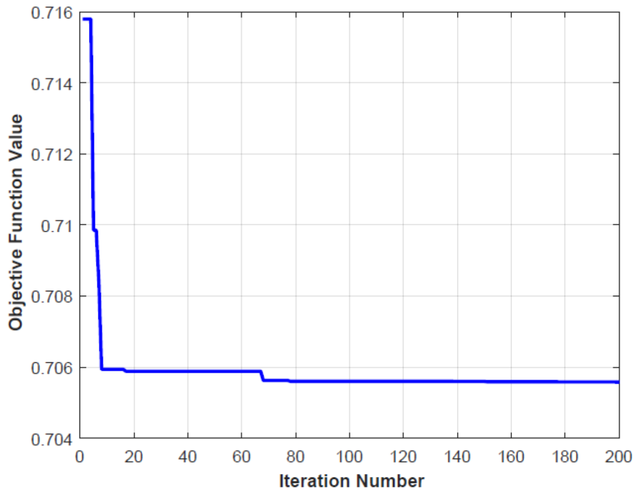


Fig. 11 Convergence curve

Table 6
Stiffness and damping coefficient for semi-rigid connections

Degree of freedom	Stiffness	Damping coefficient
U_X	$6.7035E7\ N{\cdot}m^{-1}$	$4.264E7\ N{\cdot}s{\cdot}m^{-1}$
U_Y	$6.7035E7\ N{\cdot}m^{-1}$	$4.264E7\ N{\cdot}s{\cdot}m^{-1}$
ROT_Z	$1.4633E7\ N{\cdot}m{\cdot}rad^{-1}$	$4.264E7\ N{\cdot}s{\cdot}m{\cdot}rad^{-1}$

With the optimum configuration of connection dampers, a modal analysis is conducted in the same way for the case of the structure with perfectly rigid connections. The first three natural frequencies as well as the structure damping ratios are presented in Table 7.

Then, the wind action is applied to the structure and a dynamic analysis is conducted. The wind force profile is the same applied in the configuration of rigid connections.

Table 7
Natural frequency and damping ratio for semi-rigid connections

Vibration mode	Natural Frequency (Hz)	Damping ratio (%)
1	0.6011	7.44
2	1.7096	115
3	2.8061	336

The horizontal displacements at the top of the structure (node 27) are shown in Fig. 12, in which can be seen that the maximum displacement at the top of the tower (node 27) with semi-rigid connections is 0.7056m ($D\max = 0.7056m$).

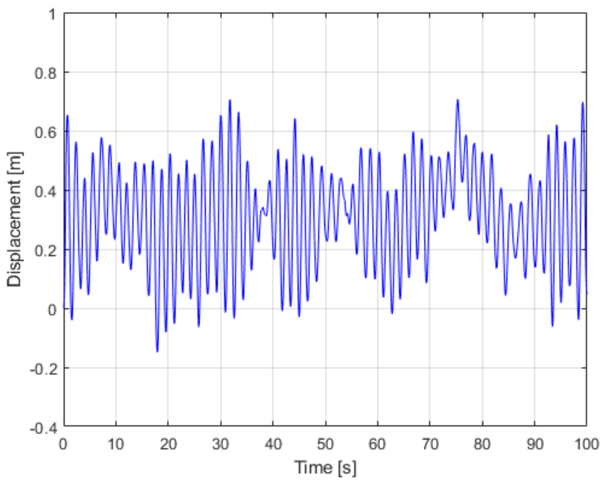


Fig. 12 Horizontal displacement at node 27 with semi-rigid connections

7.4. Comparison of results

The comparison of results for both rigid and semi-rigid connections is presented in Table 8, and the comparison of the horizontal displacements at the top of the structure (node 27) is presented in Fig. 13.

Table 8
Comparison of results for both rigid and semi-rigid connections

Data	Rigid	Semi-rigid	Difference
Fundamental frequency (Hz)	0.6187	0.6011	-2.84%
Damping ratio $(\%)$	0.5	7.44	1388%
Maximum horizontal displacement (m)	0.9173	0.7056	-23.08%
KU_X and KU_Y $(N{\cdot}m^{-1})$	1E12	6.7035E7	-
$KROT_Z$ $(N{\cdot}m{\cdot}rad^{-1})$	1E12	1.4633E7	-
CU_X , CU_Y $(N{\cdot}s{\cdot}m^{-1})$	0	4.264E7	-
$CROT_Z$ $(N{\cdot}s{\cdot}m{\cdot}rad^{-1})$	0	4.264E7	-

As can be seen in Table 8, comparing the results obtained, it is possible to notice that, by inserting a semi-rigidity in the connection, there is a small reduction (2.84%) in the fundamental frequency of the structure, however the damping ratio increases considerably (1388%), causing the maximum horizontal displacement at the top of the tower to be reduced by 23.08%.

Initially, the structure did not meet the maximum horizontal displacement requirements at the top of the Brazilian standard NBR8850/2003 [48], exceeding it by approximately ten centimeters (11.87%). However, after adding the connection dampers to the structure, the maximum horizontal displacement decreased by approximately twelve centimeters (13.95%) in relation to the standard limit [48].

Still, looking at Fig. 13, it is possible to visually notice that all horizontal displacements decreased. This indicates that the presence of the connection dampers also helps to reduce the displacement amplitude throughout the period of application of the wind force.

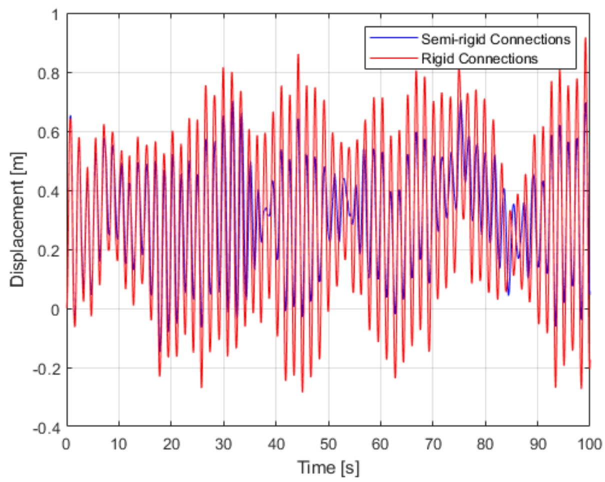


Fig. 13 Comparison of displacement at node 27 for both rigid and semi-rigid connections

8. Conclusions

The use of energy dissipation devices to minimize the effect of high displacements on structures susceptible to vibrations is widely studied. However, the vast majority of these studies are related to the addition of active or passive external dampers in the original structure. On the other hand, few studies focus on the research of the semi-rigidity of connections by inserting connection dampers to improve energy dissipation.

Thus, in order to reduce the dynamic response of a steel tower, in the present work, the insertion of connection dampers was proposed. A methodology to optimize the stiffness and damping constants of the connection dampers was proposed, minimizing the dynamic response of the structure.

Regarding the fundamental frequency of vibration, it was possible to notice that the frequency remains very similar to the one of the original structure, even with the decrease in translational and rotational rigidity, not allowing the dynamic action of the wind to be amplified.

Through the evaluation of the damping ratio in these two connection configurations, it was possible to notice a huge increase in relation to the original damping (1388%). This indicates that the presence of connection dampers in the structure improves the energy dissipation of the system.

Regarding the maximum horizontal displacement values, there was an important decrease in these values. The damping coefficient inserted in the connections was responsible for decreasing the maximum horizontal displacement at the top of the structure by approximately 23%.

Initially, the structure did not meet the maximum horizontal displacement requirements at the top required by the technical standard and exceeded it by approximately 12%. However, after adding the connection dampers, the maximum horizontal displacement decreased by approximately 14% in relation to the stipulated maximum limit, and 23% in relation to the horizontal displacement with rigid connections.

Comparing the results obtained for the structure with rigid and semi-rigid connections, it was possible to perceive that the structure has improved its performance against wind action, with more flexible elements in the connections.

Thus, it is believed that the methodology proposed in this work can be an excellent alternative to reduce the dynamic response of steel structures.

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