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ORIGINAL ARTICLE

A contribution to the study of flexural reliability of steel-concrete composite beams designed according to NBR 8800:2008

Uma contribuição ao estudo da confiabilidade à flexão de vigas mistas de aço e concreto projetadas segundo a NBR 8800:2008

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Abstract: This work assesses the flexural reliability of compact sections and full-interaction steel-concrete composite beams designed according to NBR 8800:2008. Reliability indexes are obtained for different grades of concrete, slab heights, and load combination. The results show a lower level of safety in situations where live loads are predominant in the load combination. A partial safety factors calibration procedure is also proposed based on a target reliability index $\beta_T = 3.5$. It is shown that the calibrated partial safety factors reduce the variability of reliability indexes in the proposed design situations.

Keywords: steel-concrete composite beams; structural reliability; NBR 8800; FORM; PSO.

Resumo: Este trabalho avalia a confiabilidade à flexão de vigas mistas aço-concreto com seção compacta e interação completa dimensionadas conforme prescrições da NBR 8800:2008. Os índices de confiabilidade são obtidos para diferentes classes de concreto, alturas de mesa comprimida e combinações de ações. Os resultados obtidos indicam um nível de segurança inferior nas situações em que as ações acidentais predominam na combinação de ações. Propõe-se também a calibração dos coeficientes parciais de ponderação com base em um índice de confiabilidade alvo $\beta_T = 3.5$. É evidenciado que os coeficientes parciais de ponderação calibrados reduzem a variabilidade dos índices de confiabilidade nas situações de projeto analisadas.

Palavras-chave: vigas mistas aço-concreto; confiabilidade estrutural; NBR 8800; FORM; PSO.

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1 INTRODUCTION

Steel-concrete composite beams are a solution obtained by combining a structural steel element with a concrete slab, the connection between these two components being carried out through so-called shear connectors. Alva [1] highlights that the association of concrete and structural steel enables solutions that explore the individual structural potential of each material, allowing the design of structures with great resistance and stiffness, able to transpose long spans with construction speed and good cost-benefit relation. Figure 1 illustrates two solutions of steel-concrete composite beams of great application, which consist of the association of an "I" section to flat slabs of reinforced concrete (Figure 1a) and to composite slabs with metal decking (Figure 1b).

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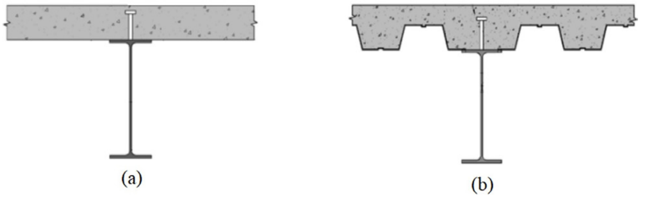

Figure 1. Steel-concrete composite beam: (a) with flat concrete slab; (b) with composite slabs with metal decking [2].

Historically, the combination of concrete and steel structures began to be used as a structural solution after World War II. The scarcity of steel in the post-war period led engineers of the time to use concrete as part of the framework [3]. Even in the first applications, concrete was used as a coating material for the steel structure, protecting it from fire action and corrosion. Despite the concrete slab's already known contribution to the overall structure behavior, this contribution was usually disregarded in the structure design [1]. In Brazil, the first constructions using composite structures, built between the 1950s and 1960s, were restricted to buildings and small bridges. Due to the increase in steel production and the need for new architectural solutions, the use of composite structures has been growing in recent years, especially in industrial, commercial, and bridge structures [4].

In Brazil, the design of steel-concrete composite structures is guided by NBR 8800 – *Projeto de estruturas de aço e de estruturas mistas de aço e concreto* [5]. More recently, NBR 16694:2020 – *Projeto de pontes rodiviárias de aço e mistas de aço e concreto* came into force, indicating a growth perspective in the use of composite structures at the national level, emphasizing the relevance of the development of research on the subject. In Brazilian structural design codes and most international ones, safety is introduced through partial safety factors called Load Resistance Factor Design (LRFD). Such partial safety factors increase loads and reduce resistances to apply an appropriate safety margin to the structure. Although several international codes have their partial safety factor calibrated based on reliability theory, Brazilian standards, so far, present their criteria fundamentally based on the requirements of international regulations [6]. Given this scenario, an effort has been made in recent years to assess the current safety level of Brazilian structural design codes and establish partial safety factors for Brazilian codes based on a target reliability level. Although several works are directed to reinforced concrete, prestressed concrete, and steel structures [7]–[20], few research focus on steel-concrete composite structures designed according to Brazilian codes. In this subject, a highlight can be directed to the study proposed by Chaves et al. [21].

This work aims to contribute to the safety study of steel-concrete composite beams under flexure. The calibration procedure of partial safety factors based on reliability theory and optimization algorithms is also discussed.

2 FLEXURAL DESIGN OF STEEL-CONCRETE COMPOSITE BEAMS ACCORDING TO NBR 8800:2008

2.1 Ultimate limit states to be considered

In steel-concrete composite beams, the flexural strength is directly dependent on the strength of the concrete slab, the steel profile, and the shear connectors responsible for the transfer of forces at the steel-concrete interface. However, the entire plastic capacity of the cross-section can not always be mobilized since the strength may be limited as a function of the ultimate limit states associated with the steel profile. These limit states are web local buckling (WLB), flange local buckling (FLB), and lateral torsional buckling (LTB). Thus, the flexural strength of a composite beam depends not only on the mechanical characteristics of the materials but especially on the geometry of the cross-section. Slenderness intervals and the ultimate limit states to be considered in the flexural verification process are established by NBR 8800:2008. This work presents the flexural design criteria of composite beams with compact cross-sections and full-interaction subject to positive bending moments.

2.2 Slenderness limits for simply-supported composite beams

In the case of simply-supported composite beams, the concrete slab supports the compressive forces due to positive bending. It also acts as a lateral containment to the upper flange of the steel profile, avoiding the FLB. In these cases, the flexural design is guided by WLB. The design procedure, according to NBR 8800:2008, depends on the so-called web slenderness (h_w/t_w). In this way, the so-called compact and semi-compact sections are defined, for which the web slenderness limits correspond to the values presented in Equations 1 and 2, respectively.

$$\frac{h_w}{t_w} \leq 3.76\sqrt{\frac{E_s}{f_y}}\ (compact\ sections) \tag{1}$$

$$3.76\sqrt{\frac{E_s}{f_y}} < \frac{h_w}{t_w} \leq 5.70\sqrt{\frac{E_s}{f_y}}\ (semi-compact\ sections) \tag{2}$$

Being h_w the height of the steel profile web; t_w the web thickness of the steel profile; f_y the steel yield strength; and E_S the Young's modulus of the steel. For the compact cross-sections, the flexural limit state will be reached with the total plastification of the steel and concrete sections before the LWB.

2.3 Design bending moment resistance (M_{Rd}) for beams with compact section

The flexural design verification of composite beams with compact cross-section and full interaction is done assuming the plastification of the steel or concrete section. The position of the plastic neutral axis (PNA) is obtained based on the equilibrium of forces acting on the cross-section so that the conditions presented in Equation 3 are met.

$$\sum F = 0\ ;\ F_{cd} = F_{td} \tag{3}$$

Where F_{cd} is the compressive force acting on the concrete slab and F_{td} is the tensile force acting on the steel profile. If R_{cd} is the maximum value of the compressive strength of the concrete section, and R_{td} is the maximum value of the tensile strength of the steel section, then if $R_{cd} > R_{td}$, the PNA is in the concrete slab (Figure 2) and, if $R_{td} > R_{cd}$, the PNA is located in the steel profile (Figure 3).

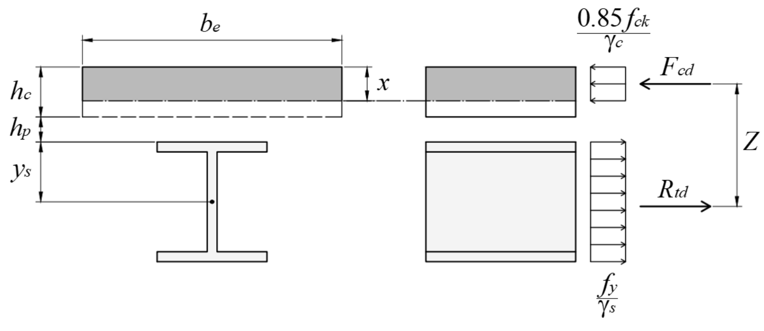

Figure 2. Stresses/forces distribution in the cross-section of a compact section and full-interaction steel-concrete composite beam (PNA in the concrete slab).

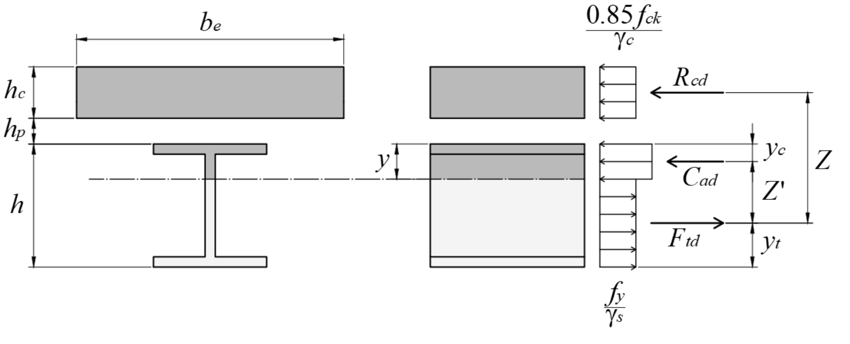

Figure 3. Stresses/forces distribution in the cross-section of a compact section and full-interaction steel-concrete composite beam (PNA in the steel profile).

The values of R_{cd} and R_{td} are obtained through Equations 4 and 5.

$$R_{cd} = \frac{0.85 \cdot f_{ck} \cdot b_e \cdot h_c}{\gamma_c} \tag{4}$$

$$R_{td} = \frac{f_y \cdot A}{\gamma_{a1}} \tag{5}$$

Being f_{ck} the characteristic compressive strength of concrete; h_c the height of the concrete slab; b_e the effective width of the concrete slab; A the cross-section area of the steel profile; γ_{a1} and γ_c the partial safety factors of the strengths for steel and concrete, respectively, taken as $\gamma_{a1} = 1.10$ and $\gamma_c = 1.40$ in the so-called *normal combinations*.

In cases where $R_{cd} > R_{td}$, the PNA and the design bending moment resistance of the cross-section (M_{Rd}) are obtained through Equations 6 and 7.

$$x = \frac{R_{td}}{0.85 \cdot f_{ck} \cdot b_e / \gamma_c} \tag{6}$$

$$M_{Rd} = R_{td} \cdot \left(y_s + h_p + h_c - \frac{x}{2}\right) \tag{7}$$

Where y_s is the distance from the geometric center of the steel profile to its top face, and h_p is the height of the rib in slabs with metal decking.

3 FLEXURAL RELIABILITY OF STEEL-CONCRETE COMPOSITE BEAMS

3.1 General aspects of structural reliability

The reliability of a structure or structural element can be understood as its ability to perform satisfactorily during a specific period and under certain conditions of use. The fact is that most of the phenomena related to engineering show a degree of uncertainty, such as the mechanical properties of materials, loads, and cross-sectional dimensions, among others. In this context, there will always be the probability of a failure, which may be related to the collapse of the structure or structural element (Ultimate Limit State, ULS) or simply to the low performance in the face of service conditions (Service Limit State, SLS).

Reliability analysis assumes the knowledge of a function that classifies structural performance as satisfactory or unsatisfactory concerning the failure mode under investigation. This function is usually called the limit-state function. For ULS, the limit-state function is generically stated as presented in Equation 8.

$$g(R, S) = R - S \tag{8}$$

Where R and S are the resistance and load effect, respectively. It is observed that the resistance will be a function of the structural element and of the materials that compose it. Therefore, $R = R\ (f_{ck}, f_y, b, h, d, \ldots)$; and the load effect will be a function of the load combination and the kind of the loads acting on the structure, that is, $S = S\ (g, q)$, being g and q the dead and live loads, respectively. From the limit-state function, the probability of failure (p_f) of the structure or structural element can be obtained by the Monte Carlo Simulation through Equation 9. This probability can be estimated by the ratio of the number of failures (N_f) observed in N computational simulations of the limit-state function, performed from the statistics and probability distributions of the random variables [22].

$$p_f = P[g(R, S) < 0] \cong \frac{N_f}{N} \tag{9}$$

The safety level, for convenience, is usually expressed by the reliability index β, which is related to the failure probability through Equation 10.

$$\beta = -\Phi^{-1}(p_f) \tag{10}$$

Where Φ represents the cumulative distribution function for the standard normal variable. Figure 4 illustrates the relationship between the probability of failure and the reliability index.

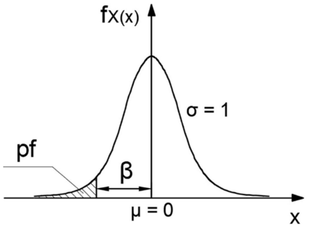

Figure 4. Relationship between the probability of failure p_f and the reliability index β.

As an alternative to Equation 10, the reliability index can be obtained directly through the First-Order Reliability Method (FORM). Initially expressed in the real space of the variables, the limit-state function, $g(\mathbf{X})$, must be rewritten for the space of standard and statistically independent normal variables, $g(\mathbf{Z})$, as expressed by Equation 11 for a generic variable X_i.

$$X_i = \mu_{Xi}^N + Z_i \cdot \sigma_{Xi}^N \tag{11}$$

Where μ_{Xi}^N and σ_{Xi}^N are the mean and standard deviation. The superscript 'N' indicates that these are values for an equivalent normal function if the original probability distribution of the variable is different from the normal distribution. Since the distance between the origin and a given point in the system of standard normal variables is given by $D = (\mathbf{z}^T\mathbf{z})^{1/2}$, the reliability index β is defined as the smallest distance between the origin of the system and the hyperplane $g(\mathbf{Z}) = 0$, being obtained by Equation 12.

$$\beta = -\frac{\sum_{i=1}^{n} z_i^* \left(\frac{\partial g}{\partial Z_i}\right)^*}{\sqrt{\sum_{i=1}^{n} \left(\frac{\partial g}{\partial Z_i}\right)^{2*}}} \tag{12}$$

In Equation 12, the superscript '*' indicates that the vector of random variables and partial derivatives are evaluated at the *design point*. As this point is not known a priori, β is obtained by successive approximations [22]. The directional cosine (α_i) and the design point in the standard normal space (z_i^*) of each random variable are obtained respectively by Equations 13 and 14.

$$\alpha_i = \frac{\left(\frac{\partial g}{\partial Z_i}\right)^*}{\sqrt{\sum_{i=1}^{n} \left(\frac{\partial g}{\partial Z_i}\right)^{2*}}} \tag{13}$$

$$z_i^* = -\alpha_i \beta \tag{14}$$

Alternatively, knowing the vector $\mathbf{z}^*$, the reliability index can be expressed by Equation 15.

$$\beta = \sqrt{\mathbf{z}^{*T}\mathbf{z}^*} \tag{15}$$

3.2 Geometry and materials properties of the proposed cross-section

The reliability study proposed in this work is carried out considering a compact composite cross-section, whose slenderness meets the presented in Equation 1 and whose design follows the step-by-step described in section 2. The proposed cross-section consists of a beam in profile VS400x49 and a flat slab with a height varying from 10.0 to 15.0 cm. It is assumed that the section presents full interaction. The yield strength of the steel profile was taken as 350 MPa, while for the concrete slab, concretes ranging from grade C20 (f_{ck} = 20 MPa) to grade C30 (f_{ck} = 30 MPa) were considered. The slab presents an effective width b_e = 200 cm. Figure 5 shows the main characteristics of the composite beam adopted in this work.

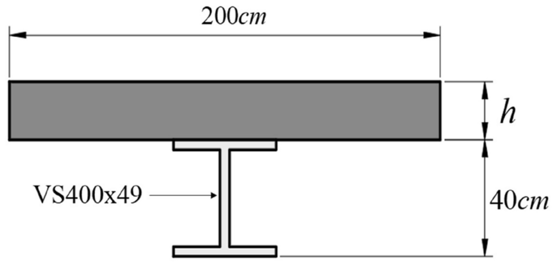

Figure 5. Steel-concrete composite cross-section considered in the reliability analysis.

Table 1 summarizes the design bending moment resistance (M_{Rd}) values for the composite beams studied as a function of the design parameters. In all design cases, the PNA resulted in the concrete slab being M_{Rd} obtained by Equation 7.

Table 1. Bending moment resistance M_{Rd} (kNm) for the proposed cross-section.

h **(cm)**	f_{ck} **(MPa)**		
	20	**25**	**30**
10.0	511.7	527.7	538.4
12.5	561.0	577.0	587.7
15.0	610.3	626.4	637.0

3.3 Statistics of random variables and limit-state function

The eight random variables presented in Table 2 were adopted to conduct the reliability analysis. The probability distributions and main statistics were obtained through the works of Santiago [7], Santos et al. [17], Ellingwood and Galambos [23], Biodini et al. [24], and Costa et al. [25].

Table 2. Statistics of random variables adopted in this work.

Random Variable	**Description**	**Type of distribution**	**Mean (μ_x)**	**Standard Deviation (σ_x)**	**Ref.**
f_c	Compressive strength of concrete	Normal	$1.17 \cdot f_{ck}$	$0.15 \cdot \mu_x$	[17]
f_y	Yield strength of the steel of the profile	Normal	$1.08 \cdot f_{yk}$	$0.08 \cdot \mu_x$	[17]
h	Concrete slab height	Normal	h_n*	5 mm	[24]
d	Steel profile height	Normal	d_n*	3 mm	[7]
M_g	Dead load bending moment	Normal	$1.05 \cdot M_{Gk}$	$0.10 \cdot \mu_x$	[23]
M_q	Live load bending moment	Gumbel	$0.92 \cdot M_{Qk}$	$0.25 \cdot \mu_x$	[25]
θ_R	Uncertainty parameter of the resistant model	Lognormal	1.00	$0.05 \cdot \mu_x$	[17]
θ_S	Uncertainty parameter of the load model	Lognormal	1.00	$0.05 \cdot \mu_x$	[17]

* h_n and d_n correspond to the nominal design values

The reliability indexes for flexure ultimate limit state were assessed by FORM, using the limit-state function given by Equation 16.

$$g(\mathbf{x}) = \theta_R \cdot M_R - \theta_S \cdot (M_g + M_q) \tag{16}$$

Where M_R is the bending moment resistance of the cross-section, obtained through Equation 17 as a function of the material's mechanical properties and the cross-section's geometry.

$$M_R = A \cdot f_y \cdot \left(d + h - 0.5 \frac{A \cdot f_y}{0.85 \cdot f_c \cdot b_e}\right) \tag{17}$$

3.4 Methodology to assess the reliability indexes

The flexural design of composite beams usually consists of determining a section that meets the design inequality presented in Equation 18.

$$M_{Sd} \leq M_{Rd} \tag{18}$$

Being M_{Rd} and M_{Sd} obtained through the partial safety factors (γ) applied to the characteristic values of the strength of the materials as well as to the load effects. In this way, after obtaining the load bending moment through a load combination, it is verified whether the cross-section meets the inequality given by Equation 18 so that, in an ideal situation (economic design), $M_{Rd} = M_{Sd}$. For the analyzes carried out in this work, the following procedure was adopted:

a) Based on the steel profile and the concrete slab, as well as the mechanical properties of the materials and their partial safety factors (γ_c and γ_{a1}), the design bending moment resistance (M_{Rd}) is obtained;
b) An economical design is assumed, resulting $M_{Rd} = M_{Sd}$;
c) A load ratio $r = M_{qk}/M_{gk}$ is assumed, and through the partial safety factors of the dead and live loads, the characteristic load bending moment arising from the dead load (M_{gk}) and live load (M_{qk}) are obtained;
d) Based on the characteristic or nominal values of the design variables and the statistics and probability distributions adopted for the random variables, the reliability analysis is carried out, and the reliability index β is determined for each proposed design situation.

Step (c) is performed using the load combination given by Equation 19. Assuming the load ratio (r), M_{gk} and M_{qk} are obtained through Equation 20.

$$M_{Sd} = \gamma_g \cdot M_{gk} + \gamma_q \cdot M_{qk} \tag{19}$$

$$M_{gk} = \frac{M_{Sd}}{\gamma_g + r \cdot \gamma_q}; \quad M_{qk} = r \cdot M_{gk} \tag{20}$$

The partial safety factors applied to the dead and live loads were $\gamma_g = 1.40$ and $\gamma_q = 1.40$, respectively. These partial safety factors are prescribed by NBR 8800:2008 [5] and NBR 8681:2003 [26] for the so-called Type 2 buildings in unfavorable load combinations in which direct dead loads are taken grouped into a single dead load and live loads that can act simultaneously are taken grouped into a single live load.

3.5 Reliability indexes obtained

The reliability indexes obtained are shown in Figures 6 to 8, considering the parametric variation of the compressive strength of concrete (f_{ck} = 20; 25; 30 MPa), the height of the concrete slab (h = 10; 12.5; 15 cm) and the load ratio (r = 0.00; 0.25; 0.50; 1.00; 1.50; 2.00). As a target index, the value $\beta_T = 3.5$ was adopted, which is discussed and justified in section 4.3.1.

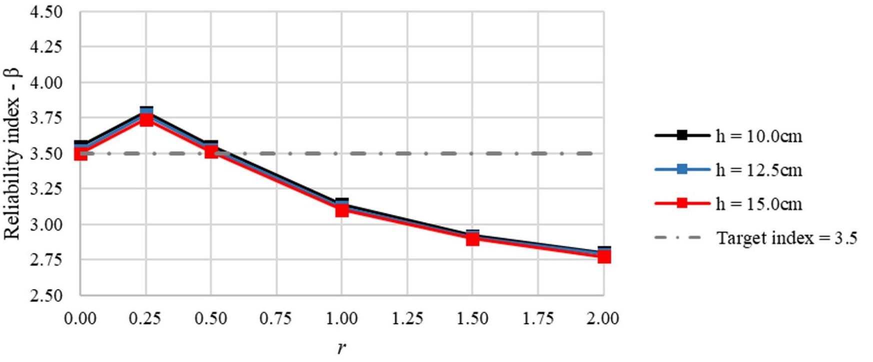

Figure 6. Reliability indexes for f_{ck} = 20 MPa, as a function of *h* and *r*.

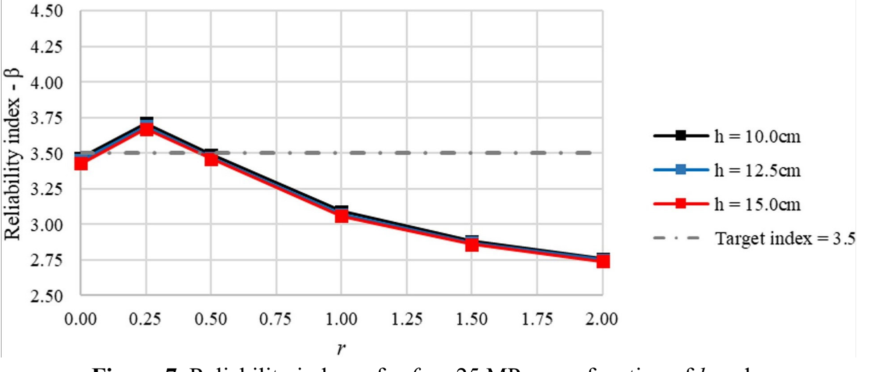

Figure 7. Reliability indexes for f_{ck} = 25 MPa, as a function of *h* and *r*.

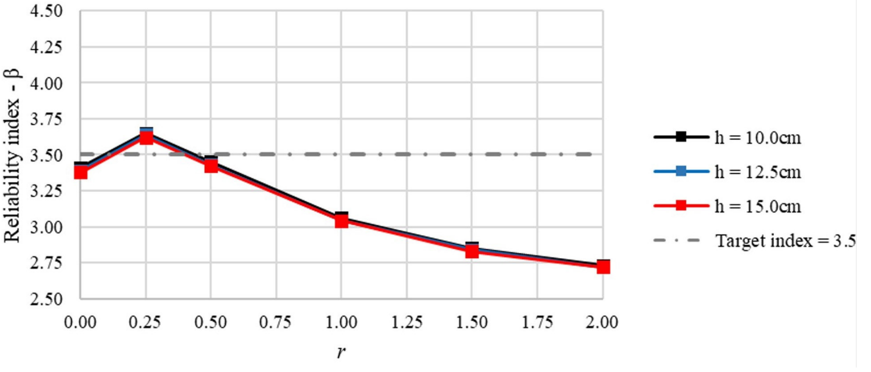

Figure 8. Reliability indexes for f_{ck} = 30 MPa, as a function of *h* and *r*.

Concerning the influence of the load combination on the reliability index, except for the situation $r = 0.00$, there is a systematic reduction in the reliability index while live loading is predominant on load combination. This behavior can be attributed to the greater variability of live load, which leads to greater uncertainty in the values of the load effect, increasing the probability of failure and decreasing the reliability index. This result suggests that, in the design stage, adopting the same partial safety factor for the dead (γ_g) and live (γ_q) loads produces lower levels of safety in the structures where the live load is predominant. For the case $r = 0.00$, in which there is only dead load acting on the structure, load distribution presents less variability but a higher mean value. This right-shifting of the load bending moment distribution causes a higher overlapping of the distribution of the bending moment resistance when compared to the case $r = 0.25$, as illustrated in Figure 9. This behavior reduced the reliability index, justifying the trend observed in Figures 6 to 8.

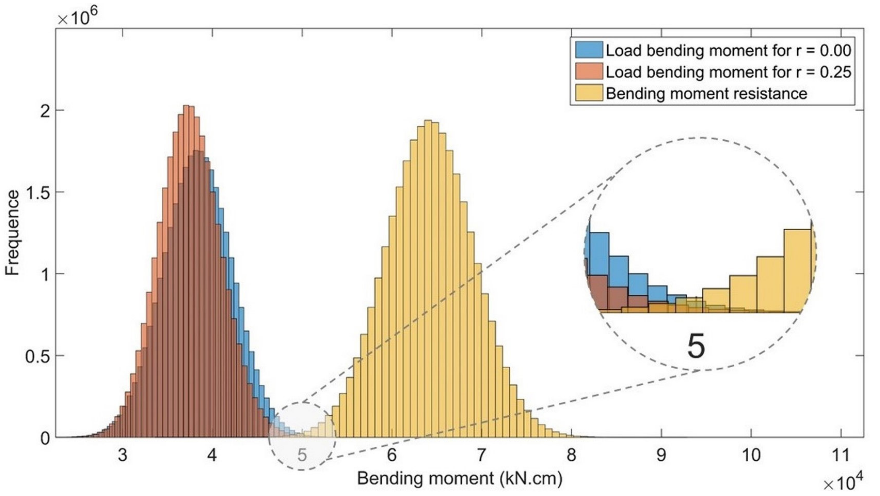

Figure 9. Comparison of M_R and M_S overlap for cases r = 0.00 and r = 0.25.

Low sensitivity of the reliability index was also observed under variations in the height of the concrete slab (h) and the characteristic compressive strength of concrete (f_{ck}). The variation of these design parameters does not significantly change the safety level reached by the composite beams analyzed.

4 CALIBRATION OF PARTIAL SAFETY FACTORS BASED ON A TARGET RELIABILITY INDEX

4.1 Contextualization of the problem

The evaluation of the reliability index of a structure about a given limit state requires the determination of the design point in the space of standard and statistically independent normal variables, which is used to assess the reliability index through the geometric interpretation of the "shortest distance" proposed by the FORM [27]. In this process, the design point and the reliability index depend on the probability and statistical distributions of the random variables and the normative partial safety factors (γ_c, γ_{a1}, γ_g and γ_q, for example). On the other hand, the calibration procedure consists of a reverse process that sets the partial safety factors to reach a target reliability index β_T [28]. The target reliability index suggested by *fib* Model Code 2010 [29] for ultimate limit states is presented in Table 3.

Table 3. Recommended target reliability indexes for structures to be designed according to *fib* Model Code 2010

Limit states		Target reliability index β_T	Reference period
Serviceability	Irreversible	1.5	50 years
		3.0	1 year
Ultimate	Low consequences of failure	3.1	50 years
		4.1	1 year
	Medium consequences of failure	3.8	50 years
		4.7	1 year
	High consequences of failure	4.3	50 years
		5.1	1 year

The main concepts of the calibration procedure for structural codes are presented in detail by Nowak and Collins [27].

4.2 Procedures for partial safety factors calibration

4.2.1 Calibration of isolated design situation based on FORM

The calibration procedure sets the partial safety factors to meet $\beta = \beta_T$ [27]. Assuming that the limit state equation $g(\mathbf{x}^*) = 0$ is satisfied, being $\mathbf{x}^*$ the so-called design point, the coordinate of each random variable can be written according to Equation 21.

$$x_i^* = \mu_{Xi}^N + z_i^* \cdot \sigma_{Xi}^N \tag{21}$$

The design point can also be expressed as a function of the FORM directional cosine and the pre-established β_T according to Equation 22.

$$z_i^* = -\alpha_i \cdot \beta_T \tag{22}$$

By combining Equations 21 and 22, Equation 23 can be obtained.

$$x_i^* = \mu_{Xi}^N - \alpha_i \cdot \beta_T \cdot \sigma_{Xi}^N \tag{23}$$

Once the design point is known, each variable x_i^* can be related to a nominal value used in the design procedure, according to design codes [28]. The main strength and load variables presented by Brazilian structural design codes are referred to in their characteristic values $x_{i,k}$. The partial safety factors of the resistance (γ_m) and load (γ_f) variables are given, respectively, by Equations 24 and 25.

$$\gamma_m = \frac{x_{i,k}}{x_i^*} \tag{24}$$

$$\gamma_f = \frac{x_i^*}{x_{i,k}} \tag{25}$$

4.2.2 Calibration based on the optimization algorithm

The calibration procedure presented in 4.2.1 allows finding a set of partial factors to reach a target reliability index in an isolated design situation. From the perspective of code requirements, it is necessary to have a set of partial safety factors that satisfactorily meet the design of all structural elements covered by the code in different design situations. This objective can be achieved by solving a minimization problem, stated as an RBDO (Reliability-Based Design Optimization) problem. In this work, the partial safety factors are calibrated through the scalar $W(\gamma)$ minimization presented in Equation 26 [8].

$$W(\gamma) = \sum_{j=1}^{L} w_j \left(\beta_j(\gamma) - \beta_T\right)^2 \tag{26}$$

Being $j = 1...L$ the design situations considered; $\beta_j(\gamma)$ the reliability index obtained for the design situation j as a function of each partial safety factor γ; β_T the target reliability index adopted for calibration purposes; and w_j the weighting factor adopted for the design situation j. To be consistent with previous calibration studies and to the practice of the current design codes, the search domain of the optimization problem can be limited by imposing constant constraints of the type $\gamma^L < \gamma < \gamma^U$, being γ^L and γ^U, respectively, the lower and upper bounds of each partial safety factor γ [30]. As a reference, γ^L and γ^U can be obtained from previous results assessed by FORM according to the procedure presented in 4.2.1 or taken in such a way as to have a box centered on the values of γ already practiced by the design codes. In general, metaheuristic algorithms are used to solve optimization problems such as the one presented in Equation 26. The PSO - Particle Swarm Optimization [31] is one of the most used and was also adopted in this work.

4.3 Calibration of partial safety factors for the design of composite beams under flexure

4.3.1 Calibration based on FORM

To improve the reliability level of concrete-steel composite beams under flexure, it is proposed the calibration of the partial safety factors γ_c, γ_{a1}, γ_g and γ_q. Five sets of partial safety factors for the load ratios r = 0.25; 0.50; 1.00; 1.50 and 2.00 are obtained according to the procedure presented in 4.2.1. In this context, the FORM-calibrated partial safety factors are taken as reference bounds values and as a first estimate of expected results for a general PSO calibration procedure.

Regarding the target reliability index β_T, Nowak and Collins [27] suggest that it can be selected based on evaluating the performance of existing structures designed by previous structural codes since they historically present satisfactory designs. According to the results presented in Figures 6 to 8, it is observed that the β values obtained for the composite beams analyzed are mainly in the range β = 3.0 to β = 4.0. Thus, in this work, the target index was adopted as an intermediate value $\beta_T = 3.5$. This target value is also the same one that Szernsen and Nowak [32] adopted for ACI 318 calibration and is still similar to those presented in Table 3. The results obtained through the calibration by FORM are shown in Table 4.

Table 4. Partial safety factors obtained by FORM.

r	Partial safety factor			
	γ_c	γ_{a1}	γ_g	γ_q
0.00	0.90	1.22	1.33	-*
0.25	0.90	1.21	1.28	1.37
0.50	0.88	1.13	1.18	1.91
1.00	0.88	1.08	1.12	2.14
1.50	0.88	1.06	1.10	2.19
2.00	0.88	1.05	1.08	2.21

* For r = 0.00, the load combination has no live load.

Concerning the partial safety factors of loads, an increase in γ_q as the load ratio r becomes higher is observed. This result indicates that a higher value of γ_q is expected to cover the more significant uncertainties due to the greater variability of live load. In the entire calibration range, except for r = 0.00, it was obtained $\gamma_q > \gamma_g$.

In all analyzed situations, γ_c was less than unity, indicating that the design point presents $f_c^* > f_{ck}$. In other words, the characteristic value f_{ck} showed to be more pessimistic for flexural failure than the coordinate f_c^* at the design point for $\beta_T = 3.5$. Similar behavior was obtained by Nogueira and Pinto [15] in the reliability analysis of reinforced concrete beams under flexure. This behavior reflects a particularity of the flexure failure mode under study, and there is little sense in adopting $\gamma_c < 1$, given that this result goes against the general concepts of the limit-state method. In this sense, γ_c obtained by FORM will not be considered representative.

The results also indicate a reduction in γ_{a1} given the increase in r. This behavior is justified since, in these situations, the safety level of the structure becomes more dependent on live load, reducing the sensitivity of the results to the yield strength of the steel.

4.3.2 Calibration based on PSO

The set of calibrated partial safety factors is obtained by solving the optimization problem presented in Equation 26. Ellingwood and Galambos [23] present recommendations for weighting factors (w_j) for different design situations in reinforced concrete and steel structures. As no specific recommendations are presented for steel-concrete composite structures, $w_j = 1.00$ was adopted for all design situations.

Given that $\gamma_c < 1.0$ was found by FORM for all load ratios r, it was chosen to set $\gamma_c = 1.40$, which is an adequate value for current Brazilian code practice. As a result of the general calibration performed through the PSO, $\gamma_{a1} = 1.16$ (1.15), $\gamma_g = 1.29$ (1.30), and $\gamma_q = 1.62$ (1.60) were obtained. The rounded values in parentheses were adopted as final results for practical purposes.

4.4 Reassessment of reliability indexes for the set of calibrated partial safety factors

The reliability analysis proposed in section 3 is again carried out to compare the results presented in 3.5 with those obtained with the new set of partial safety factors. Figures 10 to 12 illustrate the parametric behavior of the new reliability indexes obtained for $\gamma_c = 1.40$, $\gamma_{a1} = 1.15$, $\gamma_g = 1.30$ and $\gamma_q = 1.60$, as a function of r for different combinations of f_{ck} and h. To make more accessible the comparison with the results obtained in section 3, the figures below show the mean value before calibration (MVBC) silhouette. A discussion of the results is presented in section 4.5.

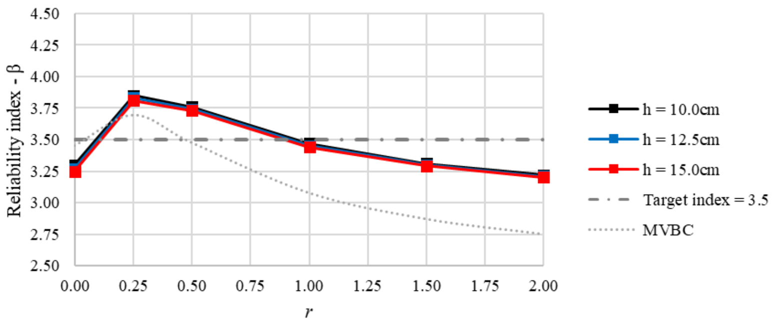

Figure 10. β x r for f_{ck} = 20 MPa and calibrated values of partial safety factors.

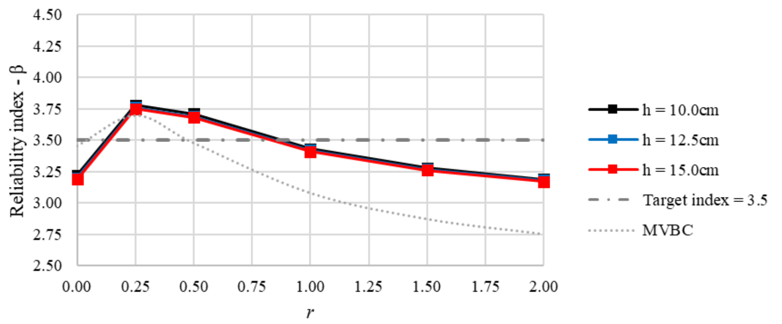

Figure 11. β x r for f_{ck} = 25 MPa and calibrated values of partial safety factors.

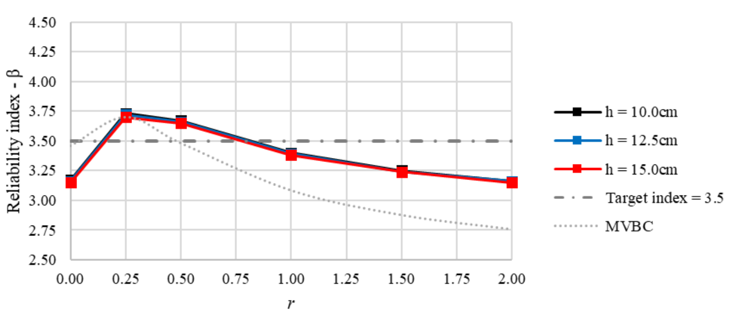

Figure 12. β x r for f_{ck} = 30 MPa and calibrated values of partial safety factors.

4.5 Discussion of results

The results presented in Figures 10 to 12 indicate that the calibration procedure reduces the variability of reliability indexes in different design situations since the results came near to $\beta_T = 3.5$. The maximum and minimum reliability indexes obtained before calibration were 3.79 and 2.72, respectively. After calibration, the maximum and minimum values obtained were 3.85 and 3.15. The coefficient of variation of the reliability index in the proposed calibration range ($0.00 \leq r \leq 2.00$) showed a reduction from 10.8%, before calibration, to 6.9%, after calibration, and the average of reliability indexes rose from 3.22 to 3.43. According to the behavior presented in Figures 10 to 12, a significant improvement in safety levels is readily observed in design situations where $r \geq 1.00$.

The results indicate a tendency to obtain the highest reliability indexes for the lowest load ratios r, except for the case $r = 0.00$. This result can be justified once $r = 0.00$ presents only dead load in the load combination, while all other design situations within the range of $0.25 \leq r \leq 2.00$ present live load. Therefore, it is a particular situation within the proposed calibration range. Consequently, considering that the optimization process is performed jointly for all load ratios r, the results for $r = 0.00$ are penalized.

Concerning the partial safety factors obtained in this work, the results are consistent with those obtained by Santiago [7] and Costa et al. [25], which developed a calibration study for concrete and steel structures designed according to the main Brazilian codes. Differences are justifiable given that the study developed by the authors above considered different failure modes for steel and concrete elements but not for steel-concrete composite structures.

5 CONCLUSIONS

This work proposed the study of the flexural reliability of steel-concrete composite beams. The reliability analysis for beams designed according to the requirements of NBR 8800:2008 was carried out for compact and full-interaction sections, subjected to different load combinations and assuming a slab with different heights and different grades of concrete. The results indicated that the reliability indexes present significant variations as a function of load combinations, reducing the reliability level as the live load is predominant in the load combination. The maximum and minimum reliability indexes obtained were 3.79 and 2.72.

A calibration procedure of the partial safety factors based on PSO was proposed to reduce de variability of the reliability indexes in different design situations. For this purpose, a target reliability index $\beta_T = 3.5$ was considered. It was observed that the calibration procedure significantly reduces the variability of reliability indexes within the calibration range, so the maximum and minimum values of β obtained after calibration were 3.85 and 3.15. The average value of β, considering all the analyzed design situations, resulted in 3.43.

It should be noted that this work included one structural element and a few design situations. Consequently, the results are limited to configurations similar to those proposed in this work. The complete calibration of a design code must consider exhaustive design situations and different structural elements, such as the study presented by Santiago [7] and Costa et al. [25] for concrete and steel structures. Concerning the reliability of steel-concrete composite structures designed according to Brazilian codes, however, few technical publications are available, indicating a gap to be explored.

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