IBRACON Structures and Materials Journal
Revista IBRACON de Estruturas e Materiais
ISSN 1983-4195 ismj.org

ORIGINAL ARTICLE

Reliability analysis of simply supported RC beam cross-sections in a fire situation according to Brazilian standards

Análise de confiabilidade de seções transversais de vigas simplesmente apoiadas de CA em situação de incêndio de acordo com normas brasileiras

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Received 24 April 2023
Accepted 29 July 2023

Abstract: This article aims to evaluate the reliability of cross-sections of simply supported reinforced concrete (RC) beams in a fire situation considering the utilization of the simplified method proposed in the Brazilian standard NBR 15200 [1]. The impact of the concrete cover, the live-to-total load ratio, and the quantity of heated beam faces were all taken into account in this analysis. Besides, a sensitivity analysis is performed showing the random variables that are of major impact on reliability. In the end, it was concluded that the number of heated faces of the beam is very important to the safety of these structures. Also, it is found that a higher concrete cover can promote an increase in reliability and that a bigger proportion of live load relative to the total load generates a decrease in the probability of failure. Regarding the sensitivity analysis, the fire temperature and the concrete cover were the variables with the greatest observed impact.

Keywords: fire, reliability, reinforced concrete beams.

Resumo: Este artigo tem como objetivo avaliar a confiabilidade de seções transversais de vigas de concreto armado (CA) simplesmente apoiadas em situação de incêndio, considerando a utilização do método simplificado proposto na norma brasileira NBR 15200 [1]. O impacto do cobrimento do concreto, a relação carga variável/carga total e a quantidade de faces aquecidas da viga foram levados em consideração nesta análise. Além disto, uma análise de sensibilidade é feita mostrando as variáveis aleatórias que apresentam maior impacto na confiabilidade. Ao final, concluiu-se que o número de faces aquecidas da viga é muito importante para a segurança dessas estruturas. Além disso, verifica-se que um maior cobrimento de concreto pode promover um aumento na confiabilidade e que uma maior proporção de carga variável em relação à carga total gera uma diminuição na probabilidade de falha. Em relação à análise de sensibilidade, a temperatura do incêndio e o cobrimento de concreto foram as variáveis de maior impacto observado.

Palavras-chave: incêndio, confiabilidade, vigas de concreto armado.

How to cite: R. M. Pires and H. M. Gomes, "Reliability analysis of simply supported RC beam cross-sections in a fire situation according to Brazilian standards," *Rev. IBRACON Estrut. Mater.*, vol. 17, no. 4, e17403, 2024, https://doi.org/10.1590/S1983-41952024000400003

1 INTRODUCTION

The occurrence of fires in buildings and their consequences create the need to carry out more in-depth studies regarding the resistance of structures and their elements when subjected to high temperatures. In these situations, it must be ensured, above all, that slabs, beams, and columns do not collapse and that they last long enough for the occupants of the buildings to be rescued or to abandon the construction properly. Secondarily, structural safety aims to protect property, as this type of accident can cause great loss to the buildings' owners. So, quantifying the probability of failure (or its counterpart, the reliability) of such elements is of utmost importance for the proper design.

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Financial support: The authors would like to acknowledge the financial support of CAPES.
Conflict of interest: Nothing to declare.
Data Availability: The data that support the findings of this study are available from the corresponding author, R. M. Pires, upon reasonable request.

In this context, in order to ensure reasonable levels of safety or reliability for the construction, the structural engineer must know the response of a given structure to a fire, a task that can be very difficult because there are too many uncertainties related to this problem, such as the site in the structure where the fire starts, what is the fuel load present in these places, how the air flow connects the different compartments of the building, the temperature that the fire will reach and how this temperature can influence the resistance of the structural components, the dimensions of these components, the parameters of the beams, the calculation model used, the loads acting, just to name a few.

In Brazil, it is known that a great number of buildings are constructed using reinforced concrete (RC) technology. In addition, much of the design methodology addressed by standards for fire situations is based on scarce studies on the subject. For these reasons, it is perceived that there is a need to study the reliability of reinforced concrete elements in fire situations, mostly due to the unpredictability and uncertainties above mentioned, aiming to develop a better understanding of the phenomenon and to find measures that can improve the safety of buildings in case of fire.

Therefore, the main objective of this article is to access the reliability, in the ultimate limit state, of cross-sections of reinforced concrete beams in a fire situation, considering the intrinsic variability of important parameters that govern the situation of a fire in a structure. The conducted analysis could conclude what are the analyzed parameters that most influence the reliability of the elements. Besides, this study will consider the recommendations and determinations of the Brazilian standards related to the subject, especially NBR 15200 [1] and some topics of NBR 6118 [2] and NBR 6120 [3].

2 BRAZILIAN STANDARDS

The main Brazilian standard that deals with the design and detailing of reinforced concrete elements is NBR 6118 [2]. Besides, to design a given structure, NBR 6120 [3] should also be used because it determines the loads to be considered in each situation. However, the key recommendations, when the topic is RC structures in fire situations, come from NBR 15200 [1] and NBR 14432 [4]. It is known that a fire situation decreases the resistance of both steel and concrete because of the high temperatures developed in the cross-section of the beams. Therefore, NBR 15200 [1] establishes the compressive strength of concrete as a function of the temperature reached by the material, as shown in Equation 1,

$$f_{c\theta} = k_{c,\theta} f_{ck} \tag{1}$$

where $f_{c\theta}$ is the compressive strength of concrete at a given temperature, $k_{c,\theta}$ is the reduction factor of resistance and f_{ck} is the characteristic compressive strength of concrete at ambient temperature. The reduction factors corresponding to each temperature are determined by Table 1.

Table 1. Reduction factors of the compressive strength of concrete (NBR 15200 [1]).

T (°C)	20	100	200	300	400	500	600	700	800	900	1000	1100	1200
$k_{c,\theta}$	1.00	1.00	0.95	0.85	0.75	0.60	0.45	0.30	0.15	0.08	0.04	0.01	0.00

It is important to make it clear that, according to NBR 15200 [1], these reduction factors refer only to concretes with siliceous aggregates and that the standard allows linear interpolations for temperatures in between the established values. Another modification that high temperatures impose on concrete is the change in the stress-strain diagram, as Figure 1 shows.

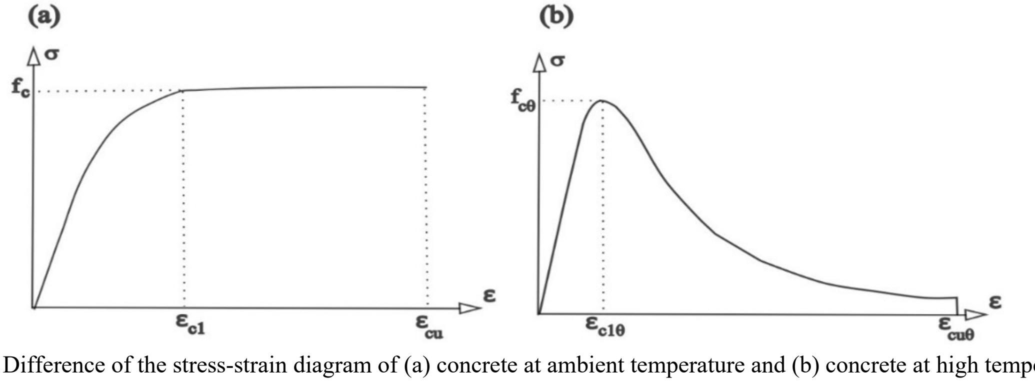

Figure 1. Difference of the stress-strain diagram of (a) concrete at ambient temperature and (b) concrete at high temperatures according to NBR 15200 [1].

As noticed, at ambient temperature, it is considered a simplification of the stress-strain diagram, called the parabola-rectangle diagram. At high temperatures, the behavior of the concrete is modified, characterized by a softening diagram, meaning that an increase in the strain after achieving the maximum stress corresponds to a decrease in the concrete stress. As this article aims to study the situation in fire, it is interesting to determine the values of the ultimate strain of the concrete for this case. Table 2 indicates these values for different temperatures according to NBR 15200 [1].

Table 2. Ultimate strain of the concrete for different temperatures (NBR 15200 [1]).

T (°C)	20	100	200	300	400	500	600	700	800	900	1000	1100
$\epsilon_{cu\theta}$(‰)	2.00	2.25	2.50	2.75	3.00	3.25	3.50	3.75	4.00	4.25	4.50	4.75

Concerning the influence of high temperatures on steel, the Brazilian standard also considers a reduction in the yield strength of steel given as indicated by Equation 2.

$$f_{y,\theta} = k_{s,\theta} f_{yk} \tag{2}$$

where $f_{y,\theta}$ is the yield strength of steel for a certain temperature, $k_{s,\theta}$ is the reduction factor and f_{yk} is the characteristic yield strength of steel at ambient temperature. A similar effect is observed in the modulus of elasticity of the steel, as can be seen in Equation 3.

$$E_{s,\theta} = k_{Es,\theta} E_s \tag{3}$$

where $E_{s,\theta}$ is the modulus of elasticity of the steel for a certain temperature, $k_{Es,\theta}$ is the reduction factor and E_s is the modulus of elasticity of the steel at ambient temperature, considered equal to 210 GPa according to NBR 6118 [2]. The reduction factors mentioned above are summarized in the Table 3 considering the steel temperature.

Table 3. Steel reduction factors f_{yk} and E_s (NBR 15200 [1]).

T (°C)	$k_{s,\theta}$			$k_{Es,\theta}$	
	Tension		Compression CA-50 or CA-60	CA-50	CA-60
	CA-50	CA-60			
20	1.00	1.00	1.00	1.00	1.00
100	1.00	1.00	1.00	1.00	1.00
200	1.00	1.00	0.89	0.90	0.87
300	1.00	1.00	0.78	0.80	0.72
400	1.00	0.94	0.67	0.70	0.56
500	0.78	0.67	0.56	0.60	0.40
600	0.47	0.40	0.33	0.31	0.24
700	0.23	0.12	0.10	0.13	0.08
800	0.11	0.11	0.08	0.09	0.06
900	0.06	0.08	0.06	0.07	0.05
1000	0.04	0.05	0.04	0.04	0.03
1100	0.02	0.03	0.02	0.02	0.02
1200	0.00	0.00	0.00	0.00	0.00

Note that the Brazilian standard determines the reduction factors for two types of steel (CA-50 and CA-60). According to Araújo [5], CA-50 refers to ribbed steel bars obtained by hot rolling that have a minimum diameter equal to 6.3 mm. On the other hand, CA-60 refers to steel wires obtained by drawing with a maximum diameter equal to 10 mm. In structural projects in Brazil, it is common to use CA-60 steel for stirrups and CA-50 steel for longitudinal reinforcement bars.

Another important topic covered by NBR 15200 [1] are the structural verification methods for reinforced concrete structures subject to fire. This verification must be made only for the ultimate limit state considering an exceptional combination of load given in Equation 4.

$$F_{d,fi} = \gamma_g F_{gk} + F_{qexc} + \gamma_q \sum_2^n \psi_{2j} F_{qjk} \tag{4}$$

where $F_{d,fi}$ is the exceptional combination due to the fire, γ_g is partial safety coefficient for dead load, F_{gk} is the characteristic value of the dead load, F_{qexc} is the representative value of the exceptional loads of occurrence of a fire, γ_q is partial safety coefficient for live loads, ψ_{2j} is the combination factor, F_{qjk} is the characteristic value of the live loads. According to NBR 15200 [1] the values of the partial safety coefficients and combination factors in fire situations are $\gamma_g = 1.2$, $\gamma_q = 1.0$ and $\psi_{2j} = 0.3$ or 0.4. The combination factor varies if the construction is for residential or commercial use. In the first case, it equals 0.3, and in the second case, 0.4. The NBR 15200 [1] also mentions that in fire situations, the combination factor may be reduced by a multiplying factor of 0.70.

It is important to note that F_{qexc}, which refers to the load action of the fire, is not considered the same way that live and dead loads are. In fact, considering the fire situation, this loading corresponds to the decrease of the resistance of the concrete and steel by utilizing the reduction factors shown in Table 1 and Table 3 or using another method (allowed by the standard) to evaluate the design resistance. That way, the usual verification of a reinforced concrete structure results in Equation 5, where $S_{d,fi}$ represents the design effect of actions for the fire situation.

$$S_{d,fi} = \left(\gamma_g F_{gk} + \gamma_q \sum_2^n \psi_{2j} F_{qjk}\right) \leq R_{d,fi}[f_{c\theta}, f_{y,\theta}] \tag{5}$$

As can be seen, to calculate the design resistance ($R_{d,fi}$), one must consider the compressive strength of concrete at a given temperature ($f_{c\theta}$) and the yield strength of steel for a certain temperature ($f_{y,\theta}$).

Another significant topic is that the action of the fire corresponds to a standard fire exposure time established by NBR 14432 [4], which is called Fire Resistance Required Time (TRRF). The standard fire follows Equation 6, where θ_g is equal to the temperature of the fire gases and t is the time in minutes.

$$\theta_g = 20 + 345 \log(8t + 1) \tag{6}$$

Concerning the verification methods mentioned on the NBR 15200 [1] to establish if the reinforced concrete beam is safe or not in a fire scenario, the standard refers to the following methods: (*i*) the tabular method, (*ii*) the simplified method, (*iii*) the advanced method, and (*iv*) the experimental method. These methods will be briefly described below, with a greater focus on the simplified method since it will be used in the reliability analysis that will be conducted in this work.

The tabular method of NBR 15200 does not require any calculation, it only determines, using tables, the minimum width that the beam needs to present and determines the minimum distance between the centroid of the longitudinal reinforcement and the face of the concrete exposed to fire. This determination is based on the fire resistance time required, which is given by NBR 14432 [4] according to the characteristics of the building or the analyzed construction.

The simplified method considers three hypotheses: (*i*) the fire load is assumed as shown in Equation 4; (*ii*) the ultimate load-bearing capacity of the element can be calculated from the temperature distribution of the cross-section for the fire resistance time required and this temperature distribution can be determined from the technical literature or calculated from computer programs; (*iii*) the ultimate load-bearing capacity can be calculated in the same way as for a normal situation (ambient temperature), but the average resistance for steel and concrete in a fire situation must be adopted. This is done by evenly distributing the concrete strength loss in the compressed part along the cross-section and the steel strength loss in the total area of the reinforcement.

Alternatively, according to the simplified method proposed by NBR 15200 [1], some methods consider that a reduced concrete section in a fire situation can be used to determine the ultimate load-bearing capacity, such as the 500º Isotherm Method, which will be discussed further in this article. In addition, the standard allows that in the analysis of reinforced concrete elements in a fire situation, the compressive strength of concrete is no longer modified by the long-term effect accounted by the Rüsch's coefficient (equal to 0.85), which is usually accounted for in normal situations due to the loss of resistance of the elements to long-term loads.

Regarding the other mentioned methods, the advanced methods perform a more accurate analysis of reinforced concrete structures in a fire situation, considering the non-linearities involved and the effects of thermal deformations, and the experimental method is exclusively based on results from experiments carried out on reinforced concrete elements.

3 METHODOLOGY

As previously mentioned, this article aims to carry out a reliability analysis of reinforced concrete beams to determine which beam parameters have a major influence on the safety of these structures in a fire situation. So, the main parameters involved in the analysis will be treated as random variables, which will be characterized by a probability distribution, with variability represented by the coefficient of variation and with a given bias factor.

From this, considering nominal values for each of these parameters, the FORM (First Order Reliability Method) will be used to determine the reliability indexes of the analyzed beams. This determination is based on the evaluation of the limit state function (LSF), which will consider whether the actual loading bending moment on the critical cross-section of the beam is greater or not than its ultimate resistant bending moment. Very costly tests with the Monte Carlo Simulation Method were performed for some of the examples evaluated in this work that resulted in negligible differences in the Probability of Failure obtained using the FORM Method, as reported in Pires [6].

It is important to point out that the ultimate resistance moment is calculated using the simplified method allowed by NBR 15200 [1], and that the influence of the high temperatures of the fire on the concrete is considered based on the use of the 500°C Isotherm Method. Likewise, such influence on the steel is considered from the use of the reduction coefficients shown in Table 3. These coefficients will be determined from the calculation of the temperature inside the cross-section in the steel bars, applying the Wickström [7] Method.

The consideration of the simplified method (500°C Isotherm Method) is justified by the fact that it was not intended to consider in deep the non-linearities involved in the analysis and the effects of constrained thermal deformations, which is characteristic of the advanced method according to NBR 15200 [1]. In addition, the use of a semi-empiric method (Wickström Method) in detriment to a Finite Element Analysis (FEA) to determine the temperature in the beam's cross-section is justified due to the large computational costs involved in FEA approaches, according to Eamon and Jensen [8]. The low computational costs of the Wickström Method allowed that the multiple parametric studies were presented in this work. Each LSF evaluation took less than a second, while a complete FEA for thermal diffusivity (with reasonable mesh refinement) may spend dozens of times more.

Next, the following topics covered in the methodology will be explained: structural reliability, FORM, Wickström Method, 500°C Isotherm Method, and the determination of the ultimate resistant bending moment. Subsequently, the hypotheses considered in the reliability analyses will be treated and finally, the results discussed.

4 STRUCTURAL RELIABILITY

When a given structural system does not respect certain requirements or behaves in an undesired way, it is said, from the point of view of reliability, that this system is in a failure situation. On the opposite, the system is said to be in a safe condition, which is sought by all designers. These two regions are delimited by the so-called limit state function, which shows the criterion that the structure under analysis must comply with, being evaluated from the values of the random variables involved in the problem.

In order to determine the probability of failure or safety of a structural system, it is necessary to know that its behavior will occur as a function of a vector ($\vec{\mathbf{X}}$) that encompasses the variables of the problem. From this, it is necessary to determine when the limit state function $g(\vec{\mathbf{X}})$, which depends directly on the vector of random variables, assumes positive, negative, or null values. It is defined that when $g(\vec{\mathbf{X}}) \leq 0$, the structure is assumed to fail concerning the established criterion. When $g(\vec{\mathbf{X}}) > 0$, the structure is assumed in a safe condition. Therefore, the probability of failure (P_f) is calculated from Equation 7 (Ang and Tang [9]).

$$P_f = \int_{g(\vec{\mathbf{X}})\leq 0} f_{\vec{X}}\left(\vec{\mathbf{X}}\right) d\vec{\mathbf{X}} \tag{7}$$

where $f_{\vec{\mathbf{X}}}$ is the joint probability density function of the random variables involved in the problem. The solution of Equation 7 is difficult to perform because most of the time the joint probability density function is not known or even due to the huge computational effort to evaluate the integral for limit state functions that take into account several random variables. Because of that, different methods can be used to determine the reliability of a certain structure or system.

In this context, there is the reliability index (β), which is a measure of safety for a given system, and the higher it is, the lower the probability of failure. This index relates to the probability of failure by Equation 8, where Φ is the standard normal cumulative distribution function.

$$P_f = \Phi(-\beta) \tag{8}$$

So, for a $\beta = 0$, the corresponding probability of failure is $P_f = 0.5$, for a $\beta = 5.2$, the corresponding probability of failure is $P_f = 1 \times 10^{-7}$, and so on.

5 FORM (FIRST ORDER RELIABILITY METHOD)

The FORM (First Order Reliability Method) is a method that linearizes the limit state function at the analyzed failure point, allowing the statistical information of the random variables involved in the problem under analysis to be considered, as well as the possible correlation between them. In general, the method transforms all variables to the standardized and uncorrelated space to perform the reliability analysis.

In this process, the reliability index is determined by calculating the shortest distance between the considered LSF and the origin of the standardized space of variables, as Figure 2 indicates for the case of two random variables. The point in the LSF for this shortest distance is called the Most Probable Point (MPP).

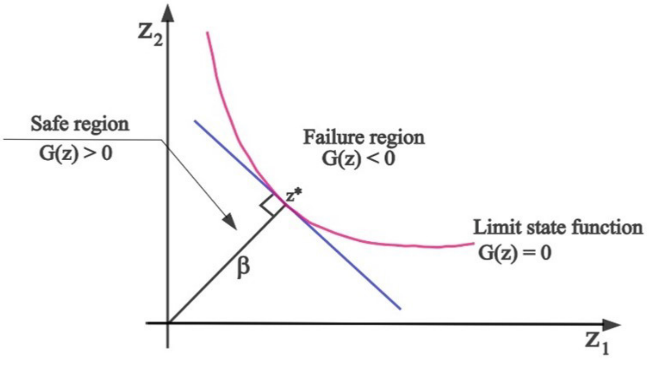

Figure 2. Determination of the reliability index by FORM.

6 THE WICKSTRÖM METHOD

Wickström's method was developed from a series of finite element analyses that constituted a cross-section of concrete in a fire situation. In these analyses, the temperatures of the steel bars and the concrete were determined as a function of time and the model considered the variable thermal conductivity for the reinforced concrete, the influence of water evaporation, and the existing non-linear boundary conditions according to Wickström [7].

By using these considerations, the author found that the method offers results very close to finite element approaches for regular cross-sections. In addition, Eamon and Jensen [8] compared the results from the model to results from the SAFIR software, which uses the finite element approach, and, in fact, only small percentage differences were observed.

Wickström's method covers the possibility of considering a one-dimensional or even a two-dimensional heat flow. In the first case, considering for example a beam being heated only on one of its faces or even on two parallel faces, the increase in temperature at the considered point ($\Delta\theta_{xy}$) is given by Equation 9.

$$\Delta\theta_{xy} = n_w n_x \Delta\theta_f \tag{9}$$

As for the two-dimensional heat transfer flow, considering the possibility of the beam being heated at 3 faces, the temperature rise at the considered point ($\Delta\theta_{xy}$) is given by Equation 10.

$$\Delta\theta_{xy} = \left(n_w\left(n_x + n_y - 2n_x n_y\right) + n_x n_y\right)\Delta\theta_f \tag{10}$$

where n_w is the ratio between the temperature increase on the surface of the structural element and the fire temperature increase, given by Equation 11, $n_{x,y}$ is the ratio between the internal temperature increase at the point of coordinates x and y and the temperature increase on the surface of the structural element, given by Equation 12 and $\Delta\theta_f$ is the fire temperature rise.

$$n_w = 1 - 0.0616t^{-0.88} \quad (11)$$

$$n_{x,y} = 0.18ln\left(\frac{\alpha_r t}{s^2}\right);\ s \geq 2h - 3.6(\alpha t)^{0.5} \quad (12)$$

where t is the elapsed time of the fire, in hours, α_r is the thermal diffusivity ratio with a reference value of 0.417×10^{-7} m^2/s, α is the thermal diffusivity of the analyzed beam, s is the distance from the point being analyzed in the concrete element up to the nearest heated surface of the cross-section, in meters, which must be limited to the maximum value indicated by Equation 12, where h is the dimension of the concrete cross-section in the considered direction (x or y).

By the analysis of Wickström's diagrams, the parameter n_w is lower limited to a value close to 0.45, which corresponds to a time of approximately 5 minutes. This means that for times less than 5 minutes, there is no change in the internal temperature of the concrete cross-section. Furthermore, the parameter $n_{x,y}$ is limited to a minimum value of 0.03.

7 THE 500°C ISOTHERM METHOD

According to Eamon and Jensen [8], the 500°C Isotherm Method was developed by the researcher Yngve Anderberg in 1978. This procedure considers a reduced cross-section, which is determined from the positioning of the isotherm (lines with equal temperature) of 500°C. Thus, the method assumes that up to this temperature, the concrete has 100% of its compressive strength and that at higher temperatures, the concrete has no strength at all.

Thus, it is noted that the use of this method depends directly on the determination of the position of the 500°C isotherm, which can be done from Equation 13 according to Purkiss [10], considering the use of the Wickström Method.

$$x_{500} = \sqrt{\alpha_r t/\exp\left(4.5 + \frac{480}{0.18 n_w T}\right)} \quad (13)$$

where α_r is the thermal diffusivity ratio considered for a reference value of 0.417×10^{-6} m^2/s, t is the time in hours, n_w is a parameter of the Wickstrom model and T is the fire temperature in °C. It is important to note that x_{500} is measured from the outer edge of the beam section, which means that for a fire that affects both lateral sides of the section, the effective width of the beam to be considered is equal to the original width reduced by two times x_{500}. On the other hand, when the fire affects only one of the lateral sides of the section, the original width must be discounted by x_{500} once.

It is important to note that the region that matters to calculate the ultimate bending moment of the beam is the effective compressive cross-section since it is not considered the tension strength of the concrete, as allowed by NBR 6118 [2]. Besides, in the compressive zone in positive bending moments, when the fire affects three faces, the heat that comes from the bottom of the structure has a neglectable influence in the determination of the temperature because of the large distance from the compressed zone to the bottom of the beam. Because of that, the position of the 500°C isotherm is calculated by Equation 13 also in a scenario of fire in three faces, which is confirmed by Eamon and Jensen's [8] procedure. The reduced cross-section is shown in Figure 3 for all considered fire scenarios.

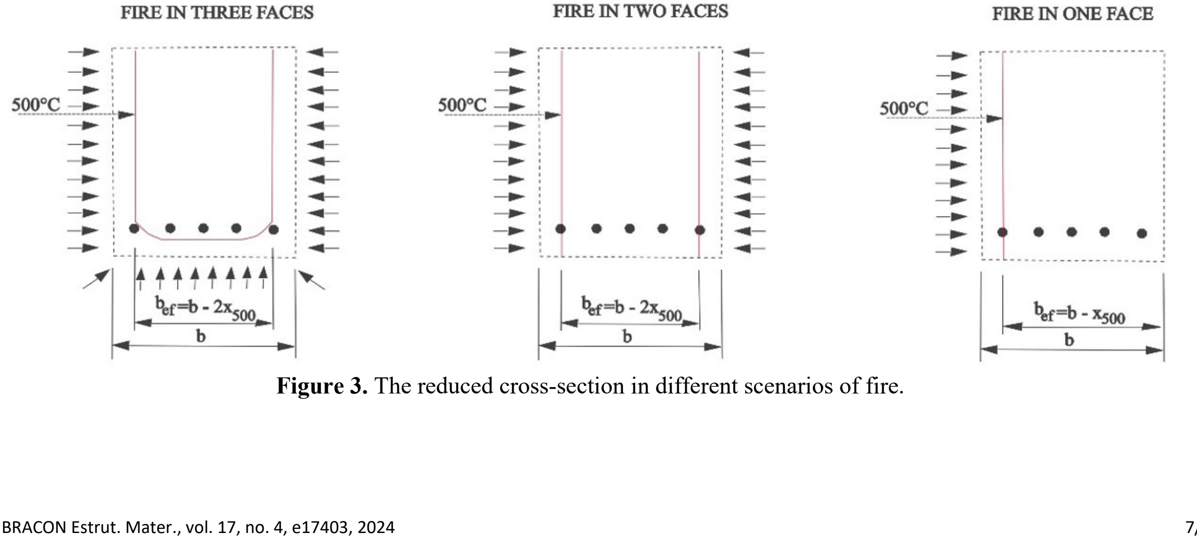

Figure 3. The reduced cross-section in different scenarios of fire.

When considering this method, it is possible that eventually some of the steel bars are positioned in a region outside the reduced cross-section, which means that these bars reach temperatures greater than 500°C. Even in these cases, these bars must also be considered in the calculation of the ultimate bending moment of the beam section, provided that a temperature-dependent reduction factor in resistance is applied.

8 ULTIMATE RESISTANT BENDING MOMENT

The ultimate resistant bending moment is determined for beams with rectangular cross-sections with upper and lower reinforcement, aiming at the possibility that the stress developed in the steel bars is lower than the yield stress, which can be considered depending on the values assumed for the random variables in the reliability analysis. Therefore, Figure 4 shows the situation being analyzed, with the dimensions and variables involved in the problem of determining the resistant bending moment of the beams.

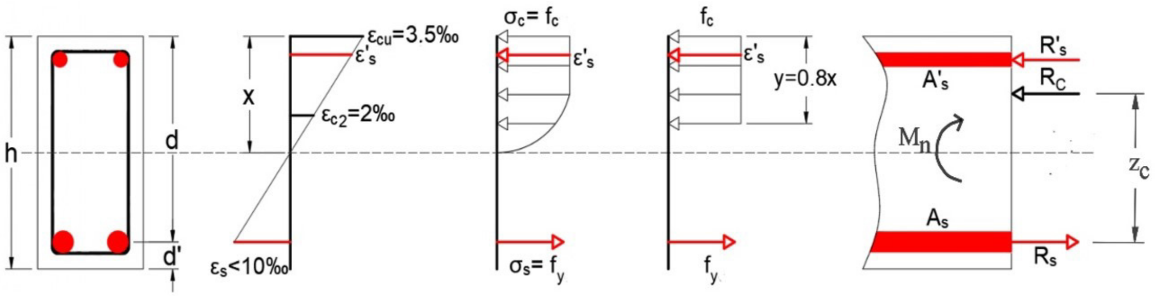

Figure 4. Determination of the ultimate resistant bending moment of a reinforced concrete beam according to Leite and Gomes [11].

From this illustration, Equation 14 and Equation 15 arise, which represent the balance of forces and the balance of moments in the section, respectively.

$$R_c + R_s' - R_s = 0 \tag{14}$$

$$M_n = R_c z_c + R_s'(d - d') \tag{15}$$

where R_c is the force resulting from the compressive stresses that occur in the concrete, R_s' is the force resulting from the compressive stresses in the upper reinforcement of the beam, R_s is the force resulting from the tensile stresses in the lower reinforcement of the beam, z_c is the lever arm of the resultant compression of the concrete in relation to the centroid of the lower reinforcement, d' is the distance from the most compressed edge of the section to the centroid of the upper rebar, and d is the distance from the most compressed edge of the section to the centroid of the lower rebar, also known as lower rebar position .

Knowing that the resultant compression of the concrete (R_c) is equal to $f_c b_w y$, where f_c is the compressive strength of the concrete, b_w is the width of the beam and y is the height of the block of the compressive stresses, that the resultant of the stresses of the reinforcements R_s' and R_s are equal to $A_s'\sigma_s'$ and $A_s\sigma_s$, where A_s' and A_s are the steel areas of the upper and lower reinforcement, respectively, and that σ_s' and σ_s are the stresses in the upper and lower reinforcement, respectively, the balance of forces is expressed by Equation 16 and the balance of moments is expressed by Equation 17.

$$f_c b_w y + A_s'\sigma_s' - A_s\sigma_s = 0 \tag{16}$$

$$M_n = f_c b_w y(d - y/2) + A_s'\sigma_s'(d - d') \tag{17}$$

The block height of compressive stresses (y) can be replaced by $0.8\beta_x d$, where β_x is the relative neutral axis depth given by the ratio x/d. This consideration is aligned with the simplification of the parabola-rectangle diagram of concrete stresses by a rectangular diagram, as permitted by NBR 6118 [2] for concretes with characteristic strength less than or equal to 50 MPa. Therefore, the equilibrium equations can be written as shown in Equation 18 and Equation 19.

$$0.8 f_c b_w \beta_x d + A'_s \sigma'_s - A_s \sigma_s = 0 \tag{18}$$

$$M_n = 0.8 f_c b_w \beta_x d^2 (1 - 0.4\beta_x) + A'_s \sigma'_s (d - d') \tag{19}$$

Another important concept related to the determination of the ultimate resistant bending moment is the relative neutral axis depth limit of the lower reinforcement (β_{lim}) and of the upper reinforcement (β'_{lim}), which represents the limit values for the cross-section to have a ductile failure. This means a failure characterized by intense cracking and deformation before failure, which is highly desirable from a design point of view and occurs when the reinforcement has reached yield strength. The relative neutral axis depth limit of the lower reinforcement is given by Equation 20, already considering the ultimate deformation of the concrete (ε_{cu}) and $\varepsilon_y = f_y / E_s$ is the yielding strain of the lower reinforcement.

$$\beta_{lim} = \varepsilon_{cu} \,/\, (\varepsilon_{cu} + \varepsilon_y) \tag{20}$$

For the upper reinforcement, the relative neutral axis depth limit (β'_{lim}) is determined as a function of the beam design domain. In case the beam was designed in domain 2, β'_{lim} is given by Equation 21.

$$\beta'_{lim} = (\eta + \frac{\varepsilon'_y}{0.01}) \,/\, (1 + \frac{\varepsilon'_y}{0.01}) \tag{21}$$

where η represents the ratio d'/d and $\varepsilon'_y = f_{yc}/E_s$ is the yield strain of the upper reinforcement, where f_{yc} is the yield stress of the compressed steel. If the beam is designed in domain 3, β'_{lim} is given by Equation 22.

$$\beta'_{lim} = \eta / (1 - \frac{\varepsilon'_y}{\varepsilon_{cu}}) \tag{22}$$

The steps to determine the ultimate resistant bending moment follows:

a) The value of the relative neutral axis depth (β_x) is calculated from Equation 18 by considering the steel has achieved the yield strength.

b) Check if $\beta_x \leq \beta_{lim}$ and $\beta_x \geq \beta'_{lim}$. In case it is true, calculate the ultimate resistant bending moment from Equation 19.

c) In case of step b) is not true, calculate the strain in the reinforcement according to the domain of design and then calculate the steel stress.

d) With the steel stress of letter c), reevaluate the relative neutral axis depth (β_x) from Equation 18 and calculate the ultimate resistant bending moment from Equation 19.

It should be emphasized that this guide for calculating the resistant bending moment will be used for all the specific times chosen to evaluate the reliability of the beams in a fire situation, which means that parameters like the ultimate deformation of the concrete (ε_{cu}) and the Elastic modulus of steel (E_s) will not be constant, since they show variability as the temperature increases as already mentioned.

9 CONSIDERED HYPOTHESES FOR RELIABILITY ANALYSES

Next, the hypotheses that were considered throughout the reliability analyses of reinforced concrete beams in a fire situation will be cited and explained.

9.1 Loading

The actions considered in the reliability analysis of the reinforced concrete beams were related to dead load, resulting from the structure's weight and an eventual wall load, and were also related to the live load, resulting from the weight of furniture and users of buildings. The incidence of wind is not considered because an extreme event resulting from this action has a low joint probability of occurring with the occurrence of a fire, according to Ellingwood [12]. Regarding the thermal load, it was considered the possibility of the beam cross-sections being heated at one, two, or three faces.

9.2 Design features of reinforced concrete beams

The reliability analysis carried out will consider simply supported reinforced concrete beams with a rectangular section that were dimensioned according to NBR 6118 [2] recommendations, i.e., the structure has a ductile rupture. Besides, the analyzed structural elements present dimensions, loads, and characteristics of beams that are representative of residential buildings, considering only normal strength concretes with siliceous aggregate.

9.3 Type of fire

The fire considered in the analysis is the standard fire contained in NBR 15200 [1]. This standard brings the standard fire curve from ISO 834 [13], which has a temperature given by Equation 6. Figure 5 shows how the standard fire temperature develops over time.

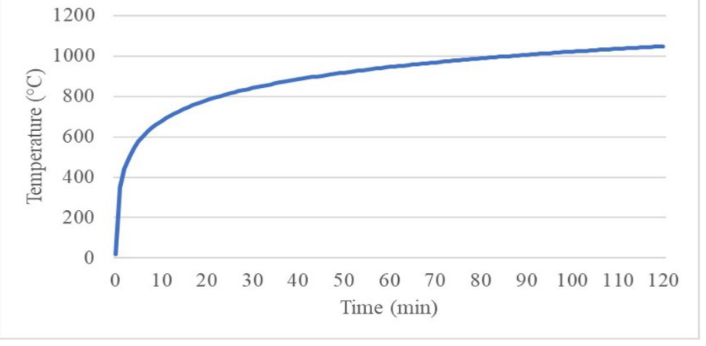

Figure 5. Standard fire from ISO 834.

9.4 Heat transfer flow

The heat transfer flow will be modeled using the Wickström Method, considering 3 possible situations to be chosen: (a) the first one refers to a two-dimensional flow in which the fire acts on 3 faces of the beam (on the two lateral sides and the lower side) since on the upper face it is considered that the beam is protected by a slab; (b) the second refers to a unidimensional flow in which the fire acts on 2 faces of the beam (only on the two lateral faces), considering the eventuality of having a wall below the beam that protects it thermally and (c) the last refers to a one-dimensional flow in which the fire acts only on one of the lateral faces of the beam. Figure 6 summarizes these situations.

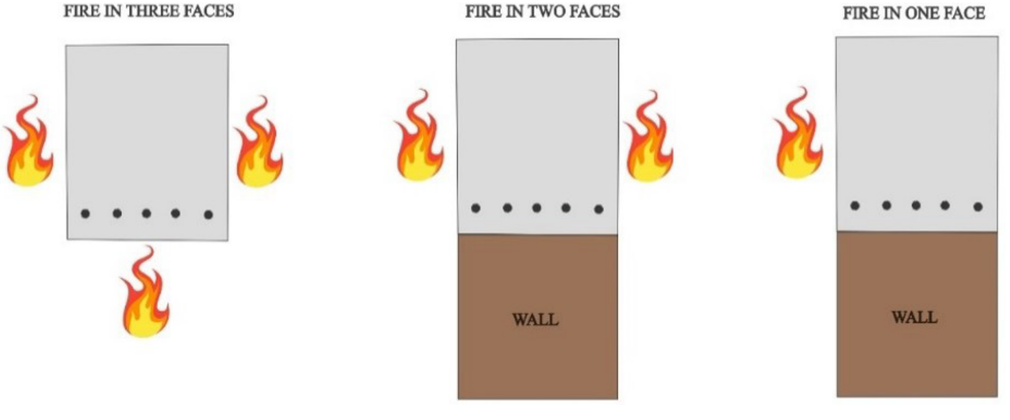

Figure 6. Scenarios of fire acting on a beam cross-section.

When using the Wickström Method in this work, the points of interest are the steel bars. Thus, the centroids coordinates of the bars are used in the method.

9.5 Concrete thermal diffusivity coefficient calibration

The coefficient of thermal diffusivity of concrete, which can be understood as a measure of how fast heat propagates in the material, has a great impact on the variability of the results of the reliability analysis of reinforced concrete beams in a fire situation. Furthermore, to this day there are few experimental data that can definitively establish statistics to represent accurately mean values to be used in the analyses. Therefore, the determination of this magnitude will occur in the same way as proposed by Eamon and Jensen [8].

First, the authors determine the resisting time to standard fire of the beam according to the methodology presented by Kodur and Dwaikat [14], which is called R. The expected resistance time to fire is determined according to the procedure proposed by the Australian standard AS 3600 [15], which defines this time considering the concrete cover and the width of the beam. After that, this time provided by the Australian standard is modified by multiplying some coefficients, like $\emptyset_{ag}$ that considers the beam's aggregate type, $\emptyset_{cs}$ that accounts for the concrete's compressive strength, the characteristics of the cross-section, and the combination load for the fire situation. These two last cited are considered in the determination of the structural coefficient ($\emptyset_{st}$). In this methodology, an increase in the combination load for the fire situation results in a smaller structural coefficient ($\emptyset_{st}$) and, in turn, a shorter expected resistance time (R). Because of that, the calibrated concrete thermal diffusivity is increased, which means that the heat spreads faster. In other words, a bigger combination load for the fire situation implies a beam which quickly develops higher temperatures in the cross-section.

Since this methodology was applied for design purposes and not for reliability analysis, according to Eamon and Jensen [8], the best estimate of the resistance time of the beams is found by multiplying the previously determined time (R) by the average ratio between the actual time that the beam samples failed and the time predicted by the model, which was found to be equal to 1.38 for simply supported beams.

Therefore, the time used to calibrate the thermal diffusivity will be equal to $1.38R$, which means that for this time interval, the resistant bending moment will present a value similar to the loading bending moment in the reliability analysis carried out due to the uncertainties of the several random variables involved, including the fire curve itself. So, the value is calibrated as to the cross-section time to failure matches the $1.38R$ resistant time to fire value.

9.6 Deterministic variables

Deterministic variables are those that assume single, fixed values, not showing variability or with variability that can be disregarded, so not being described by their probability distribution like random variables. So, the deterministic variables assumed in the present study are: (*i*) Span; (*ii*) Steel area; (*iii*) Steel bar diameters; (*iv*) Type of aggregate; (*v*) Initial room temperature and (*vi*) Live-to-total load ratio.

9.7 Random variables

The random variables and their statistical properties considered in the analysis are summarized in Table 4.

Table 4. Random variables.

Random Variable	Distribution	Bias Factor	COV	Reference
Dead loads	Normal	1.00	0.10	Jovanović et al. [16]
Live loads	Gamma	0.20	0.95	Jovanović et al. [16]
Fire temperature	Normal	1.00	0.45	Eamon and Jensen [8]
Steel yield strength	Normal	1.145	0.05	Eamon and Jensen [8]
Steel modulus of elasticity	Lognormal	1.00	0.06	Hamutçuoglu and Scott [17]
Concrete cover	Beta [0;3c_{nom}]	1.06	0.26	Silva [18] and Van Coile et al. [19]
Lower rebar position (d)	Normal	0.99	0.04	Eamon and Jensen [8]
Concrete compressive strength	Normal	Equation 23	Equation 24	Santiago [20]
Beam width	Normal	1.01	0.04	Eamon and Jensen [8]
Concrete thermal diff. coefficient	Normal	1.00	0.06	Eamon and Jensen [8]

Model uncert. Resistant Bend. Mom.	Normal	1.02	0.06	Eamon and Jensen [8]
Model uncert. Loading Bend. Mom.	Normal	1.00	0,05	Coelho [21]

The bias factor of each variable is the ratio between the medium value and the nominal value and the coefficient of variation (COV) is the ratio between the standard deviation and the medium value. This way, with the nominal values of each variable and the corresponding bias factors, the medium values are determined, which are used in conjunction with the coefficients of variation to determine the standard deviation. The nominal values considered for each variable are shown later in this paper in Table 5 when the reference beam is presented and detailed.

The bias factor and coefficient of variation of the compressive strength of concrete are given in Equation 23 and Equation 24, where f_{ck} is the nominal or also called the characteristic value of the compressive strength of concrete and σ_{f_c} is the standard deviation of this property.

$$Bias\ factor = 1 + 1.645 \times \sigma_{f_c}/f_{ck} \tag{23}$$

$$COV = \frac{8}{10^5} f_{ck}^2 - 0.009 f_{ck} + 0.3482 \tag{24}$$

The coefficient of variation of the compressive strength of concrete is modeled as polynomial function. It was modified to the information provided by Santiago [20], who between 2011 and 2016 examined f_c (at 28 days) in more than 39,000 cylindrical test specimens molded *in situ* on building sites throughout five Brazilian regions.

It is important to highlight that the model considered for the live load is the model for an arbitrary point in time and not for the maximum value in 50 years, since in this article a reliability analysis will be carried out in a fire situation, which can occur at any time during the life of the structure. In addition, concrete cover statistics data were taken from the measurement of beam cover before molding in nine buildings in the city of Porto Alegre, in the south of Brazil.

9.8 Limit state function

The limit state function will be analyzed at the cross-section level of the critical section of the beam. This is justified by experimental tests in a fire situation that shows the failure of the beams normally occurs by bending or bending with compression and not by shear NBR 15200 [1].

Because the reliability analysis is only performed for simply supported beams with uniform loads, this section with the critical section will always be in the middle of the beam span. Thus, the limit state function (LSF) used in this article can be written as $LSF = P \times M_n - Q \times M_a$, where P represents the model uncertainty associated with the resistant bending moment, M_n represents the bending moment resistance of the cross-section of the beam, Q represents the model uncertainty associated with the loading bending moment and M_a the loading bending moment. The latter can be determined by the equation for a simple supported beam under uniform load as $M_a = CL^2/8$, with C being the load considered on the beam (the sum of dead and live load), and L is the beam span. The ultimate resistant bending moment must be evaluated following the steps presented in section 8.

10 RELIABILITY ANALYSIS AND RESULTS

The reliability analysis will be carried out considering a reference beam, which will have some parameters modified throughout the process in order to access the influence of these parameters on the reliability of the structures in a fire situation. Concrete cover of the beam, the live to total load ratio and the number of heated faces of the beam will be the analyzed parameters. The first two parameters will consider that the beams are being heated on the three faces. Besides, a sensitivity analysis will be applied in the reference beam to determine which of the random variables has more impact on the limit state function and at when it occurs along the fire development in time.

10.1 The reference beam

The characteristics of the reference beam were chosen based on representative reinforced concrete beams of residential building in Brazil. This way, this structure was idealized as been the support of two rectangular concrete slabs with spans in both directions equal to four meters. The same span was assumed to the beam, which has a height of 40 cm and a width of 15 cm. Considering the load values established by NBR 6120 [3], the ultimate combination of loads determined by NBR 6118 [2] and the good design practices, the lower reinforcement resulted in 4 steel bars of

10 mm arranged in two layers and the upper reinforcement resulted in 2 steel bars of 8.0 mm. The concrete cover was assumed equal to 3 cm. Figure 7 shows these parameters and characteristics of the reference beam.

It is important to know that all steel bars are CA-50 and that the transversal reinforcement is only important to determine the correct position of the steel bars and for this purpose, the stirrups were assumed to have an usual diameter of 5 mm. Besides, the load of 15.5 kN/m is the characteristic value, which was increased 1.4 times by a required standard coefficient of NBR 6118 [2] to result in the reinforcement presented above. This load is composed of a 12.5 kN/m dead load and a 3 kN/m live load. Besides, the initial room temperature was assumed equal to 20°C. The considered nominal values of each random variable for the reference beam are shown in Table 5.

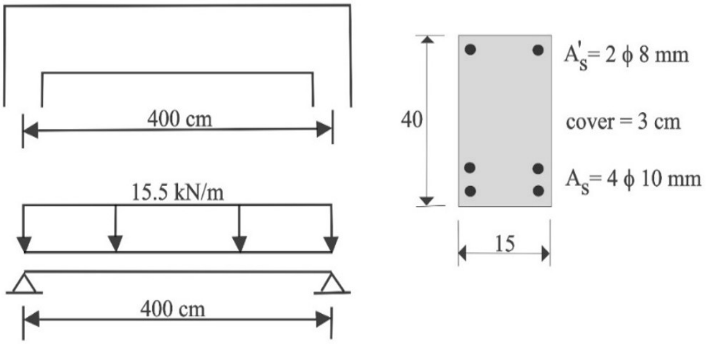

Figure 7. Reference beam.

Next will be determined the influence of the concrete cover, the live-to-total load ratio, and the number of heated faces in the reliability index. In the first two tests, there will be a change in the nominal values of concrete cover and the values of load (dead and live load). Regarding the number of heated faces, the change will be in the equation used to calculate the temperature inside the cross-section. The minimum value considered for the reliability index was zero because it represents a 50% of probability of failure. Besides, it is assumed in the analysis that the fire develops fully in four hours.

Table 5. Nominal value of random variables for the reference beam

Random Variable	Nominal Value
Dead load	12.5 kN/m
Live load	3 kN/m
Fire temperature*	1
Steel Yield strength	500 MPa
Steel Modulus of elasticity	210 GPa
Concrete cover	3 cm
Lower rebar position	36 cm
Concrete compressive strength of	25 MPa
Beam width	15 cm
Coefficient of thermal diffusivity of concrete	0.417×10^{-6} m²/s
Model uncertainty of resistant bending moment (P)*	1
Model uncertainty of loading bending moment (Q)*	1

*multiplicative random variables

10.2 Influence of the concrete cover

The analyzed values of the concrete cover correspond to the aggressiveness class established by NBR 6118 [2]. The standard considers different classes according to the conditions of the ambient of the structure. Class I refers to rural environments and determines a minimum of 2.5 cm for the cover. Class II refers to urban environments and requires a cover of 3 cm. Class III includes industrial and marine environments and considers a cover of 4 cm. For last, class IV refers to chemically aggressive environments and regions where the structure is hit by tidal splash, which must have a cover of 5 cm.

It should be noted that in the present article, there is a correlation between cover (c), lower rebar position (d), and height of the concrete section (h), so that the first two magnitudes were considered as random variables of the problem, as shown in Table 4. This correlation is shown in Equation 25, which also considers the diameter of the lower longitudinal bars ($\emptyset$) of the beam and the diameter of the stirrups ($\emptyset_{stirrup}$).

$$h = d + \frac{\emptyset}{2} + \emptyset_{stirrup} + c \quad (25)$$

Therefore, for each analysis, when varying the nominal cover of the beam, a variation in the nominal value of the lower rebar position was also considered so that the nominal value of the height of the concrete section is not modified. An increase in cover of 1 cm, for example, caused a decrease of 1 cm in the lower rebar position as the nominal height of the section was kept constant throughout the analysis.

Figure 8 shows the influence of the concrete cover on the reliability of the analyzed beams in a fire situation.

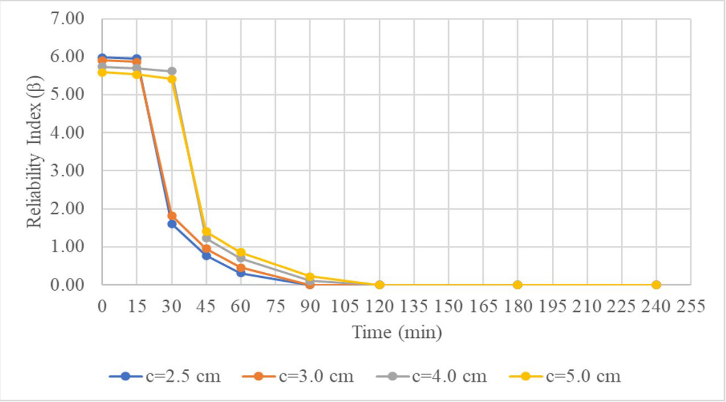

Figure 8. Influence of the concrete cover on the reliability of the beams.

As can be seen, the beams have an increase in reliability as their nominal cover is increased. More specifically, for the analyzed reference beam, there is a significant gain in reliability for the cases with a cover of 4 and 5 cm in a time of 30 minutes. This is because the centers of the steel bars, in these cases, are further away from the outer edge of the beam due to the increased cover. The increase in this distance, when using the Wickström Method and the model created in this article for a time of 30 minutes, represented a considerable decrease in the temperature value of the bars. This fact means that the bars did not lose any resistance, with only a decrease in the width of the concrete section due to the consideration of the 500°C Isotherm Method.

Another fact that deserves to be highlighted in the results generated is the reliability value in the 30-minute time and in the 15-minute time for the covers of 4 and 5 cm. It was expected, in principle, that the reliability of the beam with a 5 cm cover would be superior to that of the beam with a 4 cm cover, which ended up not happening. This is probably because, as for these two coverings, at these moments in time, the bars still had their full resistance, the negative impact of the fire only affected the concrete and not the steel. In turn, as previously mentioned, the nominal lower rebar position of the beams was changed with each cover change, so that the beam with the highest cover is the one with the lowest rebar position, which in this case, as the steel has not yet been affected, represents a disadvantage in the resistance of the beam section and that is why, for the 30 and 15 minutes instants, the beam with the highest cover has less reliability, even if this difference is minimal.

The noted behavior of the analyzed beam indicates that increasing the concrete cover represents a good solution to make the structure safer or with a lower probability of failure. In other words, the concrete cover makes the steel bars less affected by the heat and therefore they maintain or have a little decrease in their resistance, which makes the beam have a higher ultimate resistant bending moment.

10.3 Influence of the live-to-total load ratio

In this article, the live-to-total load ratio (χ) is considered as Equation 26, considering nominal values for the live and dead load.

$$\chi = Live\ load / (Live\ load + Dead\ load) \quad (26)$$

The analyzed values of the live-to-total load ratio in this article correspond to a representative range of values for usual reinforced concrete beams according to Santos et al. [22], which is between 0.1 and 0.6. The design of the reference beam considers a ratio of 0.19. This way, Figure 9 shows the influence of the live-to-total load ratio on the reliability of the analyzed beams in a fire situation.

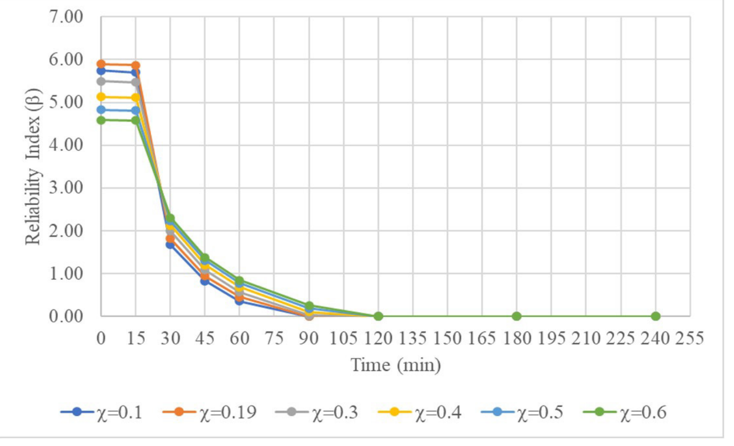

Figure 9. Influence of the live-to-total load ratio on the reliability of the beams.

Note that the graph indicates that for analyzed times 0 and 15 minutes, not considering the fire situation in the first case and considering the early stage of the fire in the second case, the reliability grows up to the rate value of 0.19 and that for higher rates, the reliability values decrease. From 30 minutes of fire, however, the situation changes as the reliability indices found were higher for cases in which the live to total load rates were higher.

The behavior perceived during the fire analysis is a direct result of the load combination for the fire situation adopted by NBR 15200 [1], which affects the thermal diffusivity coefficient calibration process as explained before. Since the dead load coefficient is 1.20 and the live load coefficient is 0.21 (already considering the reduction factor of 0.7 applied to the original coefficient of 0.3), there is a greater influence on the dead load. So, the higher it is, the lower the resistance time of the beams to fire and, consequently, the lower their reliability.

This happens because a large proportion of dead load implies a smaller χ, which in turn generates a large combination load due to fire. This represents a smaller structural coefficient ($\emptyset_{st}$) according to Kodur and Dwaikat [14] and, consequently, a smaller fire-resistant time. If the beam has a smaller fire-resistant time, the process of thermal diffusivity coefficient calibration will determine a large value, which means that the cross-section of the beam will achieve higher temperatures and therefore will have a reduction in the resisting bending moment.

The consideration of such a smaller coefficient of combination for the live load is supported by the fact a reasonable part of this load ends up being incinerated, like the furniture, and the part that represents the weight of the users is diminished since they are supposed to have escaped from the fire.

10.4 Influence of the number of heated faces

As shown before in Figure 6, there were considered 3 situations of fire: in one face, in two faces, and three faces of the beams. So far, the analysis of the influence of the concrete cover and live-to-dead load ratio considered the three faces being heated. The influence of the number of heated faces of the beam on the reliability can be seen in Figure 10.

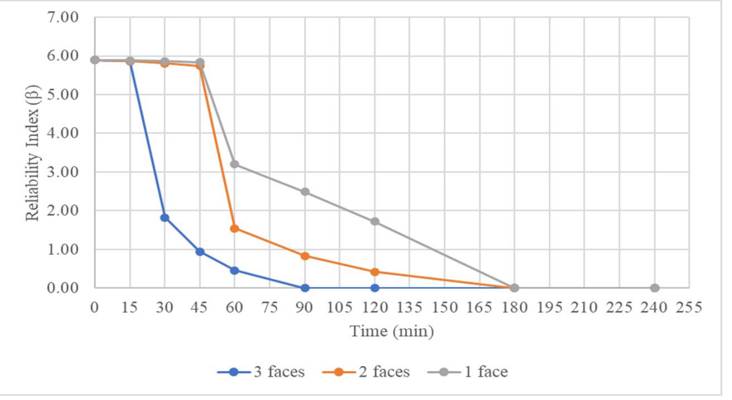

Figure 10. Influence of the number of heated faces on the reliability of the beams

As can be seen, there is a great influence of this parameter on the reliability of the beams, so the most reliable situation is the one in which the beam has the lowest number of faces affected by the fire. This is because the resulting temperatures in the cross-section are lower when considering less heated faces, especially when there is no two-dimensional flow (two and one face). Another aspect that can be noted is the linear decreasing behavior of the reliability index when the beam is subjected to a fire that heats just one face.

10.5 Sensitivity analysis

The sensitivity analysis of random variables conducted in this chapter aims to show which variables have the most importance throughout the fire. This procedure will be based on the values of the director cosines of each of the analyzed variables, given that such quantity represents the gradient of the limit state function in the standardized space. Therefore, because the director cosines are normalized, the director cosine can vary from -1 to +1, so the closer to the extreme values (-1 or +1), the larger influence such variable has on the reliability. Therefore, an illustration will be shown below indicating the value of the director cosine for each random variable considered at each instant of time of the analysis of the reference beam. It was assigned a number to each of random variables: (1) Dead load, (2) Live load, (3) Fire temperature, (4) Steel yield strength, (5) Lower rebar position, (6) Concrete cover, (7) Concrete compressive strength, (8) Beam width, (9) Concrete coefficient of thermal diffusivity, (10) Model uncertainty of resistant bending moment, (11) Steel modulus of elasticity, (12) Model uncertainty of loading bending moment. Thus, Figure 11 shows the sensitivity analysis carried out for the reference beam subjected to the standard fire acting in the three faces of the structure for 90 minutes.

At time 0 min., when there is no fire yet, the most influential variables are permanent load (-0.369) and variable load (-0.643), the yield strength of the steel (0.313), the lower rebar position (0.271) and model uncertainties associated with resisting (0.428) and loading (-0.302) moments. In the fire analysis, the variables that most influenced the limit state function were temperature (-0.799 at 30 minutes and -0.923 at 60 minutes) and beam cover (0.582 at 30 minutes and 0.370 at 60 minutes). The analysis does not present the values of the director cosine after 60 minutes because the reliability index is equal to zero after that moment.

Note, as previously mentioned, that the sign of the values of the director cosines show whether the variable has a positive or negative impact concerning the limit state function and that with the course of the fire, the importance of temperature grows and that of cover ends up decreasing. That is assumed because as the fire develops, the cover starts to be achieved by the fire with more intensity, which means that their function of protecting the steel bars is affected.

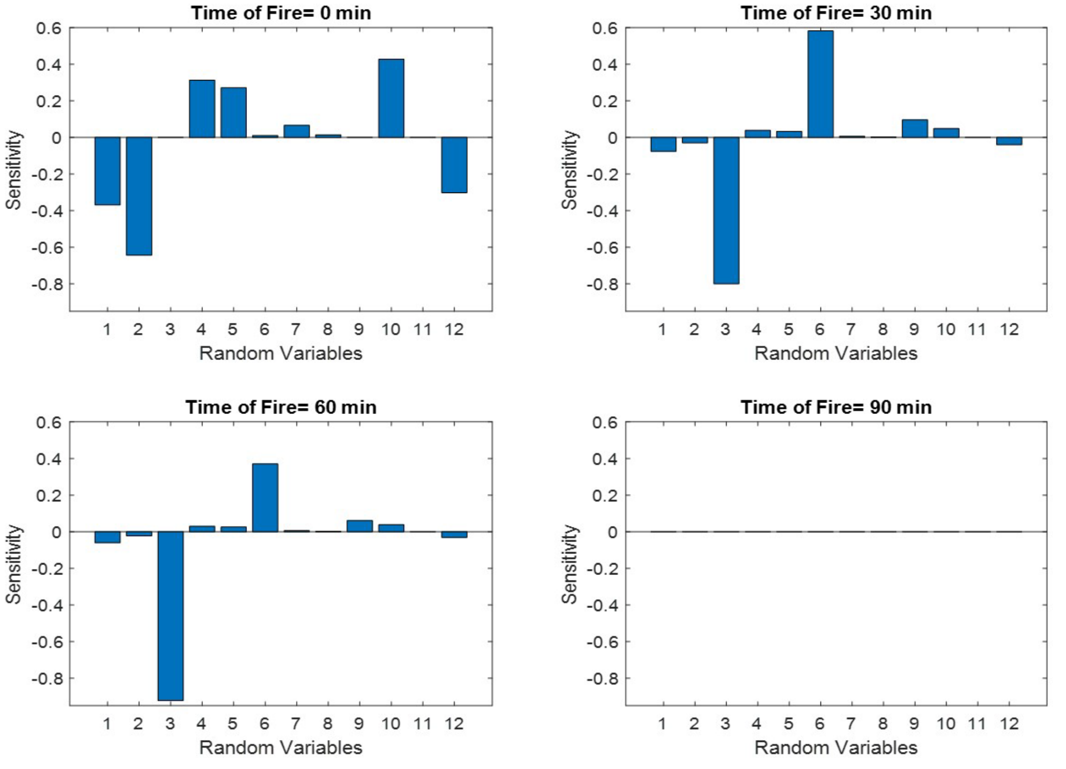

Figure 11. Sensitivity analysis.

11 CONCLUSIONS

This article presented a reliability analysis of cross-sections of simply supported beams of reinforced concrete in a fire situation. The 500°C Isotherm Method, which is a simplified method according to NBR 15200 [1], was used to consider the influence of high temperatures on concrete. In association, the Wickström Method was used to determine the temperature in the steel bars so that it could determine the reduction in their resistance, using the coefficients proposed by the previously mentioned standard.

Based on this study, the following conclusions can be summarized: (a) The concrete cover of the beam has a great influence on the reliability of these structures in a fire situation; (b) There is a reliability increase as the proportion of the live load concerning the total load increases; (c) The number of heated faces of the beam in a fire has a major role in the reliability of these structures; (d) Fire temperature and concrete cover of the beam are the tested variables that have the most influence on the analyzed limit state function; (e) The influence of the fire temperature in the limit state function increases and the influence of concrete cover decreases as the fire develops in time.

ACKNOWLEDGEMENTS

The authors wish to thank CAPES for the financial support of this work.

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Author contributions: RMP: methodology, conceptualization, writing, data curation; HMG: supervision, revision, data curation.

Editors: José Marcio Calixto, Guilherme Aris Parsekian.