STRUCTURAL RELIABILITY OF A STEEL COLUMN UNDER FIRE CONDITIONS

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Abstract. The structural projects are usually made in a deterministic way through normative codes that do not consider directly the uncertainties associated to the problem. However, for exceptional cases such as fire and earthquake conditions, the use of prescriptive methodologies may lead to projects with high costs and indeterminate or unacceptable levels of safety. In this context, the application of the reliability theory to structures under fire conditions arises as a better option than the deterministic methods contained in the normative codes. The structural reliability analysis enables that the uncertainties associated to each variable are considered, quantifying the safety level of the structure and giving the engineer a better comprehension of the structural behavior under fire situation. The present study aims to apply the structural reliability theory to a steel column under fire condition, analyzed by the ABNT NBR 14323:2013. First, the basis of the application of this kind of analysis is stated. Then, the methodology is applied through computer modules developed in Excel and MATLAB and the probabilities of failure are determined as a function of time, temperature, and other variables involved in the problem. Eventually, the results indicate that the deterministic design made according to the Brazilian code does not lead to an acceptable safety level proving the importance of the reliability analysis in the structural projects.

Keywords: Fire, Steel structures, Structural reliability

1 Introduction

Despite being one of the most used materials in the structural conception, the steel presents as one of the main disadvantages the vulnerability to fire. Steel structures suffer a rapid increase in temperature when exposed to fire, which causes a reduction in resistance, stiffness and, in some cases, additional internal forces.

On the other hand, in recent years several studies on the temperature rise in steel structures have been developed. According to Souto [1], in 1996 a study group that sought to develop criteria for the design of steel structures under fire conditions was created. Those criteria were used, in 1999, in the creation of ABNT NBR 14323 [2]. This standard had a new version released in 2013, after the update of ABNT NBR 8800 [3] in 2008.

In general, structural designs are based on semi-probabilistic requirements provided by normative codes, which prescribe values and requirements for the most varied situations. However, there are several uncertainties related to the variables of the problem. Changes in the mechanical and geometric properties of the structure, in the environment where the element is located or even in the load acting throughout the useful life can lead to situations in which the structural resistance is lower than the load demand.

The situation becomes even more critical for exceptional conditions such as earthquake or fire, since the number of uncertainties becomes even greater. Additionally, according to Tavares [4], the application of a prescriptive standard for problems that present unique characteristics can lead to projects with high costs and inadequate or undetermined safety levels.

In this context, the structural reliability analysis emerges as an alternative to the traditional prescriptive methods since it allows a more careful analysis of each project, enabling its particularities and the uncertainties associated with the problem to be considered. In addition, the structural reliability analysis allows the designer to have a better understanding of the structure in the most varied situations providing through the probability theory, a measure of the safety level of the structure, the failure probability.

In view of this, this work aims to apply the structural reliability to the study of a steel column under fire conditions designed according to ABNT NBR 8800:2008 [3] and verified according to ABNT NBR 14323:2013 [2]. For that, computational modules developed and coupled with the function of analyzing the structure under fire conditions and applying the methods of structural reliability are used.

2 Fire models

In order for the design of steel structures under fire conditions to be carried out properly, it is necessary that the variation of the ambient temperature as a function of the time of fire exposure is known. However, due to the great variability of the parameters involved in a fire (ventilation openings, fuel loads, among others), the construction of a real fire temperature-time curve becomes difficult, which leads to the use of more simplified models.

Silva [5] highlights two models that are widely used for fire simulation: the standard fire and the natural compartment fire. The first is the one in which it is considered that the gas temperature obeys the curves used in fire resistance tests, not necessarily representing a real fire.

The main characteristic of standard fire curves is that they have only one ascending branch, assuming that the temperature of the gases is only increasing over time and independent of the characteristics of the environment and of the fire load. ABNT NBR 14432:2000 [6] and EN 1991-2-2:2002 [7] indicate the use of the expression provided by ISO 834:1999 [8] for the construction of the standard fire curve, which is presented in Eq.(1).

$$\theta_g = 20 + 345 \log(8t + 1). \tag{1}$$

In the Eq. (1), t is the time in minutes and θ_g is the gas temperature (ºC).

On the other hand, the natural compartment fire according to Guimarães [9], is the one for which it is admitted that the temperature of the gases respects the natural temperature-time curves, built from tests that simulate the situation of a burning room. This model is directly influenced by three factors: the fire load; the opening factor of the environment; and the thermal inertia of the wall lining materials.

An example of a temperature-time curve for natural compartment fire is shown in Fig. 1. It can be noted that the main characteristic of these curves is the existence of an ascending branch and a descending branch, assuming rationally that the fire gases do not have their temperature always increasing with time.

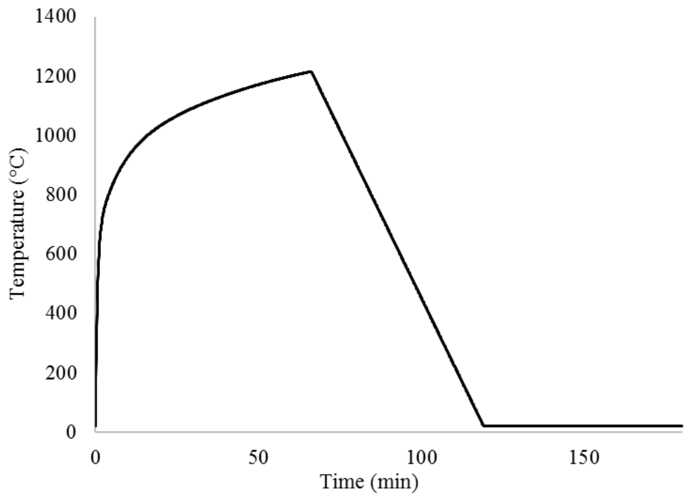


Figure 1. Temperature-time curve for a natural fire.

The parametric fire curves given by EN 1991-1-2:2002 [7], used in this work are widely used for the modeling of natural fires. These curves are valid for fire compartments up to 500 m^2 of floor area, without openings in the roof and for a maximum compartment height of 4 m. The procedure for the construction of these curves is detailed in this standard. ABNT NBR 14323:2013 [2] does not indicate any procedure for construction of the natural fire curve, but allows its use.

3 Design of compressed steel elements under fire conditions

The ABNT NBR 8800:2008 [3] presents the script for obtaining the design axial load capacity at normal temperatures while the ABNT NBR 14323:2013 [2] presents the same under fire conditions and the procedure for determining the temperature on the steel structure.

For I-sections at normal temperatures, according the ABNT NBR 8800:2008 [3] the design axial load capacity can be calculated by the Eq. (2).

$$N_{c,Rd} = \frac{\chi Q A_g f_y}{1.10} . \tag{2}$$

In the Eq. (2), $N_{c,Rd}$ is the design axial load capacity; A_g is the cross-sectional area of the element; f_y is the yield strength of the steel; Q is the strength reduction factor due to local buckling obtained according to Annex F of ABNT NBR 8800:2008 [3]; and χ is the strength reduction factor due to initial imperfections and geometric and material nonlinearities calculated by Eq. (3) or Eq. (4).

$$for\ \lambda_0 \leq 1{,}5 \rightarrow \chi = 0{,}658^{\lambda_0^2} . \tag{3}$$

$$for\ \lambda_0 > 1{,}5 \rightarrow \chi = \frac{0{,}877}{\lambda_0^2} . \tag{4}$$

In the above equations, the parameter λ_0 is the reduced slenderness of the compressed element given by Eq. (5).

$$\lambda_0 = \sqrt{\frac{QA_g f_y}{N_e}}. \tag{5}$$

In the Eq. (5), N_e is the lowest between the critical buckling loads of column calculated for bending and torsion, according to Annex E of ABNT NBR 8800:2008 [3].

For I-sections under fire conditions, according to ABNT NBR 14323:2013 [2], the design axial load capacity is given by Eq. (6), if the section does not present local instabilities or through Eq. (7), otherwise.

$$N_{fi,Rd} = \chi_{fi} k_{y,\theta} A_g f_y. \tag{6}$$

$$N_{fi,Rd} = \chi_{fi} k_{\sigma,\theta} A_{ef} f_y. \tag{7}$$

In those equations, A_{ef} is the effective cross-sectional area provided in Annex F of the ABNT NBR 8800:2008 [3]; $k_{y,\theta}$ and $k_{\sigma,\theta}$ are the reduction factors for yield strength both given in Table 1; and χ_{fi} is the strength reduction factor under fire conditions given by Eq. (8).

Table 1. Reduction factors for yield strength as a function of temperature.

Temperature (ºC)	$k_{y,\theta}$	$k_{\sigma,\theta}$
20	1.00	1.00
100	1.00	1.00
200	1.00	0.89
300	1.00	0.78
400	1.00	0.65
500	0.78	0.53
600	0.47	0.30
700	0.23	0.13
800	0.11	0.07
900	0.06	0.05
1000	0.04	0.03
1100	0.02	0.02
1200	0	0

$$\chi_{fi} = \frac{1}{\varphi_{0,fi} + \sqrt{\varphi_{0,fi}^2 - \lambda_{0,fi}^2}}. \tag{8}$$

The parameters $\varphi_{0,fi}$, α and $\lambda_{0,fi}$ are given by Eq. (9), Eq. (10) and Eq. (11) respectively.

$$\varphi_{0,fi} = 0.5(1 + \alpha\lambda_{0,fi} + \lambda_{0,fi}^2). \tag{9}$$

$$\alpha = 0.022\sqrt{\frac{E}{f_y}}. \tag{10}$$

$$\lambda_{0,fi} = \frac{\lambda_0}{0.85}. \tag{11}$$

In the above equations, E is the modulus of elasticity of the steel and λ_0 is the reduced slenderness given by Eq. (5).

4 Structural reliability

The structural reliability aims to evaluate the safety level of the structures through the consideration of the uncertainties associated with the variables involved in the analyzed problem. For this purpose, the basic performance requirements of the structure are represented by limit state equations (failure functions). These equations involve several variables that are associated with uncertainties, which can be considered through probabilistic distributions and statistical parameters. In this way, it is possible to obtain a quantitative measure associated with the safety of the structure - the failure probability.

Let $G(\boldsymbol{X})$ be a failure function, in which $\boldsymbol{X}$ represents the vector of random variables considered in the analysis. For each failure mode of a structure, the $G(\boldsymbol{X})$ function separates the failure domains Ω_f and the survival domains Ω_s from the structure where positive values of $G(\boldsymbol{X})$ represent safe events and the $G(\boldsymbol{X}) \leq 0$ condition indicates failure events. The failure probability can be obtained by the integration of the joint probability density function of the random variables of the problem over the failure domain, according to Eq. (12).

$$P_f = \int_{\Omega_f} f_X(\boldsymbol{X})\, dx. \tag{12}$$

However, according to Santos and Gouveia [10], in most applications is too complex to evaluate analytically the Eq. (12). Thus, numerical methods are generally employed to conduct the reliability analysis. In this work the First Order Reliability Method (FORM) and the Monte Carlo simulation were used. Both are briefly described below. A more detailed basis can be found in Beck [11], Ditlevsen and Madsen [12] and Ang and Tang [13].

4.1 First Order Reliability Method (FORM)

FORM is an analytical method based on the linearization of the limit state equation of the problem and on the transformation of the original random variables into equivalent normal random variables. This method, according to Silva and Lima Júnior [14] presents as a great advantage the ability to use all the statistical information of the random variables of the problem, being possible to deal with any statistical distributions and allowing the consideration of correlations between variables. The reliability problem is formulated as a nonlinear optimization problem with restriction, in which the design point is sought. This point, defined in the transformed space of the equivalent normal variables, contains the values of the random variables that have the highest probability of leading to the failure of the structure. From this definition, the reliability index (β) represents the shortest distance between the origin of the transformed space and the limit state equation.

The failure probability can be approximated as a function of the reliability index β through the cumulative distribution function of the standard normal variables $\Phi()$ according to Eq. (13), which can be found in Beck [11].

$$P_f = \Phi(-\beta). \tag{13}$$

In addition to the failure probabilities and reliability indexes, the FORM allows the determination of sensitivity indexes, which are indicative of the relative contribution of each random variable in the composition of the probability of failure. Among these indexes, the importance factor used in this work is highlighted.

4.2 Monte Carlo simulations

According to Ang and Tang [13], simulation methods are processes that seek to reproduce the real world based on a number of hypotheses and models that represent reality. One of the most used simulation methods is the Monte Carlo simulation, since it presents robustness, simplicity and flexibility in its application. The method is based on the generation of n random events followed by the verification of the failure occurrence for each event, based on the indicator function given by Eq. (14). Then, the failure probability can be estimated by the ratio of the number of failure events to the total number of events.

$$I(\boldsymbol{X}) = \begin{cases} 1 \text{ if } \boldsymbol{X} \in \Omega_f \\ 0 \text{ if } \boldsymbol{X} \notin \Omega_f \end{cases}. \quad (14)$$

In the Eq. (14) $I(\boldsymbol{X})$ is the indicator function, $\boldsymbol{X}$ is the vector that contains the random variables and Ω_f is the failure domain.

The accuracy of the results generated by Monte Carlo simulation depends on the number of scenarios tested. For problems that present low failure probabilities, it is necessary to perform a large number of simulations in order to achieve an adequate response, which implies a large computational demand.

5 Methodology

5.1 Mechanical module

The mechanical module was implemented in Microsoft Excel® software, presenting the functions of modeling the temperature-time curves according to the standard and natural fire models, performing the thermal analysis and obtaining the values of the structure axial load capacity at normal temperature and under fire conditions, according to the recommendations of ABNT NBR 8800:2008 [3] and ABNT NBR 14323:2013 [2]. This module was validated based on results from Silva [5], Ricardo [15] and Buchanan and Abu [16].

5.2 Application: Protected steel column exposed to compartment fire

The reliability analysis was performed in a steel column with yield strength (f_y) of 250 MPa and a height of 3 m with a cross-section of CS 250 x 52 as shown in Fig. 2. The structure was designed at normal temperature to support a compressive axial load containing both live (N_q) and dead (N_g) load components of 250 kN and 400 kN, respectively. The column was considered to belong to a hypothetical 20 m high commercial building with an area of 300 m^2 on the largest floor, which results in a Required Fire Resistance Time (*TRRF – Tempo Requerido de Resistência ao Fogo*) of 60 minutes, according to ABNT NBR 14432:2000 [6]. In addition, the material present in the room was admitted as mainly cellulosic, being considered the existence of alarms, independent water supplies and safe access routes, since these fire fighting measures are commonly found in Brazilian buildings.

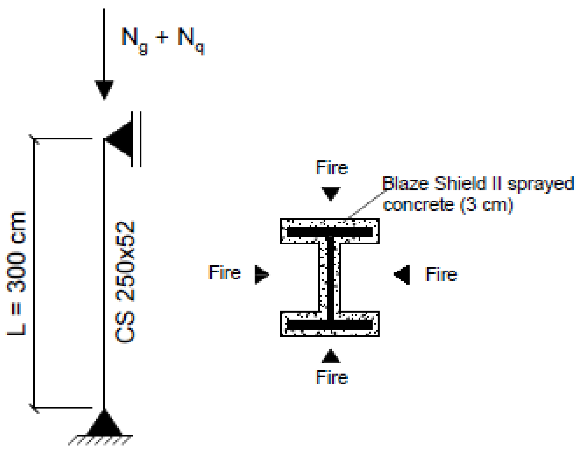


Figure 2. Protected steel column exposed to fire.

The standard and natural fire curves were used for fire modeling. A fire load of 564 MJ/m^2 was adopted based on Guo *et al.* [17]. The room considered for fire modeling was a room 4 meters wide by

6 meters long and 3 meters high with an opening of 2 m x 1.5 m and another one of 2 m x 2 m.

The fire protection material adopted was the Blaze Shield II sprayed concrete, whose thermal properties were adopted based on Guimarães [9] with thermal conductivity, specific heat and density equal to 0.15 W/mºC, 2300 J/kgºC and 240 kg/m^3, respectively. It was designed with a thickness of 3 cm.

5.3 Statistics of random parameters

Some of the parameters that significantly affect the fire design of steel columns were chosen as random variables, and their means, COV and distribution types were obtained based on the literature indicated and they are summarized in Table 3. The standard deviation can be obtained by multiplying the mean by the coefficient of variation (COV).

Table 3. Statistical data for the random parameters.

Parameter	Distribution	Mean	COV	Reference
Fire load density ($q_{f,k}$)	Gumbel	564 MJ/m^2	0.62	[17]
Thickness of fire protection material (t_m)	Lognormal	31.5878 mm	0.20	[18]
Thermal conductivity of fire protection material (λ_m)	Lognormal	0.15 W/mºC	0.24	[17]
Dead load (N_g)	Normal	420 kN	0.10	[17]
Live load (N_q)	Gumbel	250 kN	0.25	[19]
A_a	Normal	1.0	0.04	[20]
B_a	Normal	1.0	0.20	[20]
E_a	Normal	1.0	0.05	[20]
Yield strength (f_y)	Lognormal	257.5 MPa	0.063	[21]

Guo *et al.* [17] indicates that the nominal value of the dead load is multiplied by a factor of 1.05 to give the corresponding mean. Similarly, Hamilton [21] recommends the multiplication of the yield strength by a factor of 1.03.

The random parameters A_a, B_a and E_a are indicated by Ravindra and Galambos [20] and take into account the variability of the loads, being used in the composition of the load demand according to Eq. (15).

$$S = E_a (A_a N_g + B_a N_q). \tag{15}$$

On the other hand, the resistance R is a function of the other variables presented in Table 3, as indicated by Eq. (16).

$$R = R(q_{f,k}, t_m, \lambda_m, f_y). \tag{16}$$

Since $\boldsymbol{X}$ is the vector that contains the random variables of the problem, the limit state equation $G(\boldsymbol{X})$ can be written according to Eq. (17).

$$G(\boldsymbol{X}) = R - S. \tag{17}$$

The evaluation of Eq. (17) in the reliability analyses was made by a coupling between the mechanical module presented in 5.1 and the reliability module, which consisted of a set of subroutines programmed in MATLAB [22].

5.4 Performed reliability analysis description

First, the structure presented in item 5.2 was analyzed at normal temperature using FORM. Through this analysis it was determined the failure probability of the structure and the corresponding reliability index. From this method, the sensitivity indexes for the random variables of the problem were also obtained.

After that, a convergence analysis was performed to determine the necessary number of simulations to obtain an adequate result in the Monte Carlo method, which was used to analyze the structural reliability of the steel column exposed to fire and obtain the failure probability as a function of time of fire exposure, temperature, thickness of fire protection material, yield strength and load. Additionally, a sensitivity analysis was performed for different times of the fire.

The reliability methods mentioned were used through subroutines developed in MATLAB [22], and the application of these to structures under fire conditions was validated using the results obtained by Ricardo [15].

6 Results and discussion

6.1 Structure at normal temperature

The failure probability of the structure analyzed at normal temperature and the corresponding reliability index, determined via FORM, were equal to $10^{-4.40565}$ and 3.9486, respectively. Table 4, extracted from the JCSS [23], presents target reliability indexes, as well as the corresponding failure probabilities as a function of the cost of safety and of the consequences of the failure. It can be noted that the reliability index can be considered acceptable if it is in the range of 3.1 to 4.7. In this way, the value obtained is adequate and the structure presents a satisfactory safety level for normal use conditions.

Table 4. Target reliability indexes according to the JCSS.

Cost of safety	Ultimate Limit State			Serviceability Limit State
	Consequences of the failure			
	Minor	Moderate	Large	
Large (A)	$\beta = 3.1$ ($p_F \approx 10^{-3}$)	$\beta = 3.3$ ($p_F \approx 5x10^{-4}$)	$\beta = 3.7$ ($p_F \approx 10^{-4}$)	$\beta = 1.3$ ($p_F \approx 10^{-1}$)
Normal (B)	$\beta = 3.7$ ($p_F \approx 10^{-4}$)	$\beta = 4.2$ ($p_F \approx 10^{-5}$)	$\beta = 4.4$ ($p_F \approx 5x10^{-6}$)	$\beta = 1.7$ ($p_F \approx 5x10^{-2}$)
Small (C)	$\beta = 4.2$ ($p_F \approx 10^{-5}$)	$\beta = 4.4$ ($p_F \approx 5x10^{-6}$)	$\beta = 4.7$ ($p_F \approx 10^{-6}$)	$\beta = 2.3$ ($p_F \approx 10^{-2}$)

The graph shown in Fig. 3 presents the result of the sensitivity analysis for the structure at normal temperature. It is observed that the live load (N_q) and the B_a parameter are the variables with the highest importance factors, which can be explained by the statistical behavior of these variables, which presents a large dispersion in comparison to the others, making them cause greater disturbances in the response. Since the fire load and the thickness and thermal conductivity of the fire protection material are not considered in this analysis, they present no contribution to the composition of the failure probability.

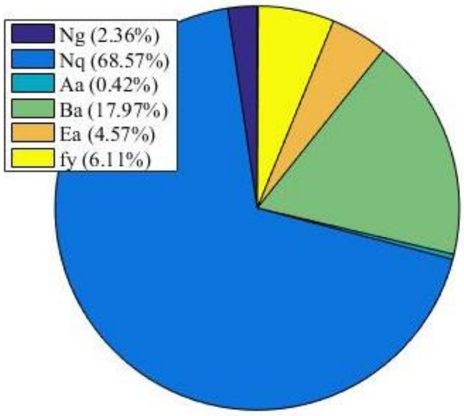


Figure 3. Sensitivity indexes for the structure at normal temperature.

6.2 Structure under fire conditions

The results of the convergence analysis are presented below. This analysis is based on the oscillation of the mean and variance of the failure probabilities, which constitute a confidence interval (i.c.), as the number of simulations used in the Monte Carlo method varies. In order to determine an adequate number of simulations for the application of the method, the convergence at the times of 30 and 60 minutes of fire was evaluated. Since structures under fire conditions are associated with high failure probability values, in both cases the probability calculated for 1000 simulations was taken as exact. The results are presented in Fig. 4.

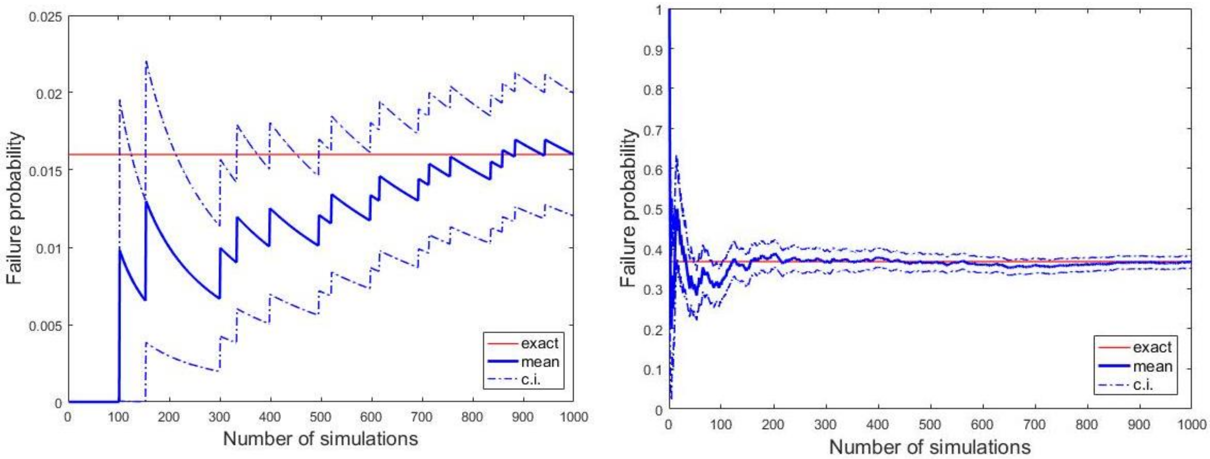


Figure 4. Convergence graphs of the failure probability and confidence interval for 30 (on the left) and 60 (on the right) minutes of fire.

According to Fig. 4, it can be noted that for 30 minutes of fire, the exact failure probability corresponded to 1.60%. Furthermore, for a number of simulations greater than 500, the difference between the exact failure probability and the average is less than 0.50%. Similarly, for 60 minutes of fire, the exact failure probability was 36.80%. In this case, starting from 300 simulations, the mean is almost equal to the exact value. Therefore, in order to obtain an adequate result that did not lead to a large computational demand, it was opted for using 500 simulations.

The variation of the failure probability with the time of fire exposure according to the standard and natural fire models is presented in Fig. 5. The analysis was performed up to 80 minutes because the temperatures of the natural fire curve reached the maximum value approximately at that time. For the 60-minute TRRF, the failure probabilities were 15% for the standard fire model and 39.40% for the natural fire model. On the other hand, for 80 minutes of fire, the probabilities reached the values of 40.80% and 66% for the standard and natural fire models, respectively.

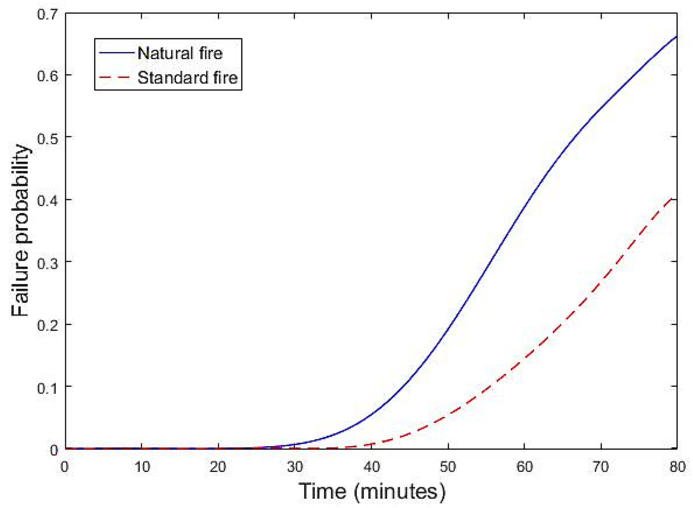


Figure 5. Failure probability as a function of time of fire exposure for standard and natural fire models.

Besides, it can be noted that the failure probability starts to grow faster around 30 minutes for the natural fire and 40 minutes for the standard fire. The differences observed result from the fact that the standard fire curve does not take into account in its modeling characteristics such as the fire load, the thermal inertia of the environment and the opening factor, which are considered in the modeling made by the natural fire curve. Consequently, the variation in temperature and the failure probability tend to be underestimated by the standard fire model.

The failure probability as a function of the steel temperature is shown in Fig. 6. For values under 300 ºC, it is noted that the chance of collapse of the structure is practically nil. However, between 300 and 400 ºC, the failure probability grows faster, reaching 100% to a temperature of approximately 600 ºC. This result is consistent with the observation made by Silva [5], who states that most steel structures have a critical temperature between 500 and 700 ºC.

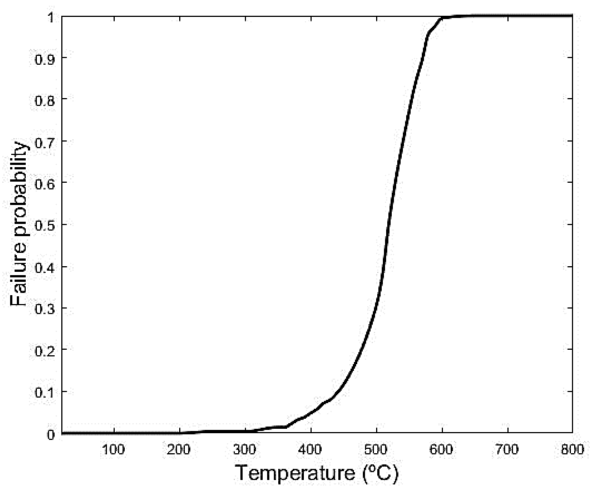


Figure 6. Failure probability as a function of steel temperature.

Figure 7 shows the failure probability as a function of the thickness of fire protection material. Curves referring to 20, 40, 60 and 80 minutes of fire were obtained. In all of them the reduction of the failure probability with the increase of the thickness is observed, as expected. For 60 minutes, which corresponds to the TRRF of the analyzed problem, it can be noted that the failure probability for the thickness of 30 mm adopted is 48.80%, which indicates a low safety level for this instant. Only over 60 mm thickness, the failure probabilities assume values less than 1%. This indicates that a thickness at least twice greater would be required to ensure the safety of the structure at the time indicated by the standard.

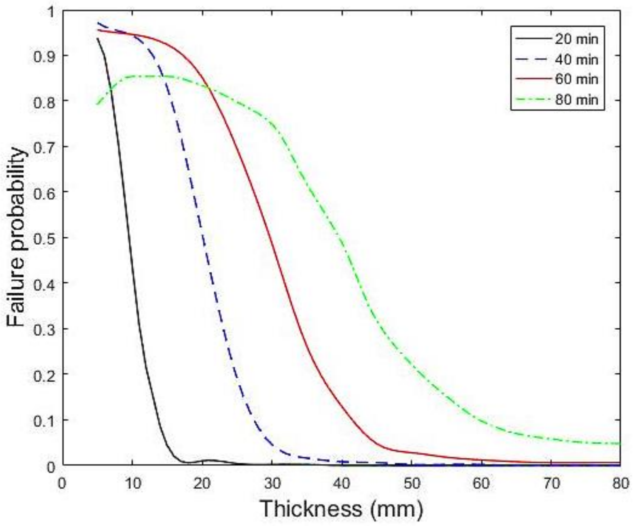


Figure 7. Failure probability as a function of the thickness of fire protection material.

The influence of the yield strength on the failure probability of the analyzed column can be visualized in Fig. 8. It is visible the existence of a discontinuity in the curves for yield strengths close to 250 MPa. This peculiarity is caused by the occurrence of local instabilities in the plates that compose the cross-section of the structure. These, when demanded by compressive or bending forces, are subject to compressive stresses and, consequently, to instability. The increase in yield strength makes the slenderness limit indicated by ABNT NBR 8800:2008 [3] for which the instabilities should be considered in the design is reduced. For this case, starting from about 250 MPa, the slenderness of the plates becomes higher than this limit, causing the resistance to be reduced and the failure probability to be increased. Moreover, it can be observed that the variation in the failure probability is less sensitive to changes in the yield strength for initial instants of fire.

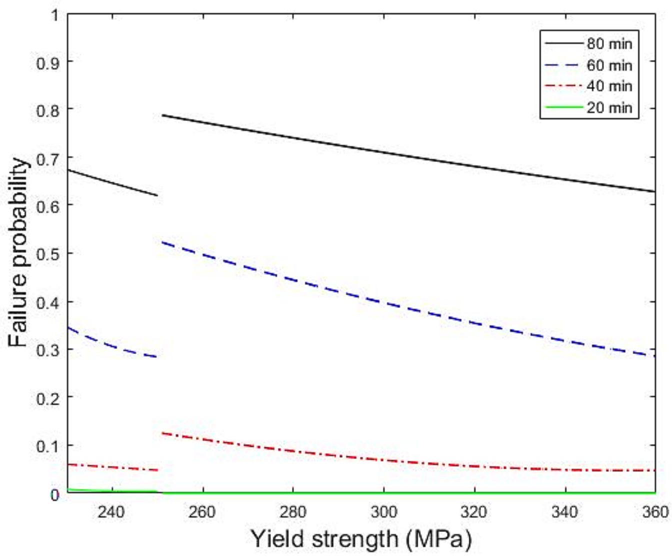


Figure 8. Failure probability as a function of the yield strength of the steel.

Figure 9 shows the failure probability as a function of the load applied to the column for the instant of 60 minutes of fire. The deterministic resistance of the structure obtained for this time was 761.35 kN.

However, according to Fig. 9, the failure probability has a greater increase from a load of about 200 kN, reaching 48% to 750 kN and 100% at approximately 1200 kN.

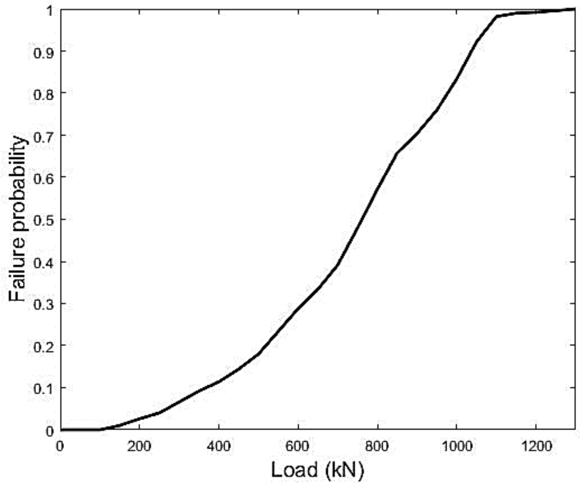
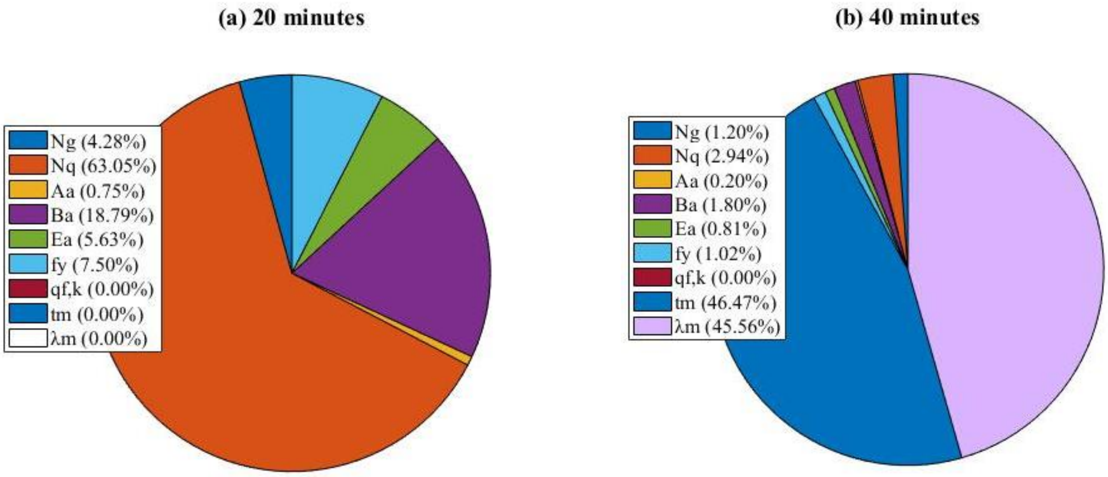
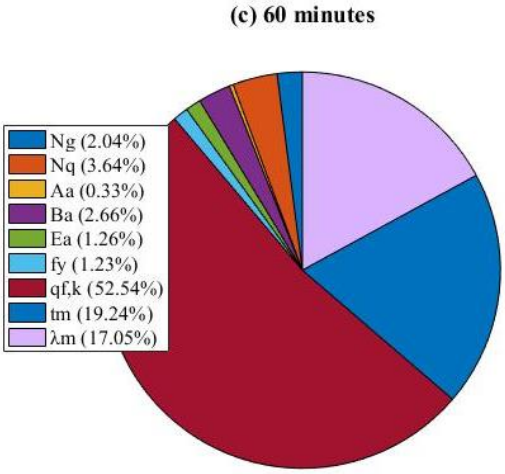


Figure 9. Failure probability as a function of the applied load.

Finally, in Fig. 10 (a) to (d) are the results of the sensitivity analysis for the instants of 20, 40, 60 and 80 minutes of fire. For 20 minutes, the variables associated with the structural analysis of the problem are those with higher contribution to the failure probability, with emphasis on the live load (N_q) and the B_a parameter. The result is similar to that obtained for the structure at normal temperature (Fig. 3), since, at this moment, the temperature that causes loss of resistance in the steel has not yet been reached. On the other hand, according to Fig. 10 (b), for 40 minutes of fire, the most significant variables are the thickness (t_m) and thermal conductivity (λ_m) of the fire protection material. This occurs because the temperatures for which the steel resistance is reduced start to be reached. As the fire protection material is the element responsible for the heat transfer between the external environment and the structure, the variables associated with it become more important.





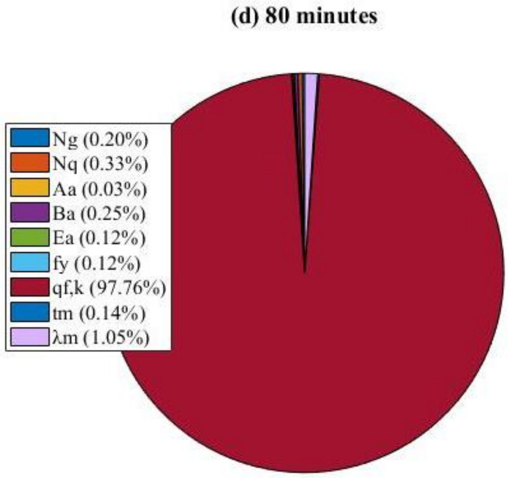


Figure 10. Sensitivity indexes for (a) 20, (b) 40, (c) 60 and (d) 80 minutes of fire.

From Fig. 10 (c), it can be observed that the fire load ($q_{f,k}$) presents greater influence on the problem for 60 minutes of fire, followed by the thermal conductivity and the thickness of the fire protection material. Furthermore, for 80 minutes (Fig. 10 (d)) the fire load has a importance factor that is much higher than the other variables. This can be explained by the fact that the maximum temperature of the natural fire curve is reached in approximately 80 minutes, and its value is a function of the fire load of the room. Another point that contributes to this result is the high dispersion presented by the fire load in relation to the other variables.

7 Conclusions

In this study, the reliability methods FORM and Monte Carlo were applied to a steel column under fire conditions by the coupling between mechanical and reliability computational modules. Nine random variables associated with fire modeling and thermal and structural analysis were considered.

The analysis of the structure at normal temperature via FORM indicated that it presented adequate safety levels according to the literature. The application of the Monte Carlo method to the structure under fire conditions showed that the standard fire curve, recommended by the Brazilian standard, underestimates the steel temperature variation for this case, when compared to the natural fire curve, since the standard curve does not consider important variables, such as the fire load. Additionally, for the 60-minute TRRF, failure probabilities of 39.40% for natural fire and 15% for standard fire were verified.

The results of the sensitivity analysis made it possible to evaluate how the influence of the random variables of the problem changes with the time of fire exposure. In initial instants, it was observed that the variables associated with structural analysis have greater importance. However, as time passes, the thermal variables start to have a greater contribution in the failure of the structure.

It is important to note that the values of failure probabilities obtained according to the Monte Carlo method are considerably higher than those recommended in the literature, which theoretically indicates an inadequate safety level. Nevertheless, for a more careful analysis, the probability of occurrence of fire, which is generally low, should be taken into consideration. However, this analysis was not part of the scope of this work, being the evaluation of structural safety made only based on the failure probabilities of the structural element.

Considering the above, it is clear the importance of the application of reliability to structural designs, especially those exposed to exceptional conditions. As observed, although the application of standards leads to projects with adequate safety levels for normal conditions of use, the same may not occur for the cited cases, because the uncertainties play an important role in the problem and should be carefully analyzed.

It is also worth mentioning that the uncertainty in the mechanical model was not discussed. The

Beck and Dória's [24] paper (Reliability Analysis of I-Section Steel Columns Designed According to New Brazilian Building Codes) throws light on a situation in which safety is already compromised in the field of stress analysis of the section depending on the relationship between live and dead loads. This reinforces the care to be taken in a delicate analysis such as fire.

Despite this, the Brazilian standards for structural design still do not address recommendations about reliability in their scope. This methodology should therefore be used carefully and in a complementary way to the design recommended by the normative codes, in order to provide measures related to structural safety.

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