A: XXXIX-0000

RELIABILITY ASSESSMENT OF REINFORCED CONCRETE COLUMNS IN FIRE SITUATION

AVALIAÇÃO DA CONFIABILIDADE DE PILARES DE CONCRETO ARMADO EM SITUAÇÃO DE INCÊNDIO

Arthur C. Preuss (P) (1); Herbert M. Gomes (2)

(1) B.Sc., M.Sc. in Civil Eng. Candidate, Graduate Program in Civil Engineering/UFRGS, Porto Alegre, Brasil.
(2) Dr. Professor, Graduate Program in Civil Engineering/UFRGS, Porto Alegre, Brasil.
Mailing address: arthurcaneda@hotmail.com; (P) Presenter

Thematic area: Structural damage, effects of earthquakes, impacts, fire, and other accidents.

Abstract

This paper proposes a methodology for verification of reinforced concrete columns subjected to fire by combining the General and Equilibrium Methods for verification of columns with the Isotherm Method at 500°C. In addition, a Structural Reliability analysis of reinforced concrete columns in fire situation developed through the First Order Reliability Method (FORM) and Monte Carlo simulation is also presented. The developed study is based on an algorithm that enables the verification of reinforcement concrete columns subject to biaxial bending-compression situations, considering the physical and geometric nonlinearities according to NBR 6118 (2014) normative recommendations. The parameters of material degradation along the fire are determined according to NBR 15200 (2012). The random variables considered are the load actions (dead and live), strength of materials (steel and concrete), section geometry, model error, and temperature. The results shows that the most important parameters in the probability of failure are temperature, the ratio between accidental and permanent load, and the compressive strength of the concrete.
Keywords: Reliability Analysis, Reinforced concrete columns, Fire.

Resumo

Este artigo propõem uma metodologia de verificação de pilares de concreto armado sujeitos ao fogo através da combinação dos Métodos Geral e do Equilíbrio para verificação de pilares com o Método das Isotermas de 500°C. Além disso, também é apresentada uma análise de Confiabilidade Estrutural de pilares de concreto armado em situação de incêndio desenvolvido através do Método de Confiabilidade de Primeira Ordem (FORM) e da simulação de Monte Carlo. O estudo é desenvolvido a partir de um algoritmo que possibilita a verificação de pilares de concreto armado sujeitos à flexo-compressão oblíqua, levando-se em conta as não-linearidades física e geométrica de acordo com as recomendações normativas da NBR 6118 (2014). Os parâmetros de degradação dos materiais ao longo do incêndio são determinados conforme a NBR 15200 (2012). As variáveis aleatórias consideradas são as ações (permanente e variável), resistência dos materiais (aço e concreto), geometria da seção, erro do modelo e a temperatura. Os resultados demostram que os parâmetros de maior importância na probabilidade de falha são a temperatura, a razão entre carga acidental e permanente, e a resistência a compressão do concreto.
Palavras-chave: Análise de confiabilidade, Pilares de concreto armado, Incêndio.

1. INTRODUCTION

There are few references on the reliability of concrete elements subject to fire, due to the analysis of the element involving both a thermal and structural analysis. This ends up limiting the possibility of using more robust methods, such as finite elements, because the simulations of the multiphysical (thermos-structural) problem have a high computational cost. Starting with a more simplistic approach, which employ simplified methodologies, there are studies by Eamon and Jensen (2012) that studies the reliability of prestressed concrete beams subject to fire, and Eamon and Jensen (2013a) and (2013b) that evaluates the reliability of reinforced concrete (RC) beams and columns in a fire situation. When it comes to Brazilian studies, there is a M.Sc. Dissertation by Coelho (2018), which employs simplified expressions for the analysis of the reliability of RC beams sections subject to fire, using parameters defined in the NBR 15200 (2012) Brazilian standard.

In this article, the reliability of RC columns in a fire situation will be investigated using a simplified methodology that considers (using either the General or the Equilibrium Methods), an effective RC cross-section obtained by the Isotherms Method at 500°C with the appropriate corrections for mechanical properties of steel reinforcements/concrete due to high temperatures. The First Order Reliability Method (FORM) and the Importance Sampling Monte Carlo (ISMC) are used to evaluate the reliability at different times of a fully developed fire, ranging from 0 to 4 hours of exposure. It will be shown that the most important parameters in the probability of failure are temperature, the ratio between accidental and permanent load, and the compressive strength of the concrete. Finally, the results for several situations for a time (supposed to be lower than the Required Time of Fire Resistance – TRRF – Tempo Requerido de Resistência ao Fogo), the reliability resulted below target values defined by technical literature. In addition, this article brings novelty when assessing the reliability of RC columns at fire situations following Brazilian standard requirements.

2. THE WICKSTRÖM METHOD

The method was developed based on a curve fit of a series of finite element analyses of concrete sections exposed to fire, which allowed the evaluation of temperatures in both steel bars and concrete. Knowing the concrete composition, the temperature is estimated by the distance from the point of analysis to the exposed surfaces and the exposure time. The gas average temperature in the environment is estimated by the fire curve adopted in the analysis for a defined exposure time. The equations presented in Wickström (1986) were obtained through the application of the Finite Element Method with the TASEF-2 Code, and it was observed that for regular sections the results obtained through analytical expressions are very close to the numerical results. The increase in temperatures for a one-dimensional heat flow (with one exposure side), represented in Figure 1(a), is given by Equation 1.

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

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

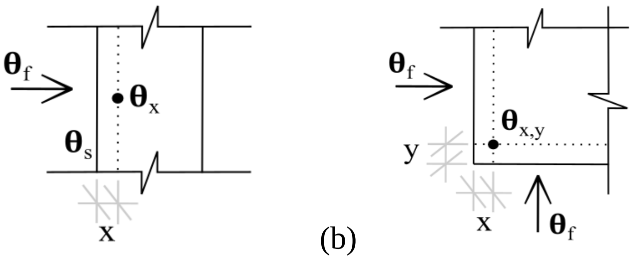


Figure 1. Exposed sections to a one-dimensional (a) and two-dimensional (b) heat flow.

For the two-dimensional flow, whose situation is illustrated in Figure 1(b), there is the increase in temperatures given by Equation 2.

$$\Delta\theta_{x,y} = \left(n_w\left(n_x + n_y - 2n_x n_y\right) + n_x n_y\right)\Delta\theta_f, \tag{2}$$

where n_x e n_y represent the relationships between the increase in surface temperature and a point inside the section. The n_x (or n_y changing y by x) is given by Equation 3.

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

where:

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

where a is the thermal diffusivity for concrete and a_c is a reference value equal to $0.417\cdot 10^{-6}\ m^2/_s$; x is the distance to the point (meters); t is the time (hours) and n_w is defined by Equation 5.

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

3. ISOTHERM METHOD AT 500°C

This method can be applied in the sizing of sections subject to single or biaxial bending. It is assumed that the heated concrete up to 500°C is not significantly affected by temperature (Costa 2008). As the incidence of heat flow happens at the faces, the areas that will eventually be discarded are superficial, and the resulting section is corresponding to a fictitious cross-sectional area. Using the Wickström Method to calculate the distances of the isotherm of 500°C in relation to the exposed faces, Equations 6 and 7 are used.

$$x_{500} = \sqrt{\frac{\frac{a}{a_c}t}{\exp\left(4.5 + \frac{480}{0.18 n_w \Delta\theta_f}\right)}} \tag{6}$$

$$x_{500,2} = \sqrt{\frac{\frac{a}{a_c}t}{\exp\left[4.5 + \frac{n_w - \sqrt{n_w^2 - (2n_w - 1)\frac{480}{\Delta\theta_f}}}{0.18(2n_w - 1)}\right]}} \tag{7}$$

This situation is illustrated in Figure 2(a), where the effective polygon of the reduced section of concrete that will be taken into account for sizing is represented. It should be noted that the approach by a polygonal to represent the corners is in the side of safety, and this procedure is recommended in Purkiss and Li (2014).

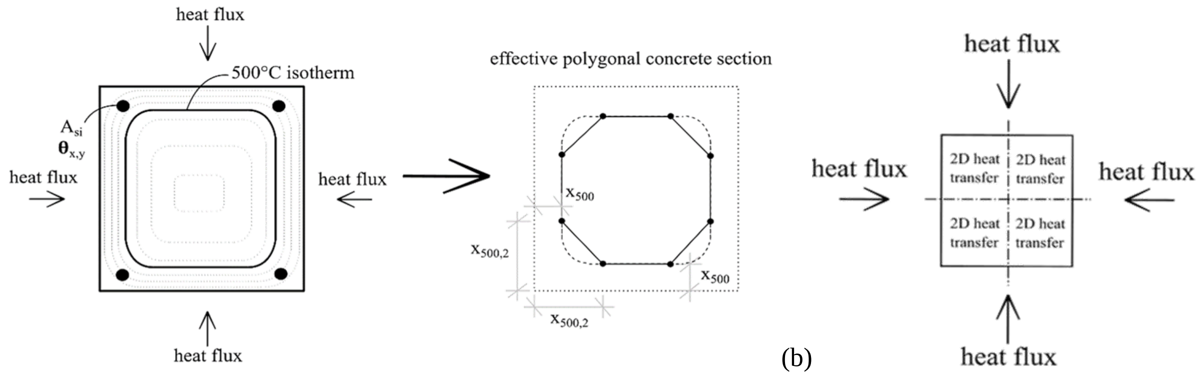


Figure 2. (a)Approximation of the isotherm of 500°C by a polygon. (b) Boundary conditions for heat in a column section with four exposed faces.

4. STANDARD CRITERIA

4.1. Ultimate Limit State

For the evaluation of the limit state function, two failure criteria are used: (*i*) first, if there was failure at the cross-section level it is said that the Equilibrium Ultimate Limit State was reached, (*ii*) second, if the failure occurs due to instability, which is named Instability Ultimate Limit State. According to NBR 6118 (2014), both Ultimate Limit States are connected to collapse or any other form of structure ruin.

The stress-strain diagram idealized for the concrete is presented in Figure 3, where for regular class concretes (strength less than C50) the strain limits ε_{c2} and ε_{cu} are equal to 2 ‰ and 3.5 ‰, respectively.

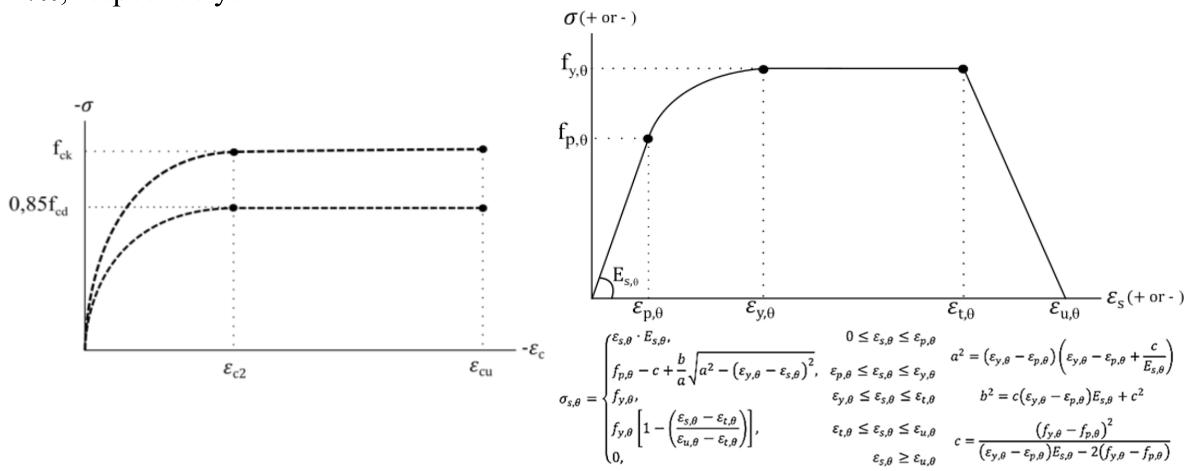


Figure 3. Material behavior: concrete (left) and steel (right).

In the same figure, the multilinear stress-strain diagram of steel in a high temperature situation can also be observed, where four distinct stages can be observed, being observed in the first section a linear behavior, in the second, elastoplastic with hardening of the material, in the third, the flow, and in the last, the decrease of the tension until rupture. The strains $\varepsilon_{sy,\theta}$, $\varepsilon_{st,\theta}$ and $\varepsilon_{su,\theta}$ are assumed as 0.02, 0.15 e 0.20, respectively. This constitutive relationship for steel is presented by EN 1992-1-2 (2004). The temperature-dependent expressions on the bars are shown in Figure 3.

4.2. General Method

This is the method recommended by NBR 6118 (2014) for sizing columns which considers the actual moment-curvature relationship in each section and the geometric nonlinearity is considered in a simplified way by second order moments due to deformed column.

4.3. Equilibrium Method

This method, when used in combination with the General Method, determines if the column is stable without determining the critical load. In this approach, it can be determined whether the horizontal deflection of the discretization's sections results in a stable situation for a certain load. The method computes the (second order) loads along the column iteratively from its deformed configuration, allowing the appropriate curvatures to be determined and the displacements to be obtained by integrating the curvatures.

4.4. Fire Curve

The standard ISO 834 fire curve presented by Equation 8 was used. This curve presents a monotone increasing gas temperature over the exposure time.

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

4.5. Reduction Factors for Steel Properties According to NBR 15200 (2012)

With the increase in temperature, the strength, modulus of elasticity and yielding limit of the steel are reduced according to coefficients. This effect is described as $f_{yk,\theta} = \kappa_{s,\theta} \cdot f_{yk}$, $E_{s,\theta} = \kappa_{sE,\theta} \cdot E_s$ and $f_{p,\theta} = \kappa_{p,\theta} \cdot f_{y,\theta}$, where sub-index θ refers to high temperatures. That is, as the temperature increases in the bars, these quantities are reduced, simulating the effect of material degradation. The coefficients are obtained by linear interpolation of the results presented in NBR 15200 (2012).

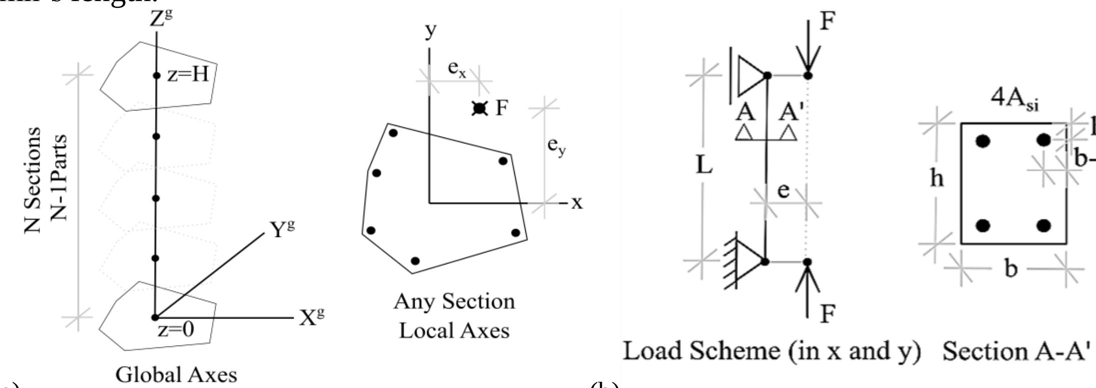
5. METHODOLOGY

5.1. Algorithm Employed

Cadamuro Junior (1997) presents the algorithm for evaluating the limit state function, in this case an Ultimate Limit State for collapse. This algorithm allows for the design and verification of RC columns with polygonal cross-section in compression and biaxial bending. As mentioned, it is taken into consideration physical and geometric nonlinearities by the Equilibrium Method and the General Method, as recommended by NBR 6118 (2014).

The curvatures, horizontal displacements, and second order moments of the discretized sections are determined using the General Method. Furthermore, it is assumed that the curvature varies linearly between two consecutive cross-sections as a simplifying assumption. The transverse displacements are calculated using a double numerical integration of the differential equation of curvature along the column's length, with the rotations determined from a first integration. The estimated displacements is allowed to have a cubic variation thanks to this approach. The second order bending moments, a key data for the iterative procedure, are calculated taking into consideration the horizontal displacements.

Figure 4(a) illustrates how to discretize a column with this approach. Along its height, the column is divided into N segments. Each segment can have any polygonal cross-section shape, with internal voids or reinforcing configuration allowing an arbitrary RC concrete sections along the column's length.



(a) (b)

Figure 4. (a) Scheme for the column discretization. (b) External loads and geometry for the studied column.

5.2. Methodology for Thermostructural Analysis

The sizing of a column member in fire situation follows a simplified method as at room temperature, provided that the effects of temperature on strength of materials are considered. Here, the reliability of RC columns is evaluated with four faces exposed to fire as shown in Figure 2(b). Thus, the 2D heat flow will affect steel strength as explained by Equation 2, and the effective cross-section of concrete is obtained by Equations 6 and 7. Briefly, the strategy in this article uses ISO 834 fire curve to obtain room temperatures and member's internal temperatures for a specific time. Concrete surfaces with temperatures above 500°C are disregarded and the properties of steel are modified according to NBR 15200 (2012) recommendations. The column is then analyzed as usual using General and Equilibrium Method. It should be mentioned that concrete spalling is not considered as well as all stress loads from temperature dilatation, as they are very small in the fire situation due to large plastic deformations.

6. PROBABILISTIC ANALYSIS

A limit state function (Equation 12) that reflects the failure condition must be defined in a reliability analysis. Here, it is defined as the difference between the resisting ultimate load and the external loads. Modelling uncertainty for loads and resistance are assumed in the formulation.

$$g(\boldsymbol{X}) = R(\boldsymbol{X}) \cdot \theta_R - S(\boldsymbol{X}) \cdot \theta_S \qquad (12)$$

where: $R(\boldsymbol{X})$ mean the resisting ultimate column load (biaxial bending and compression, completely correlated); θ_R means the model uncertainty; θ_S is the load uncertainty; $S(\boldsymbol{X})$ represents the external loads (live and dead loads). The assumed random variables of the limit state function are declared in Table 1. The design load used in the reliability analysis was determined by traditional column design with the use of partial safety coefficients.

Figure 4(b) shows the column system, a simply supported column, in both axes, with biaxial bending and compression (eccentric load). The square cross-section has 30 cm nominal value, with 4 bars with diameter 20 mm. The value of the modulus of elasticity used was 210 GPa.

The probabilistic analysis was performed using the FORM (validated previously with ISMC runs), due to low computational cost involved in solving problems with relative accuracy. The negative reliability index values were obtained from ISMC and represent failure probabilities greater than 0.5.

Table 1. Statistical properties of random variables.

Random Variable	**Probability density function**	**Mean value (μ)**	**Coefficient of variation (V)**	**Standard deviation (σ)**
Concrete compressive strength (f_c)	Normal	$\mu_{fc} = \dfrac{f_{ck}}{1 - 1.65 \cdot V_{fc}}$	0.10	$\sigma_{fc} = \mu_{fc} \cdot V_{fc}$
Steel yielding strength (f_y)	Log-normal	$\mu_{fy} = 1.09 \cdot f_{yk}$	0.05	$\sigma_{fy} = \mu_{fy} \cdot 0.05$
Cross-section dimensions (d, b, h)	Normal	Nominal design value	σ/μ	0.5
Dead load (G)	Normal	70% of characteristic load action at ambient temperature value	0.10	$\mu_G \cdot 0.10$
Live Load (Q)	Gumbel	70% of characteristic load action at ambient temperature value	0.25	$\mu_Q \cdot 0.25$
Model and Load Uncertainty (θ_R, θ_S)	Log-normal	1.00	0.05	0.05

7. PARAMETRIC RESULTS AND ANALYSIS

A total of 63 different combinations of input parameters were analyzed to perform the parametric analyses of columns subject to oblique bending-compression exposed to a fully developed fire up 4 hours of exposure. The distributions shown in Table 1 were employed, in addition to the reference section already presented.

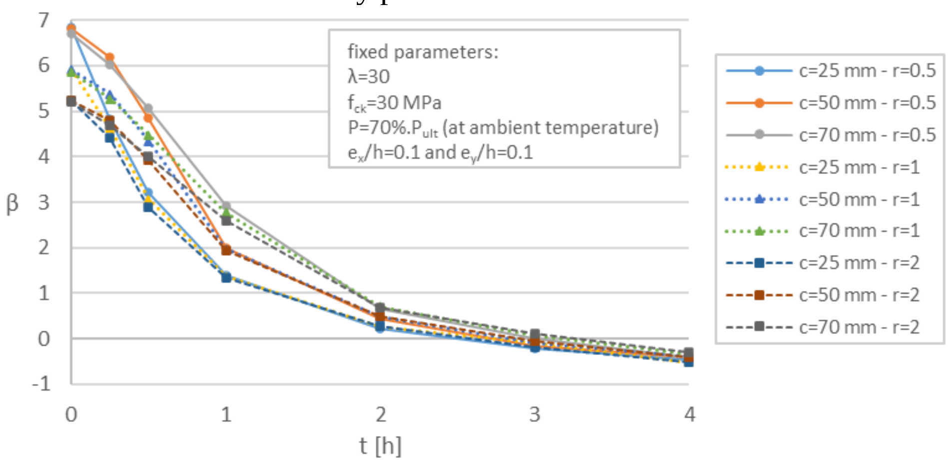


Figure 5. Reliability results for different combinations of cover, load ratio and exposure time.

Figure 5 presents the reliability results for different combinations of cover, load ratio and exposure time. It can be seen that after two hours of exposure to fire, for all the analyses performed, the reliability index resulted bellow 1. This shows that the steel degradation coefficients of NBR 15200 (2012) are either too conservative (with low values), or that in fact the structures designed in fire situation are too unsafe for exposure times longer than 1 hour. This question should be further investigated via experimental testing. As the strategy employed in this paper aims to evaluate the reliability obtained through a design methodology, a change in such coefficients would result in a change in the reliability results. In general, the results for all simulations performed above 2 hours of exposure, presented inadequate, because the reliability index resulted very close to 0, and in many cases presented a negative value that indicates a probability of failure greater than 50 %, which is not admissible for a case of structure with TRRF greater than or equal to 1 hour. Up to 1 hour of exposure to fire, the increase of the cover proved to be an efficient design parameter for increasing reliability.

Regarding the sensitivity of the variables (evaluated as a function of the director cosine), the load ratio was the most important parameter up to 1 hour of exposure (with a director cosine close to 0.4); however, for times longer than 1 hour, the temperature was of greater importance (with a director cosine above 0.8 in many cases). Moreover, the compressive strength of concrete also proved to be of great importance in the analyses, presenting in most cases a director cosine close to 0.25.

8. FINAL REMARKS

In this paper, an analysis of the reliability of RC columns exposed to fire was presented, following the recommendations of NBR 6118 (2014) and NBR 15200 (2012). It was shown that reliability depends on several factors, such as load ratio, cover, material properties, model error and exposure time. In general, the results of the reliability indices after 1 hour of exposure turned out to be very low and inadequate for structures (depicting a high propensity to failure). This issue should be further investigated, and the degradation coefficient values of the materials should be compared via experimental testing so that further reliability studies can be conducted.

ACKNOWLEDGEMENTS

The authors wish to thank CAPES and CNPq Councils for their financial support to this work.

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