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ORIGINAL ARTICLE

Probability of failure of RC columns in fire situation

Probabilidade de falha de pilares de concreto armado em situação de incêndio

Arthur de Caneda Preuss[a]
Herbert Martins Gomes[a,b]

[a]Universidade Federal do Rio Grande do Sul – UFRGS, Graduate Program in Civil Engineering, Porto Alegre, RS, Brasil
[b]Universidade Federal do Rio Grande do Sul – UFRGS, Graduate Program in Mechanical Engineering, Porto Alegre, RS, Brasil

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Abstract: The reliability of reinforced concrete (RC) columns in a fire situation is investigated in this paper by combining the General, Equilibrium, and Isotherm at $500°C$ Methods. The well-known Monte Carlo (MC) and the First Order Reliability Method (FORM) are used in a parametric study for the column's safety. The columns, in a biaxial bending-compression situations, are analyzed by an algorithm that takes into consideration physical and geometric nonlinearities. The dead and live load actions, steel and concrete material strengths, section geometry, model error, and temperature are among the random variables taken into consideration. The findings indicate that increasing the cover can dramatically reduce the probability of failure.

Keywords: reliability analysis, slender reinforced concrete columns, fire.

Resumo: A confiabilidade de pilares de concreto armado (CA) em situação de incêndio é investigada neste artigo combinando os Métodos Geral, de Equilíbrio e da Isoterma a $500°C$. O Método de Monte Carlo (MC) e o Método de Confiabilidade de Primeira Ordem (FORM) são usados em um estudo paramétrico sobre a segurança das colunas. Os pilares em situação de flexo-compressão oblíqua são analisados por um algoritmo que considera as não linearidades físicas e geométricas. As cargas permanentes e acidentais, resistências do material aço e concreto, geometria da seção, erro de modelo e temperatura estão entre as variáveis aleatórias levadas em consideração. Os resultados indicam que o aumento do cobrimento pode reduzir drasticamente a probabilidade de falha final.

Palavras-chave: análise de confiabilidade, pilares esbeltos de concreto armado, incêndio.

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1 INTRODUCTION

Reinforced concrete (RC) structures are sensitive to exposure to fire because the materials have their resistances reduced with the increase in temperature and because they can present the phenomenon called spalling, which is the detachment of complete surface layers of concrete. Due to the difficulty of releasing the vapors, regions with high pressures may arise that exceed the maximum bearable pressure, generating disaggregation of this region of the concrete. This phenomenon can be explosive and is mainly influenced by pore pressure, stresses due to temperature gradient, dilation differences, and chemical degradation at high temperatures [1]. Due to the complexity of spalling, it is not possible to predict it with simple mathematical models. Thus, in this article, it will be assumed that this phenomenon does not occur. For this to be coherent, the analyses will be restricted to the concretes of normal resistance, because in these, the effect is less likely, because, in general, more voids are observed in their microstructure.

At room temperature, the concrete material presents a homogeneous behavior, but at high temperatures, heterogeneity is observed, which makes it difficult to accurately predict the behavior. Hydrated cement paste expands

Corresponding author: Arthur de Caneda Preuss. E-mail: arthurcaneda@hotmail.com
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only at initial fire temperatures, with contraction above $300°C$. Lomba [2] presents that heterogeneity in RC occurs due to physical, chemical, and mineralogical transformations, which occur above $100°C$.

According to Fitzgerald [3], for steel, it is assumed that the critical temperature, in which the steel would lose the resistant capacity, defined as the temperature at which only 60% of the resistance remains and would probably occur the collapse, is equal to $538°C$ for conventional passive reinforcements. Both steel and concrete end up losing efficiency in a fire situation because the mechanical properties are reduced. In general, the understanding of the causes of this phenomenon is complex, as they are related to a series of physical and chemical phenomena. For simplicity, a normative approach to the problem can be given, treating the reduced resistance as a function of temperature, and applying a degraded coefficient that varies from one to zero.

Regarding the study of reliability in RC structures at room temperature, the bibliography is vast, because the most different types of structural elements have been studied, and in the fire situation, studies are scarcer. Among the few existing references, it is observed that all of them used simplified methodologies to develop thermostructural analysis. By thermostructural analysis, one should understand the study of the mechanics of structures together with the study of thermodynamics and heat transfer. Given the high computational cost involved in multiphysics analysis, it is currently impractical to apply more robust methods together with direct simulation methods, for example, finite elements, in personal computers.

Some of the main studies on structural reliability in RC elements in fire situations can be observed in [4]-[11], where it is possible to verify the safety study of different structural elements, employing various techniques to estimate reliability and approaches to evaluate thermostructural behavior. Specifically, concerning the study of the reliability of RC columns in a fire situation, there are the works of Eamon and Jensen [8] and Bai et al. [10] in which reliability for post-fire events was studied.

In Eamon and Jensen [8] the reliability of RC columns in fire situations designed according to ACI 318 [12] was studied using the Monte Carlo (MC) simulation, considered as random variables the temperature, live and dead loads, steel and concrete strengths, reinforcement positions and the geometry of the section. The load-bearing capacity of the columns was based on Rankine's approach, which is modeled by analytical expressions, whose objective was to evaluate the resistance of the column as a function of the time of exposure to fire. The study evaluated how reliability varies over time, and the analysis was developed from 0 to 4 hours of exposure, where it was possible to observe that reliability decreases in a non-linear way over time. Finally, a parametric analysis was performed where several parameters were changed, namely the fire curve, the live-to-dead load ratio, the reinforcement rate, the cover, the concrete strength the eccentricity, and the reinforcement rate.

The reliability of RC columns in fire situations will be investigated in this article using a simplified methodology. It takes into account for the verification process the General and Equilibrium Methods together with the Method of Isotherms at $500°C$ in the definition of the effective section of concrete, and the necessary corrections in the mechanical properties of steel reinforcements due to high temperatures. The work is focused on columns in biaxial bending-compression. For probabilistic assessments, the First-Order Reliability Method and the MC Simulation are used to evaluate the reliability at various stages of a fully established fire, ranging from 0 to 4 hours of exposure.

This article aims to present unpublished and unexplored reliability results for RC columns subjected to fire that are designed following Brazilian standards NBR 15200 [13] and NBR 6118 [14], as well as results for columns with only two faces exposed to fire.

2 THE WICKSTRÖM METHOD

This method makes it possible to represent the temperature field in concrete sections, subject to a one and two-dimensional heat flow over time by analytical equations. The method was developed from the adjustment of a series of finite element analyses of concrete sections exposed to fire, which allows the evaluation of temperatures in both steel bars and concrete. Knowing the composition of the concrete, the temperature is estimated based on the distance from the point of analysis to the exposed faces and the exposure time. The average gas temperature of the environment is estimated by the fire curve adopted in the analysis, obtained for a given exposure time.

In Wickström [15] the equations of Wickström's method for estimating the temperature in reinforced concrete sections are presented. The results of the temperature values obtained with such equations were compared to the respective values obtained through the finite element method using the computer program TASEF-2 and it was observed that for regular sections the results obtained using the analytical expressions were very close to the numerical ones. A full explanation of the method can be found in Wickström [1].

The temperature increases for a one-dimensional heat flow (with one exposure side), represented in Figure 1a, is given by Equation 1.

$$\Delta\theta_x = n_x \cdot n_w \cdot \Delta\theta_f \tag{1}$$

where n_x represents the relationship between the temperature rise on the surface and a point inside the section; n_w represents the relationship between the temperature rise of the exposed surface and the temperature of the environment; and $\Delta\theta_f$ is the gradient of the ambient temperature obtained by the fire curve employed ($°C$).

For the two-dimensional flow, whose situation is illustrated in Figure 1b, one has the temperature increase given by Equation 2.

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

where n_x and n_y represent the relationships between the increase in surface temperature and a point within the section.

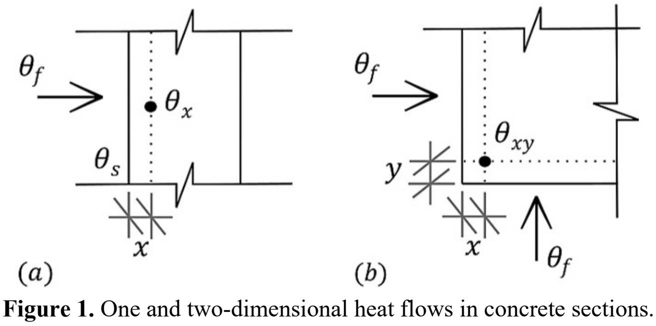

Figure 1. One and two-dimensional heat flows in concrete sections.

The relationship n_x (or n_y replacing x by y) is given by Equation 3, and the parameter is u_x determined by Equation 4.

$$n_x = 0.18\, ln\, u_x - 0.81 \tag{3}$$

$$u_x = \frac{a}{a_c} \cdot \frac{t}{x^2} \tag{4}$$

where a is the concrete thermal diffusivity; a_c is a reference value equal to $0.417 \cdot 10^{-6}$ (m^2/s); x means the distance from the face to point (m); and t is the time (h).

Parameter a is determined according to concrete density, aggregate type, and temperature, among other factors, the estimate of which can be made as presented in Purkiss and Li [16]. In this work, we adopted the value of a as being equal to that of a_c. The n_w is defined by Equation 5, using the time in hours (h).

$$n_w = 1 - 0.0616t^{-0.88} \tag{5}$$

3 THE 500°C ISOTHERM METHOD

This is an approximate method, proposed by Anderberg [17], which can be applied in the verification of RC sections subject to simple or biaxial bending, subjected to any fire curve, provided that the temperature distribution in the section is similar to that obtained considering the standard fire. It is assumed that the heated concrete up to 500°C is not significantly affected by temperature, being necessary only for the evaluation of the temperature effects on the reinforcements, disregarding the analysis of the concrete regions with a temperature higher than 500°C [18].

It should be noted that in practical situations, the resistance of concrete above 500°C is not null, even so, some experimental tests considering different types of exposure to fire and loading, showed satisfactory results [17]. It is admitted that the concrete of the resulting section presents the same compressive strength and modulus of elasticity as at room temperature. Steel bars, on the other hand, should have their mechanical properties adjusted to the temperature at the rebar position [18].

As the incidence of heat flow happens at the faces, the areas that will eventually be disregarded will be superficial, and the resulting section is corresponding to a fictitious area of calculation. Figure 2 presents two possibilities of heat incidence that aim to represent some situations that columns of a building can present. Figure 2a shows the incidence of a heat flow on four faces of a rectangular column, and Figure 2b, two faces. The 1D and 2D indications refer to the temperature field in the section, which can be one or two-dimensional in the analyzed quadrants column's corners.

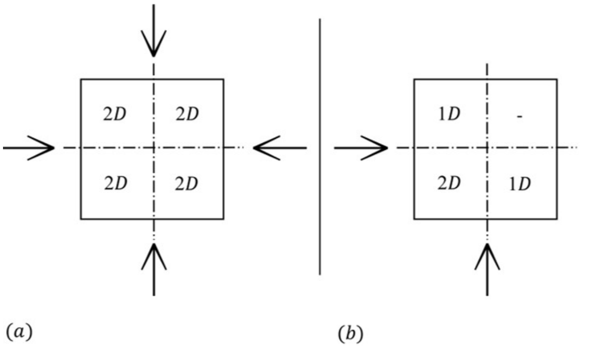

Figure 2. Distribution of temperatures in the sections with their faces exposed to fire.

Using the Wickström method to calculate the isotherm distances for 500°C concerning exposed faces, Equations 6 and 7 are used. These distances are illustrated in Figures 3 and 4 for situations with two, and four faces exposed to fire, respectively, where the effective polygons of the reduced section of concrete that will be taken into account in the verifications of the columns are represented. It should be noted that the approximation by a polygonal to represent the isotherm's level set corners is in favor of safety, and this procedure is presented in Purkiss and Li [16].

Klein Júnior [19] warns that the Method of Isotherms of 500°C was originally developed for elements submitted to simple bending, in which a rupture controlled by steel flow is usually observed. This was decided to be used also in the case of biaxial bending because it is a procedure allowed by EN 1992-1-2 [20].

$$x_{1D} = \sqrt{\frac{\frac{a}{a_c}t}{exp\left(4.5+\frac{480}{0.18n_w\Delta\theta_f}\right)}} \tag{6}$$

$$x_{2D} = \sqrt{\frac{\frac{a}{a_c}t}{exp\left[4.5+\frac{n_w-\sqrt{n_w^2-(2n_w-1)\frac{480}{\Delta\theta_f}}}{0.18(2n_w-1)}\right]}} \tag{7}$$

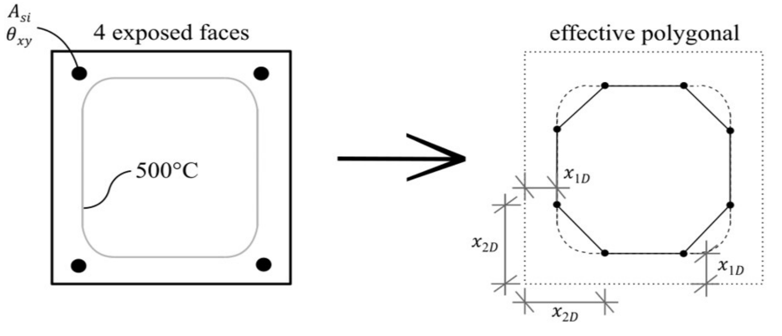

Figure 3. Approximation for the 500°C isotherm by a polygon when four faces are exposed.

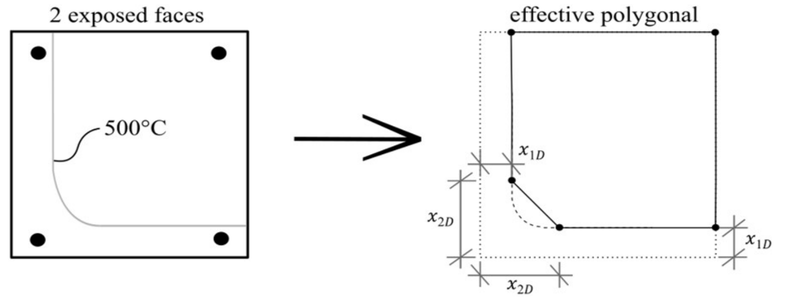

Figure 4. Approximation for the $500°C$ isotherm by a polygon when two faces are exposed.

A possibility of refinement of the method, presented by Costa [18], is to consider multilayers, for example, to use two temperature limits, $400°C$ and $600°C$, disregarding the peripheral regions of the section with a temperature higher than $600°C$, assuming that the concrete presents 70% of the room temperature resistance when $400°C \leq \theta \leq 600°C$ and, 90% for $\theta < 400°C$.

4 NORMATIVE CRITERIA

NBR 6118 [14] considers that the structures are in an environment with temperature equal to $20°C$. To consider the effects of high temperatures, the NBR 14432 [21] and NBR 15200 [13] can be applied together to NBR 6118 [14] recommendations. In these standards, the properties of materials, calculation methods, and fire resistance requirements are defined.

In NBR 15200 [13], it is indicated that its guidelines apply to normal concretes, identified by specific dry mass greater than 200 kg/m^3, and less than 2800 kg/m^3, for concretes of group I. For concretes of a class higher than C50, it is recommended to use EN 1992-1-2 [20].

4.1 NBR 6118 (2014) recommendations

NBR 6118 [14] declares that is not in its scope to state the requirements to avoid the limit states generated by fire actions, informing the need to refer to NBR 15200 [13].

4.1.1 Ultimate limit state

The Ultimate Limit State as defined by NBR 6118 [14] is related to the collapse, or any other form of structural ruin. In the present article, the ultimate limit states for cross-section equilibrium and instability is considered. In this way, the ruin will be considered for a possible rupture of a section of the column, or due to the instability of the column as a whole by large lateral deflections. NBR 6118 [14] requires the following checklist for columns: (*i*) evaluation of instability; and (*ii*) evaluation of equilibrium of each of the sections for the active loading. The ultimate limit state for the column will be the lowest value found by the two checks.

4.1.2 Ultimate limit state for cross-section equilibrium

The rupture of the sections can happen in two ways, which are by excessive plastic deformation of the steel and/or shortening rupture of concrete. In the ultimate limit state, the following basic hypotheses are assumed for linear elements subject to normal loadings: (*i*) the hypothesis of the Navier-Bernoulli is applied; (*ii*) the deformation in the steel rebars is the same as the surrounding concrete (adhesion); (*iii*) the strength of concrete in tension is disregarded; (*iv*) the maximum allowed elongation in the tensile reinforcement is equal to 10‰; and (*v*) the stress distribution in concrete is made according to the idealized parabola-rectangle stress-strain diagram presented by Figure 5.

For normal strength concretes (less than or equal to C50), the deformation limits ε_{c2} and ε_{cu}, presented in Figure 5, are equal to 2‰ and 3.5‰, respectively. This figure presents the curve with peak stress equal to $0.85f_{cd}$ which is employed in the cross-section equilibrium check. The design strength of the compressive concrete, f_{cd}, is determined by dividing the characteristic compressive strength of the concrete, f_{ck}, by the strength reduction coefficient, γ_c, which is adopted equal to 1.4 for normal combinations.

Within the range of possible deformations in the ultimate limit state, the section will be classified into any of the deformation domains presented in NBR 6118 [14], which can be: lines A, 1, 2, 3, 4, 4a, 5, or line B. Such domains are representations of how a section may be deformed when subject to normal loadings. It is admitted that the ruin can occur due to the rupture of the concrete and/or excessive deformation of the reinforcements, caused by normal loadings, being in the most general case originated by compressive biaxial bending.

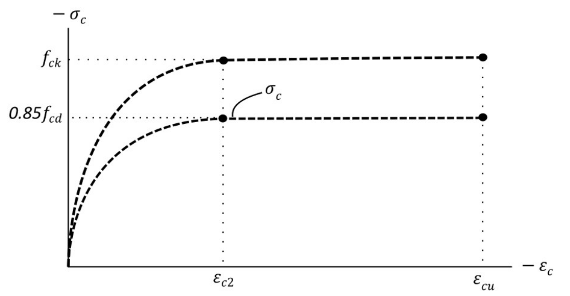

Figure 5. Idealized parabola-rectangle stress-strain diagram for concrete.

4.1.3 Ultimate limit state for instability

The structures constituted by bars submitted to biaxial bending, where the contribution of the effects of torsion can be disregarded to the second-order effects, should be evaluated concerning the ultimate limit state for instability. Instability is achieved when under increasing loads, the increase in resistant capacity becomes less than the increase in the loading, and the second-order effects should be evaluated by a geometric and physical nonlinear analysis.

NBR 6118 [14] states that in the structural analysis with the second-order effects, for the most unfavorable combinations of the design loads, there should be no instability or the design load-bearing capacity should not be reached. For the instability analysis, the evaluation of the deformability of the elements should be performed considering the peak stress of the concrete equal to $1.10f_{cd}$ in the parabola-rectangle stress-strain diagram and, that of steel, f_{yd}, with the strength reduction coefficient used for the ultimate limit state. This consideration of the peak stress already includes the effect of the maintained load (Rüsch effect).

4.1.4 The Equilibrium Method

The experiments show that the axially compressed straight columns reach a limit state known as instability in axial compression under the influence of increasing loadings, where the straight form of equilibrium is unstable and the stable form of equilibrium in the elastic regime is a bended configuration. The critical load or buckling load is the load that corresponds to such a limit state. For a load above the critical load, the change in the form of the equilibrium relates to an unstable behavior, in case the column is above the proportionality limit (material is no longer linear elastic), and therefore, the bended shape is impossible. Additionally, instability is an ultimate limit state for the structural materials like steel and concrete because the bar exhibits significant strains for loads just above the critical point, leading to rupture by biaxial bending [22].

The Equilibrium approach involves determining if the displacements of the sections (as determined by the discretization used) result in a stable situation for a particular loading. Thus, it is attempted, in conjunction with the General Method, to iteratively calculate the loadings (second order) along the column from its deformed configuration, updating the values of curvatures and displacements at each iteration, and evaluating whether the equilibrium is maintained until the process converges.

4.1.5 The General Method

The General Method, as reported by NBR 6118 [14], consists of the second-order nonlinear analysis performed with the proper discretization of the column height, considering the actual moment-curvature relationship in each section and considering the geometric nonlinearity in a non-simplified way. Therefore, this method presents more accurate results than those obtained with the use of simplified methods of the standard column. This methodology becomes mandatory for a slenderness index, λ, greater than 140, and can be applied to any λ value and to any form of the cross-section that complies with the dimensional and constructive criteria of columns in NBR 6118 [14]. In addition, the reinforcement applied stresses, and cross-section dimensions can be variable over the length, which makes the method applicable in many design situations.

The more traditional approaches of the method employ progressive load increments or eccentricity, however, the General Method can be applied from the Equilibrium, verifying the safety against the limit state for instability, from the analysis of the design loads, to evaluate whether the displacements of the analyzed sections correspond to a stable configuration of equilibrium. In this way, this is checked by calculating only one point of the loading x displacement diagram. The application of the General Method from the Equilibrium takes the following steps: (*i*) calculation of loadings along the axis of the column, from a deformed configuration; (*ii*) starting from the loads in a section, the corresponding curvatures are calculated; and (*iii*) integration of the column curvatures of the different sections along the column in each orthogonal plane, to obtain the lateral displacements.

4.2 NBR 15200 (2012) recommendations

Together with NBR 6118 [14], NBR 15200 [13] verifies the safety of the design of concrete structures in fire situations. In general, fire protection requirements aim to reduce the risk of fire, control fire in its early stages, limit the area exposed to fire, create escape routes for building users, facilitate firefighting operations and prevent the premature collapse of the structure.

The required fire resistance time can be determined as the action corresponding to the standard fire, which is represented by the standard fire exposure time interval. From this exposure, each structural element will present a temperature distribution, which results in the modification of acting loads resulting from axial elongations or thermal gradients and in the modification of the mechanical properties of the materials. Such changes result in modifications of capacity of the structural elements. Generally, the dilatation loadings due to heating are neglected, because as the temperature in the structural element increases, the stiffness decreases, but the capacity for plastic adaptation increases proportionally, justifying the non-consideration of additional loads.

The verification of reinforced concrete structures under fire situation should evaluate the compliance with the ultimate limit state for the exceptional fire combination, with the following assumptions admitted: (*i*) the fire action can be translated by reducing the strength of the materials and the capacity of the elements, and (*ii*) loadings resulting from major deformations in the fire situation are disregarded. This verification is done by following the recommendations for verification of load combinations presented in NBR 15200 [13].

The structures should be designed for the ambient temperature situation and depending on their characteristics and use should be checked in a fire situation. Regarding the methods for verifying concrete structures in a fire situation, NBR 15200 [13] recommends the use of any of the following methods: (*i*) tabular method, (*ii*) simplified method, (*iii*) advanced method, (*iv*) experimental method, and (*v*) analytical method for columns.

In this article, the simplified Method of the 500°C Isotherms will be used. Other simplified methods in the established technical literature that deserves to be highlighted are: the DTU Method, presented in DTU [23]; the PCI Method, presented by Gustaferro and Martin [24]; the ISE Method, presented in ISE [25]; and the Strip Method, presented by Hertz [26]. The simplified methods start from the load verification and should be ensured that the value for design resistance is greater than the acting design load.

4.2.1 Fire curve

In this article, the ISO 834 [27] curve was used, which represents a typical fire in buildings with cellulosic fuel, which is also the curve recommended by NBR 15200 [13]. The variation in the average temperature of the gases, $\Delta\theta_f$, as a function of the exposure time, in this case, is presented by Equation 8. In practice, this is one of the most used curves for structural verification worldwide and, for this reason, several experimental methods and calculation methodologies are based on it. It should be noted that this curve does not present a cooling phase, mainly due to the mathematical modeling by a logarithmic curve, the temperature values will always be increasing.

$$\Delta\theta_f = 345\, log_{10}(480t + 1) \tag{8}$$

where t is the time of exposure (h).

4.2.2 Multilinear steel diagram at high temperatures

NBR 15200 [13] recommends the use of the multilinear stress-strain diagram for steel at high temperatures, presented by Equation 9. It is worth noting that it is a function defined by ranges, and limits of deformation are defining criteria of each expression. Four distinct stages can be observed: in the first range a linear behavior, in the second, an elastoplastic with hardening, in the third, a yielding phase and, in the last, the softening behavior until rupture. The deformations $\varepsilon_{y,\theta}$, $\varepsilon_{t,\theta}$, and $\varepsilon_{u,\theta}$ are assumed equal to 0.02, 0.15, and 0.20, respectively, for high ductility steels (CA 25/50). Figure 6 shows some curves of steel stress-strain diagrams for f_{yk} a value equal to 500 MPa and some temperature values.

$$\sigma_{s,\theta} = \begin{cases} \varepsilon_{s,\theta} \cdot E_{s,\theta};\ if\ 0 \leq \varepsilon_{s,\theta} \leq \varepsilon_{p,\theta} \\ f_{p,\theta} - c + \frac{b}{a} \cdot \sqrt{a^2 - \left(\varepsilon_{y,\theta} - \varepsilon_{s,\theta}\right)^2};\ if\ \varepsilon_{p,\theta} \leq \varepsilon_{s,\theta} \leq \varepsilon_{y,\theta} \\ f_{y,\theta};\ if\ \varepsilon_{y,\theta} \leq \varepsilon_{s,\theta} \leq \varepsilon_{t,\theta} \\ f_{y,\theta} \cdot \left[1 - \left(\frac{\varepsilon_{s,\theta} - \varepsilon_{t,\theta}}{\varepsilon_{u,\theta} - \varepsilon_{t,\theta}}\right)\right];\ if\ \varepsilon_{t,\theta} \leq \varepsilon_{s,\theta} \leq \varepsilon_{u,\theta} \\ 0;\ if\ \varepsilon_{s,\theta} \geq \varepsilon_{u,\theta} \end{cases} \tag{9}$$

where $\sigma_{s,\theta}$ is the stress of steel at high temperatures; $\varepsilon_{s,\theta}$ is the strain of steel at high temperatures (independent variable of the function); $E_{s,\theta}$ is the modulus of elasticity of steel at high temperatures, obtained by the product between $\kappa_{E_s,\theta}$ and E_s ($E_{s,\theta} = \kappa_{E_s,\theta} \cdot E_s$); E_s is the modulus of elasticity of steel at room temperature; $\varepsilon_{p,\theta}$ is the strain of steel at high temperatures corresponding to the proportionality limit, obtained by the ratio between $f_{p,\theta}$ and $E_{s,\theta}$ ($\varepsilon_{p,\theta} = f_{p,\theta}/E_{s,\theta}$); $f_{p,\theta}$ is the strength corresponding to the proportionality limit for steel at high temperatures, obtained by the product between $\kappa_{p,\theta}$ and $f_{yk,\theta}$ $\left(f_{p,\theta} = \kappa_{p,\theta} \cdot f_{yk,\theta}\right)$; $f_{yk,\theta}$ is the characteristic yielding strength of steel at high temperatures, obtained by the product between $\kappa_{s,\theta}$ and f_{yk} ($f_{yk,\theta} = \kappa_{s,\theta} \cdot f_{yk}$); and f_{yk} is the characteristic strength to steel flow at room temperature.

The coefficients $\kappa_{E_s,\theta}$, $\kappa_{s,\theta}$ and $\kappa_{p,\theta}$ represents the variation of the modulus of elasticity of steel, the characteristic strength of steel, and the strength corresponding to the proportionality limit of steel, respectively, presented by NBR 15200 [13]. These coefficients represent the degradation of these quantities over time. Such coefficients are equal to a unit when there is no degradation (room temperature situation) and equal to zero for a temperature equal to 1200°C. Using the data provided in NBR 15200 [13], it is possible to make a linear interpolation and obtain the values for intermediate temperatures that are not indicated in the tables of the standard.

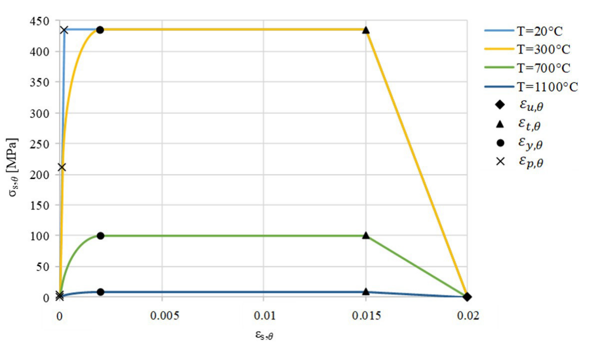

Figure 6. Steel multilinear constitutive relations as a function of temperature.

The parameters a, b and c are defined by Equations 10, 11 and 12, respectively.

$$a^2 = \left(\varepsilon_{y,\theta} - \varepsilon_{p,\theta}\right) \cdot \left(\varepsilon_{y,\theta} - \varepsilon_{p,\theta} + \frac{c}{E_{s,\theta}}\right) \tag{10}$$

$$b^2 = c \cdot \left(\varepsilon_{y,\theta} - \varepsilon_{p,\theta}\right) \cdot E_{s,\theta} + c^2 \tag{11}$$

$$c = \frac{\left(f_{y,\theta} - f_{p,\theta}\right)^2}{\left(\varepsilon_{y,\theta} - \varepsilon_{p,\theta}\right) \cdot E_{s,\theta} - 2 \cdot \left(f_{y,\theta} - f_{p,\theta}\right)} \tag{12}$$

5 METHODOLOGY FOR CHECKING RC COLUMNS IN FIRE SITUATION

The design of a structural element under fire by simplified methods is analogous to the design situation at room temperature, provided that the effects of temperature on materials are considered.

Costa [18] presents that the basic procedures employed in simplified methods for the designing of RC elements in fire situations are: (i) determining the temperature in the environment (average temperature of gases for the fire curve employed); (ii) determining the temperature in steel bars and concrete; (iii) reducing the cross-section, disregarding the surface region of the concrete with temperature above a certain limit (when the method employed admits this hypothesis); (iv) reducing the mechanical characteristics of steel and concrete as a function of high temperature; (v) evaluating the resistance of the reduced section, with the same procedures as for the usual situation, but with the mechanical properties adjusted, allowing the reinforcement to be outside the reduced section (it should be noted that this is only an abstraction, because physically the reinforcement bars would be protected by the concrete that surrounds them); and (vi) comparing the resistant values of the degraded section in the fire situation with the acting loads.

In this article, the proposed approach to the verification of columns in a fire situation is to use the ISO 834 curve to model the average temperature of gases in the fire. The Wickström method allows to obtain internal concrete and steel bars temperatures. Using the $500°C$ Isotherm Method, the surface regions of concrete are disregarded and then, NBR 15200 [13] criteria for the mechanical strength of steel is reduced as a function of temperature. Together with the procedures indicated, the columns are analyzed at room temperature, by the Equilibrium Method together with the General Method, used for the acting loads, the strategy presented in section 8. Moreover, the possibility of spalling was not foreseen, so only concretes of group I, were evaluated because the occurrence of this phenomenon is less likely in these concretes.

6 THE EMPLOYED ALGORITHM

The algorithm to check the RC of columns used in this article is the result of the studies presented in Cadamuro Júnior [28]. Therefore, the reader is directed to this work for a complete and detailed explanation. This algorithm considers the physical and geometric nonlinearities, applying the Equilibrium Method together with the General Method, according to the recommendations of NBR 6118 [14]. The generality of the algorithm is such that it allows the evaluation of columns of any slenderness ratio, with concrete sections, reinforcement, and first-order acting loads that may vary along the height of the column, and dealing with biaxial bending with compression. To verify the RC columns in a fire situation, some modifications were introduced to the algorithm to consider the Wickström method for an evaluation of the temperature distribution in the sections so that it was possible to disregard the surface region of concrete above $500°C$ and make the appropriate corrections in the mechanical properties of the reinforcements. With this, the columns were analyzed as in ambient temperature, however, with the necessary corrections due to exposure to fire.

The procedure of analysis of the columns is iterative, and the second-order moments of an iteration are added to the first moments in a new iteration and used to calculate the transverse displacements, which in turn will change the values of the second-order moments of the next iteration. In general, this process is developed until convergence is reached or a rupture is detected at the section level or due to lateral displacement instability.

In this approach using the General Method, the curvatures, transverse displacements, and second-order moments are calculated at the section level of the discretization. Moreover, as simplifying hypotheses, it is admitted: (i) perfect adhesion between the materials; (ii) a linear variation for curvature between consecutive sections; (iii) no initial stresses and deformations; (iv) the Navier-Bernoulli hypothesis; (v) small displacements hypothesis; and (vi) movement of the plane sections along height only by lateral translation or rotation.

The determination of the transverse displacements is made by double numerical integration of the curvature differential equation along the length of the column. This procedure allows for cubic displacement variation. These cross-sectional displacements are used in the second-order moment calculations.

7 VALIDATION OF THE PROPOSED METHODOLOGY

To validate the methodology proposed in this paper for the verification of RC columns subjected to fire, the experimental/numerical ultimate loads of some examples presented in Padre [29] will be compared. It is observed that the numerical results of the ultimate load of the columns were obtained by an approach based on the finite element method for those experimentally tested by Xu and Wu [30]. The experimental campaign consisted of applying on the columns a centralized load and exposing all the faces to fire until collapse. The ISO curve 834 was used for the temperature history in the numerical evaluation. The tensile strength of concrete was neglected, and the parabola-rectangle diagram was used for concrete in compression with peak stress equal to f_{cu} (strength obtained experimentally). For steel, the multilinear stress-strain diagram at high temperatures was used.

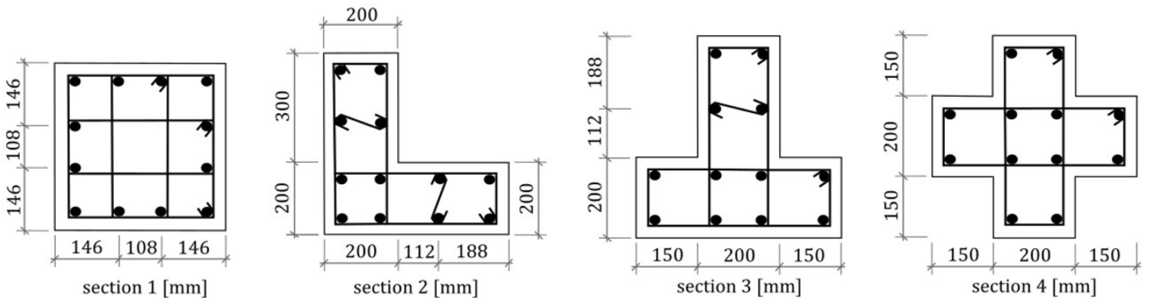

Figure 7. Column sections used for validation (dimensions not in scale).

The examples used in the validation were named columns 1, 2, 3, and 4, whose cross-sections are sections 1, 2, 3, and 4, respectively, presented in Figure 7. In all sections, the cover is equal to 30 mm. In all examples, 12 longitudinal bars of 16 mm in diameter were used, with a yielding strength f_y equal to 418 MPa. In addition, the static scheme of all columns was idealized as being simply supported on both ends with a height equal to 381 cm.

In the experimental test, column 1 resisted 169 minutes of fire until it collapses, under a centered compression load equal to 2325 kN. The concrete used in column 1 presented a compressive strength f_{cu} equal to 28.43 MPa. Column 2, on the other hand, resisted 147 minutes of fire until it collapses under a compression load centered equal to 2060 kN. The concrete used in column 2 presented a compressive strength f_{cu} equal to 30.38 MPa. Column 3, in turn, resisted 148 minutes of fire until it collapses under a centered compression load equal to 1902 kN. The concrete used in column 3 presented a compressive strength f_{cu} equal to 28.43 MPa. Finally, column 4 resisted 245 minutes of fire until it collapses under a centered compression load equal to 1480 kN. The concrete used in column 4 presented a compressive strength f_{cu} equal to 30.38 MPa. The spalling phenomena was not seen in any of the studies.

Table 1 presents a summary of the experimental and numerical results obtained by the algorithm of this paper and the methodology proposed in Padre [29] and the experimental campaign. In the fifth column of the table, named (1)/(2), it is presented the ratio between the experimental results and the numerical results obtained in this. An average ratio of 1.024 corresponding to all the tests was obtained. In the sixth column of the table, named (1)/(3), the ratio between the experimental results and the numerical results obtained by Padre [29] is presented. An average ratio of 1.016 was obtained.

Table 1. Summary of numerical and experimental results of ultimate load.

Column	(1) Experimental ultimate load [kN]	(2) Ultimate load, this paper [kN]	(3) Padre [29] [kN]	(1)/(2)	(1)/(3)
1	2325.00	2428.36	2241.54	0.957	1.037
2	2060.00	1883.00	2074.42	1.094	0.993
3	1902.00	1982.17	1998.28	0.960	0.952
4	1480.00	1364.04	1370.48	1.085	1.080

By comparing the results, it can be observed that the methodology proposed in this article is adequate for the estimation of the load for the collapse of the RC columns in a fire situation. In general, the ratio of the results between the experimental and numerical load was close to the unit. In this case, the experimental and numerical comparison serves only to prove that the algorithm is suitable for estimating the ultimate load. In any case, it is still advisable to use more experimental values from tests to better statistically characterize the modeling error, besides the study of the variability of materials and laboratory conditions in the fire situation.

8 PROBABILISTIC ANALYSES

In structural reliability analysis, one should employ a limit state function that represents a condition of violation of the system being analyzed. In this article, the limit state function used represents the collapse of the column, as presented by Equation 13, the result being the difference between the resisting load and the sum of the live and dead portions of the acting loading. In addition, in Equation 13 there are multiplicative random variables for modeling errors that aim to represent statistically the uncertainties of numerical modeling of the ultimate resisting load and the acting loading hypotheses. Whenever $g(\boldsymbol{X})$ is less than or equal to zero, a system failure is detected.

$$g(\boldsymbol{X}) = R(\boldsymbol{X}) \cdot \theta_R - S(\boldsymbol{X}) \cdot \theta_S \tag{13}$$

where $R(\boldsymbol{X})$ is the ultimate resisting load, obtained by the bisection method, using the parabola-rectangle diagram with peak stress equal to f_c (random variable of concrete compressive strength generated from f_{ck}), neglecting the tensile strength of concrete, and considering the multilinear steel stress-strain diagram for high temperatures with peak stress equal to f_y (random variable of steel yielding strength generated from f_{yk}); θ_R is the random model error variable related to the resisting modeling prediction; $S(\boldsymbol{X})$ is the sum of the live and dead portions of the loading, obtained from the General Method of NBR 6118 [14]; and θ_S is the random variable of load, related to loads modeling error.

It should be noted the value of $R(\boldsymbol{X})$ that leads to the column failure was considered to be a simple eccentric load applied to the top of the columns, so the compressive load and bending moment are perfectly correlated.

The random variables used in the reliability studies are indicated in Table 2. Given the lack of data in the literature, the statistics of random variables in fire situations were considered equivalent to the statistics of random variables at room temperature, which is widespread. This same hypothesis was used in the works of Eamon and Jensen [6]-[8] and Coelho [11]. As a simplifying hypothesis, it was admitted that the design variables are not correlated.

Table 2. Statistical properties of random variables.

Random variable	**Probability Density Function**	**Mean value ($\boldsymbol{\mu}$)**	**Coefficient of variation ($\boldsymbol{V}$)**	**Standard deviation ($\boldsymbol{\sigma}$)**
Compressive strength (f_c) $[MPa]$	Gaussian	$1.17 \cdot f_{ck}$ [a]	σ_{fc}/f_{cm} [b]	$0.15e^{[-0.036(f_{ck}-20)]}\mu_{fc}$ [a,c]
Steel yielding strength (f_y) $[MPa]$	Log-normal [d]	$1.09 \cdot f_{yk}$	0.05	$\mu_{fy} \cdot 0.05$
Cross-section dimensions (d', b, h) [e] $[cm]$	Gaussian	Design nominal values	σ/μ	0.50 [f]
Live load (G) $[kN]$	Gaussian	$1.05 \cdot G_{k,50}$ [g]	0.10	$\mu_G \cdot 0.10$
Dead load (Q) $[kN]$	Gumbel	$0.24 \cdot Q_{k,50}$ [h]	0.65	$\mu_Q \cdot 0.65$
Resistance modeling error (θ_R) $[dimensionless]$	Log-normal [i]	1.00	0.05	0.05
Load modeling error (θ_S) $[dimensionless]$	Log-normal [i]	1.00	0.05	0.05
Temperature (T) $[°C]$	Gaussian	Obtained by ISO 834 [27]	0.45 [j]	$\mu_T \cdot 0.45$

a) Expression extracted from Leite and Gomes [31]; b) A variable coefficient of variation of the compressive strength for concrete is considered such that it decreases as the compressive strength increases, representing the variability of the resistances associated with production control; c) f_{ck} should be in $\mathrm{M}Pa$ for σ_{f_c} evaluation; d) As presented by Machado [32], the Log-normal distribution is more adequate for this random variable; e) d' is the distance from the reinforcement CG to the concrete face; f) Presented value by Damas [33]; g) Expression presented in Galambos et al. [34]; h) This consideration is made in Coelho [11], whose objective is to transform the maximum loading statistic corresponding to a service life of 50 years into an arbitrary instant, consistent with a load in a fire situation; i) As presented in Ellingwood and Galambos [35]; j) Value shown in Eamon and Jensen [6]-[8].

The design loading used in the analyses (for the generation of the live and dead portions of the load) was determined by the partial safety coefficients method, taking the following values: (*i*) coefficient of reducing the strength of concrete γ_c=1.40; (*ii*) coefficient of reducing the strength of steel γ_s=1.15; and (*iii*) coefficient of increase of loads γ_f=1.40. In this way, it is possible to estimate the safety level intrinsic to the design methodologies of reinforced concrete columns in fire situation.

In Preuss and Gomes [36] the same algorithm is used to estimate the ultimate load, but at room temperature. In this work, it is possible to observe that the rectangular-parabola diagram, without considering the tensile strength of concrete, using f_{cm} and f_{ym}, is quite efficient to estimate the ultimate load of rupture of the studied columns, obtaining an average of the ratio between the experimental and numerical results equal to 0.967 and a coefficient of variation, 0.038, which justifies the use of this same strategy in this paper.

Figure 8 illustrates the studied columns for the reliability analysis, which present a square cross-section, being idealized as simply supported on both axes, presenting biaxial bending with compression. The nominal design values adopted were 30 cm for the cross-sectional side, and 4 reinforcement bars of 25 mm in diameter and with 35 mm of concrete cover. The deterministic value of the modulus of elasticity is 210 GPa, and the value of characteristic yield strength of reinforcement, f_{yk}, is 500 MPa.

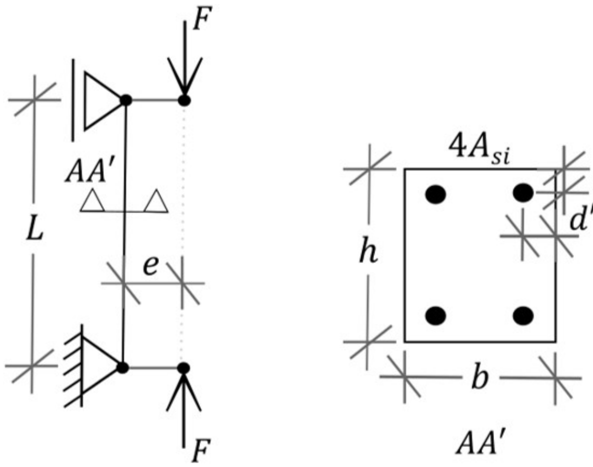

Figure 8. Geometry and boundary conditions for the studied columns.

Probabilistic analyses were performed using FORM due to the low computational cost and relative accuracy of the results compared to some results obtained by Monte Carlo via Importance Sampling (MCIS). To illustrate one of the tests performed, the reliability index, β, is analyzed using the two techniques mentioned above, for a column with geometry and static scheme as shown in Figure 8, with the following features: length of 259.81 cm, base and height of the cross-section equal to 32 cm, 4 reinforcement bars of 20 mm in diameter, f_{ck} equal to 25 MPa, f_{yk} equal to 500 MPa, first-order eccentricity of the loading equal to 3 cm in the two main axes of the cross-section, d' equal to 3.5 cm, , ratio between live and dead loads equal to 2, fire duration time equal to 60 minutes, and 4 section faces exposed to fire. The value of β obtained by FORM was 1.6297, showing a difference of 0.18% with respect to the value obtained by MCIS (equal to 1.6267). For problems solved by MCIS, the probability of failure, P_f, is determined, and the reliability index is evaluated using the cumulative normal distribution function, $\beta = -\Phi^{-1}(P_f)$.

In this paper, FORM was coded with the derivatives of the limit state function calculated using the forward finite difference method. The perturbation was taken equal to $1 \cdot 10^{-2}$ and was carried out in standard normal space. The tolerance in reliability index to stop iterations was adopted equal to $1 \cdot 10^{-2}$, and for the initial guess estimate for iteration the mean value vector was used. For the MCIS, sampling by Latin Hypercube was used to generate the random numbers, along with the adaptive technique, with variance reduction by the antithetic variables technique. The coefficient of variation of the failure probability was adopted as a statistical convergence criterion, and the simulations were performed until the value was less than or equal to 5%, with a sampling function updated every 50 simulations.

9 PARAMETRIC ANALYSES AND RESULTS

A total of 266 different combinations, 133 each type of analysis (with 2 and 4 faces exposed to fire), of the input parameters were analyzed to perform the parametric analyses of columns subjected to biaxial bending-compression exposed to a fully developed fire up to 4 hours of exposure. The distributions shown in Table 2 were employed, in addition to the reference section presented in Figure 8. The reliability results for different combinations of compressive strength of concrete, cover, first-order relative eccentricity, ratio between live and dead loads, reinforcement ratio, and slenderness index are presented in Figures 9 and 10. By analyzing the two Figures, it can be seen that the reliability decreases in a non-linear fashion over time. Furthermore, it can also be observed that for developed fires, the columns with only two faces exposed to fire showed considerably higher reliability results.

Figure 9 presents the results of the parametric analyses for the columns with two faces exposed to fire. Figure 9a shows the influence of the reinforcement ratio on the reliability results. Figure 9b, Figure 9c, Figure 9d, Figure 9e and Figure 9f show the corresponding influence of the slenderness index, the first order relative eccentricity, the concrete cover, the concrete compressive strength, the dead to live loading ratio, respectively. Figure 10 shows the same graphs, but with respect to columns with 4 faces exposed to fire, in the same order. The deterministic parameters for each analysis are shown in the figures.

Analyzing Figures 9a and 10a it is possible to notice the effect of the reinforcement ratio on the reliability index. It can be observed that as the ratio increases, the reliability index decreases. This effect occurs because in sections with high reinforcement ratios the design load is higher and results in more violations of the limit state function.

Observing Figures 9b and 10b, it can be seen that the results of the reliability index decrease as the slenderness index increases. This effect was observed in all the cases studied. Thus, it is observed that the more slender the columns are, the more unsafe they tend to be, limited to a slenderness index 90. Combining the general and equilibrium methods it is possible to study columns up to a slenderness index equal to 200, given the imposition of NBR 6118 [14]. However, this will be subject for future works.

In Figures 9c and 10c, it is shown how the increase in first order eccentricity influences reliability. It can be notice that the decrease in reliability was greater from the first to second investigated eccentricity (from $\frac{e_x}{h} = 0.1$ and $\frac{e_y}{h} = 0.1$ to $\frac{e_x}{h} = 0.2$ and $\frac{e_y}{h} = 0.1$) and then between the second and the third investigated eccentricity (from $\frac{e_x}{h} = 0.2$ and $\frac{e_y}{h} = 0.1$ to $\frac{e_x}{h} = 0.2$ and $\frac{e_y}{h} = 0.2$). This is due the fact a larger eccentricity reduces the design load more significantly.

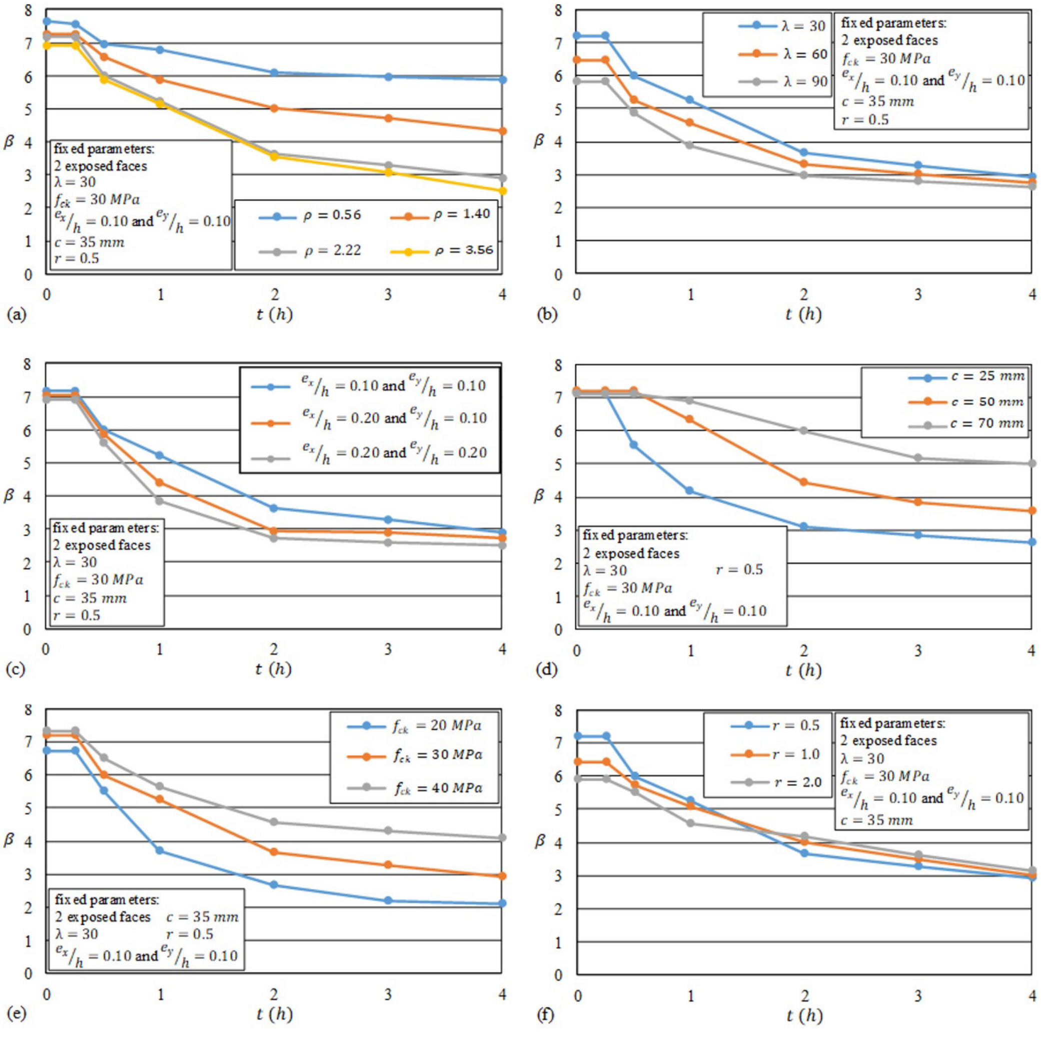

Figure 9. Parametric analyses of columns with 2 faces exposed to fire.

The effect of the concrete cover on the reliability, which is the parameter that presents more importance in the design standards in fire situations, can be seen in Figures 9d and 10d. It is noticed that for the concrete cover thicknesses studied, the reliability results considerably increases with their increase. This explains the importance given by the design standard codes to this variable as it is an effective parameter to an overall increase in safety.

Verifying Figures 9e and 10e, it can be seen that the reliability index increased with the increase in the concrete compressive strength, for all cases analyzed. This proves that concretes in group I, with higher strengths present a safer design.

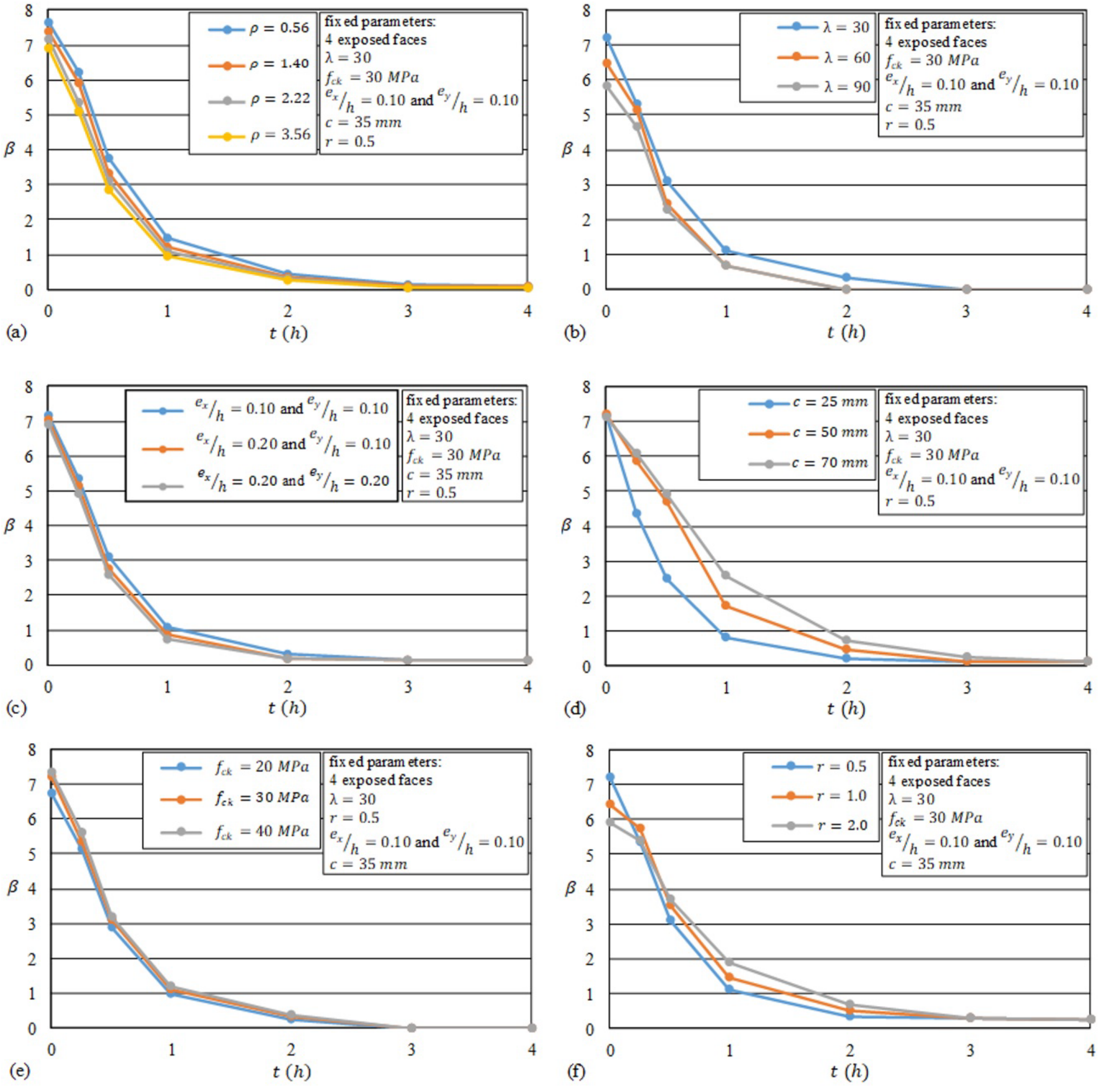

Figure 10. Parametric analyses of columns with 4 faces exposed to fire.

Analyzing Figures 9f and 10f, it is possible to observe the effect of the ratio between live and dead loads on reliability. In this case, for the fire situation a different effect was observed than the one observed in columns at room temperature presented in Preuss and Gomes [36], in which the reliability decreases as the ratio between the loads increases. In the fire situation, for columns with two faces exposed to fire, up to one hour of exposure to fire, the higher the load ratio, the lower is the reliability, but above one hour of exposure to fire, it is observed that the higher the load ratio, the higher is the reliability. This behavior in the fire situation occurs because the accidental load is being calculated for an arbitrary instant in time, differently from what is done for room temperature. For columns with four faces exposed, above 15 minutes of fire exposure, it is already observed that the higher the load ratio, the higher is the reliability.

Regarding the sensitivity of the variables (evaluated as a function of the direction cosine), the live load was the most important parameters up to 1 hour of exposure (with a direction cosine close to 0.4); however, for times longer than 1 hour, the temperature was of major importance (with a direction cosine above 0.8 in many cases).

10 CONCLUSIONS

In this work it was presented the reliability analysis of RC columns subjected to a biaxial bending-compression, subjected to fire, by a simplified approach that combines the General, Equilibrium and the Isotherm at 500°C Methods.

The algorithm used allows the consideration of physical and geometric nonlinearities, following the normative recommendations for design of RC columns in fire situation according to NBR 6118 [14] and NBR 15200 [13]. In relation to the probabilistic analyses, the results were obtained using FORM, which had some results validated with MCIS in the algorithm tests.

It was studied cases of columns with two and four faces exposed to a fully developed fire, where it was possible to observe that the reliability index decreases in a non-linear manner over time. In the studied examples, this evaluation was done up to four hours of fire exposure. The random variables considered were the dead and live loads, steel and concrete material strengths, section geometry, model error, and temperature. The graphs presented in the parametric analyses show how the reliability index variation occurs for a fully developed fire, considering live load for an arbitrary instant in the time. In these analyses, it was possible to notice how each of the investigated parameters influences the overall safety.

The main conclusions, based on the simulations of RC columns in fire situation performed in the present study, indicate that: *(i)* as the reinforcement ratio increases, reliability index decreases; *(ii)* the reliability index decreases as the slenderness index increases; *(iii)* increasing first order eccentricity decreases the reliability index; *(iv)* the reliability results increased with increasing concrete cover; *(v)* the reliability index increased with increasing concrete compressive strength; and *(vi)* the influence of the ratio between live and dead loads is different that observed in ambient temperature, because the increase in the ratio resulted in an increase in the reliability in some situations.

So far, there is little research dealing with the target safety levels during fire exposure of reinforced concrete elements, and this determination is beyond the focus of this article. Even in the standards NBR 15200 [13] and EN 1992-1-2 [20] it is not indicated what would be this target reliability value in fire situation. Thus, a continuation of this work is determination of the target reliability indices of these columns, so that in later step, the safety coefficients can be calibrated to meet the desired reliability. For this, statistical data should be used to represent the frequency of occurrence of fire, besides taking into account the existence of fire-fighting devices, the type of occupancy of the building, loss of life and the costs involved in the event of a fire accident. A study of the optimal cover that leads to the target reliability index can also be performed. For the problems investigated in this paper, it is recommended that an effective way to substantially reduce the probability of failure in columns can be simply by increasing the concrete cover.

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