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RELIABILITY ANALYSIS OF COMPOSITE STEEL-CONCRETE SLABS DESIGNED IN ACCORDANCE WITH ABNT NBR 8800:2008

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Abstract. This paper presents a study on the reliability of composite steel-concrete slabs designed according to ABNT NBR 8800:2008 (Design of steel and composite structures for buildings) for longitudinal shear failure mode. For the calculation of the shear resistance, according to *m-k* method, results of bending tests were used. The probabilistic models of the random variables considered in the analysis were found in the literature, except for the model error variable for longitudinal shear resistance, which was determined from a base of experimental studies. The reliability indexes were obtained through the Monte Carlo Simulation (MCS) and the First Order Reliability Method (FORM), implemented in MATLAB®, for several composite slabs configurations and combinations of actions. In addition, the influence of geometric parameter variation on reliability was evaluated. Regarding the partial safety factor for longitudinal shear, the ABNT NBR 8800:2008 recommends that it to be equal to that determined by the specification used in the bending tests, so a comparison was made between the safety levels recommended by EUROCODE 4 Part 1-1:2004 (Design of composite steel and concrete structures) and CSSBI S2:2008 (Criteria for the testing of composite slabs).

Keywords: Reliability, Composite slabs, Bending tests, Model error, Safety levels.

1 Introduction

In Brazil, the composite steel-concrete slabs are designed according the requirements of ABNT NBR 8800:2008 (Design of steel and composite structures for buildings) [1] and ABNT NBR 8681:2003 (Actions and safety of structures - Procedure) [2]. Both codes are largely based on the American Steel Construction Institute (AISC 360-05:2005) [3] and EUROCODE:2001 [4], codes that migrated from Allowable Strength Design (ASD) to the Load and Resistance Factor Design (LRFD) through the principles of structural reliability. In contrast, in the Brazilian codes, there was no calibration process of the partial safety factors based on reliability, so many of them were adapted or equal to those of international codes. For this reason, the study of structural reliability has developed significantly during the last years in Brazil, being the target of several publications.

Some studies, such as Santos, Stucchi and Beck [5], Pereira, Beck and El Debs [6] and Moreira [7], analyzed the reliability of composite columns and beams, demonstrating the need for a calibration of ABNT NBT 8800:2008 [1] to be adequate to the Brazilian reality, either by obtaining reliability indexes below the recommended values or because they are over-conservative.

In addition, some articles, such as those by Mohammed, Karim and Hammood [8], Degtyarev [9] and Degtyarev [10] show that the partial safety factors used according to some international codes are not ideal for optimized design of composite steel-concrete slabs. In addition, despite being a structural element widely used in Brazilian civil construction, no national studies were found on the safety level of the calculation models used for its design.

This article presents an evaluation of the reliability of composite slabs for the longitudinal shear limit state, designed according to the Brazilian standard ABNT NBR 8800:2008 [1] and ABNT NBR 8681:2003 [2], using cross-sections and commercial steel decking. It also presents the analysis of the influence of certain variables and partial safety factors on the reliability indexes. For this, Monte Carlo Simulation and the FORM method, both in MATLAB® language, were used.

This article begins with Section 1 which introduces the subject. Section 2 presents the basic principles of structural reliability. Section 3 presents some concepts and formulations of composite steel-concrete slabs. Section 4 presents the cross-sections that were analyzed and the limit state function for longitudinal shear. In section 5, the probabilistic model is presented. The results obtained through Monte Carlo Simulation and FORM are presented in Section 6. Section 7 presents some conclusions obtained on this study.

2 Structural Reliability

The reliability analysis is based on the concepts of probability and statistics. It is related to the quantification of uncertainties and to the determination of the probability of a structural element reaching a specific limit state during its useful life. According to Melchers and Beck [11] in probabilistic assumptions, any uncertainty about a variable is explicitly taken into account.

2.1 Limit State Function

The limit state function, $g(X)$, provide a boundary between failure and safety, called the failure surface and characterized when $g(X) = 0$. For each boundary state of the structure there is a function $g(.)$, where $X_1, X_2, ..., X_n$ are the variables that are important for each limit state, such as those that characterize the actions, material properties and geometric parameters, so $g(X)$ is defined as:

$$g(X) = g(X_1, X_2, ..., X_n). \tag{1}$$

Involving only two vectors, the basic reliability problem is that of the resistance (R) and load effect (S) defined by:

$$g(R,S) = R(X) - S(X). \tag{2}$$

The failure domain (D_f) and the safety domain (D_s) is defined as:

$$D_f = \{x \mid g(x) \le 0\}; \tag{3}$$

$$D_s = \{x \mid g(x) > 0\}. \tag{4}$$

2.2 Probability of failure

The probability of failure (P_f) is related to the violation of boundary states. The probability of the limit state function assuming values less than or equal to zero is:

$$P_f = P[g(X) \le 0]. \tag{5}$$

Therefore, the probability of failure is:

$$P_f = \int_{Df} f_x(x)dx \tag{6}$$

where f_x is the probability density function (PDF) of the random variables of vector X and D_f is the failure domain, that is, when $S \le R$.

2.3 Reliability Index

The reliability index (β) is the geometric distance from the origin (mean) of the standard normal distribution (ϕ) the failure region. It is given as a function of the probability of failure by:

$$\beta = \phi(1 - P_f). \tag{7}$$

The level of safety that a structure must achieve is expressed in terms of target values of reliability index ($\beta_{t\arg et}$) that aim for a minimum level of reliability or a maximum probability of acceptable failure. In addition, reliability indexes should not be too high for economic issues.

The EUROCODE:2001 [4] recommends a value of 4.7 for a reference period of one year and of 3.8 for fifty years of reference, considering the Ultimate Limit State (ULS); and 2.9 for one year and 1.5 for fifty years considering an irreversible Serviceability Limit State (SLS).

The target reliability index adopted for most structural components is 3.8. However, according to Santos, Stucchi and Beck [5], as the slab calculation models do not take into account the high capacity of redistribution of loads through plastification of the cross-section, a lower target reliability index must be adopted, therefore in this article a value of 2.5 was adopted.

2.4 Reliability Analysis Methods

In this paper a MATLAB® program was created to find reliability indexes by Monte Carlo Simulation for composite steel-concrete slabs using 10,000,000 simulations. For the comparison and validation of the results, the FORM method executed also in MATLAB® was adopted.

Monte Carlo Simulation is used as a method for evaluating a deterministic model with significant uncertainties through the use of random numbers as inputs. To perform the simulation, it is necessary to randomly generate the entries from probability distributions by simulating the sampling process of a real population, in which the corresponding distributions are defined according to the characteristics of each variable was given by O'Connor and Kleyner [12].

According to Haldar and Mahadevan [13], the Monte Carlo Simulation is generally given in six steps:

Step 1: Definition of the problem using the random variables.

Step 2: Quantification of the probabilistic character of all random variables in terms of their distribution functions.

Step 3: Generation of the values of these random variables.

Step 4: Deterministic evaluation of the problem for each set of generated values of all random variables.

Step 5: Obtaining probabilistic information from the total number of simulations.

Step 6: Determination of simulation efficiency and accuracy.

The generation of random numbers begins with a uniform distribution with a range between zero and one. From the inverse transformation, samples of the random variables are obtained according to their respective probability distributions. Numerous repetitions are made in the processes resulting in various solutions to the problem. Therefore, a set of solutions or mechanical responses of the structure are obtained and they are compared with the limit function to obtain the probability of failure. Lastly, the reliability index is obtained from standard normal distribution function.

2.5 Combination of loads

The loads are combined according to ABNT NBR 8681:2003 [2], for normal, special or exceptional combinations. In this paper, only the dead load grouped and live load regarding the use of the building were considered. These loads were combined based on item 5.1.3.1 of ABNT NBR 8681:2003 [2], considering an ultimate normal combination, that is, due to the intended use for the construction. The load combination equation for this paper is:

$$F_d = \gamma_D \cdot D_k + \gamma_L \cdot L_k \tag{8}$$

where:
F_d is the design load;
D_k is the character value of dead load;
L_k is the characteristic value of live load;
γ_D is the partial factor of the dead load;
γ_L is the partial factor of the live load.

2.6 Procedure for reliability analysis

In this article it was used the same procedure as Santos, Stucchi and Beck [5]:

1. Definition of the cross-section to be analyzed.

2. Determination of the design resistance ($V_{l,Rd}$) according to ABNT NBR 8800:2008 and assume the same value for the design load effect S_d.

3. Definition of the load ratio χ:

$$\chi = \frac{L_k}{L_k + D_k}. \tag{9}$$

5. Transformation the design load effect into values characteristic of each load by:

$$D_k = \frac{S_d}{\gamma_D + \gamma_L \cdot \chi \,/\, (1-\chi)}. \tag{10}$$

$$L_k = \frac{S_d}{\gamma_L + \gamma_D \cdot (1-\chi) \,/\, \chi}. \tag{11}$$

6. Determination the parameters of probabilistic models.

7. Obtaining the reliability indexes for load ratios χ between zero and one.

Lastly, from the FORM method, the probabilistic importance factors (ς_i) of the random variables are obtained to analyze the contribution of each variable in the reliability.

3 Composite steel-concrete slabs

3.1 Introduction

According to ABNT NBR 8800:2008 [1] the composite steel-concrete slabs are formed by a corrugated steel decking and concrete acting together. During the construction stage, that is, before the concrete reaches 75% of its ultimate compression strength, only the steel decking resists the actions of fresh concrete weight and construction loads. In the final stage, however, all loads supported by the composite slabs are considered.

For the ultimate limit state, the composite slabs rupture modes are: bending moment, longitudinal shear, vertical shear and puncture. For the determination of longitudinal shear strength by the *m-k* method, bending tests are required.

For a better view of the composite slabs the Figure 1 is presented.

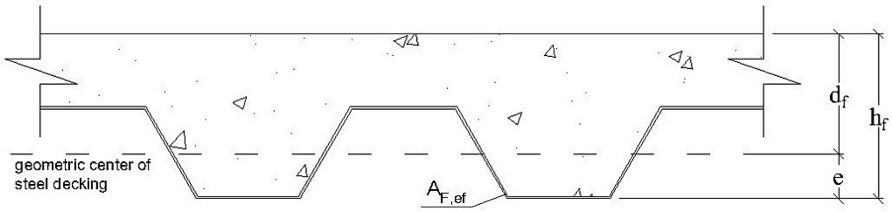

Figure 1. Cross-section of a composite slab

3.2 Longitudinal shear and m-k method

The most frequent bending test to analyze the behavior of composite steel-concrete slabs, according to EUROCODE 4 Part 1-1:2004 [14], is to apply two concentrated forces (F) positioned at $L / 4$ of the supports until the system capacity is exhausted. The m-k method consists of determining the angular (m) and linear (k) coefficients of a line and is based on results given by Schuster and Ling [18]:

$$\frac{V_u}{b \cdot d_f} = \frac{m}{L_s} + k \tag{12}$$

where:

V_u is the ultimate shear force associated with longitudinal shear (N);
b is the width of the slab cross-section (mm);
d_f is the distance from the upper face of the concrete to the geometric center of the effective section of the steel decking (mm);
m and k are design values for the empirical factors representing, respectively, the mechanical interlock and the friction between concrete and steel (N / mm^2);
L_s is the shear span (mm).

For a better view of the bending test the Figure 2 is presented.

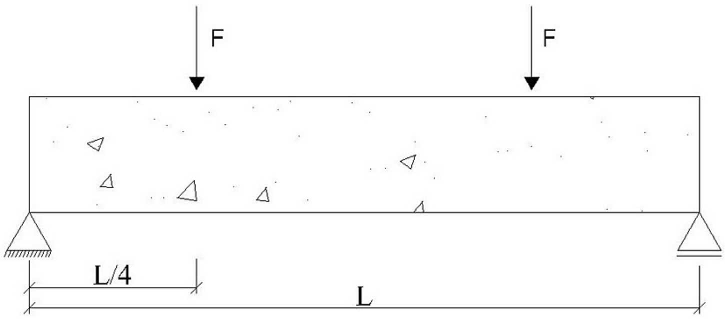

Figure 2. Representation of a bending test

The ABNT NBR 8800:2008 [1] presents the Eq. 12 with adjustments of units and nomenclatures, for the determination of the design longitudinal shear resistance:

$$V_{l,Rd} = b \cdot d_f \cdot \left[\left(\frac{m \cdot A_{F,ef}}{b \cdot L_s} \right) + k \right] / \gamma_{sl} \tag{13}$$

where:
$V_{l,Rd}$ is the design longitudinal shear resistance (N);
$A_{F,ef}$ is the nominal area of the cross-section of the steel decking (mm^2);
γ_{sl} partial safety factor for the ultimate limit state.

According to ABNT NBR 8800:2008 [1], if the bending tests for obtaining the m and k factors are performed according to CSSBI S2-2008:2008 [15] or ANSI / ASCE 3-91:1992 [16] further adaptations are necessary.

The partial safety factor (γ_{sl}) may vary according to the standard employed in the tests. If the constants m and k are derived from tests based on EUROCODE 4 Part 1-1:2004 [14], it is considered $\gamma_{sl} = 1.25$. If CSSBI S2-2008:2008 [15] and ANSI / ASCE 3-91:1992 [16] are used, an adaptation is adopted, with $\gamma_{sl} = 1.43$ and $\gamma_{sl} = 1.33$ respectively.

According to Grossi [17], there are differences in bending tests procedures of each code that change the values of m and k. So, for all codes dealing with equivalent safety levels, the partial safety factors are different.

4 Limit state function for longitudinal shear and cross-sections analyzed

The reliability analysis was conducted from sixteen models of composite steel-concrete slabs, with variations in the total slab height, theoretical span, thickness and type of steel decking. The design values of longitudinal shear resistance were calculated from Eq.13. In all analysis it was assumed:

- simply supported slab;
- uniformly distributed load;
- unshored construction;
- $b = 1000\ mm$;
- slab without end anchorage.

For the cross-sections analyzed, the performance functions are:

$$g(X) = \theta_R \cdot \left\{ b \cdot (h_t - e) \cdot \left[\left(\frac{4 \cdot m \cdot A_{F,ef}}{b \cdot L} \right) + k \right] / 1000 \right\} - \theta_S \cdot (V_D + V_L) \tag{14}$$

where:
θ_R is the model error for longitudinal shear resistance;
θ_S is the model error for load effect;
h_t is the total slab height;
e is distance from the bottom face of the slab to the geometric center of the effective section of the steel decking;
L is the theoretical span;
V_D is the vertical shear caused by dead load;
V_L is the vertical shear caused by live load;

The cross-sections analyzed are described in Table 1:

Table 1. Cross-sections analyzed

Nomenclature	Steel decking	t_f (mm)	$A_{F,ef}$ (mm²)	h_t (mm)	e (mm)	d_f (mm)	L (mm)	Ls (mm)	$V_{l,Rd}$ (kN)
S01	MD-55	1.25	1452	140	27	113	3400	850	18.896
S02	MD-55	1.25	1452	140	27	113	3200	800	20.450
S03	MD-55	1.25	1452	140	27	113	3000	750	22.211
S04	MD-55	1.25	1452	140	27	113	2500	625	27.846
S05	MD-55	1.25	1452	140	27	113	2000	500	36.300
S06	MD-55	1.25	1452	105	27	78	2300	575	21.251
S07	MD-55	1.25	1452	120	27	93	2300	575	25.338
S08	MD-55	1.25	1452	140	27	113	2300	575	30.787
S09	MD-55	1.25	1452	180	27	153	2300	575	41.685
S10	MD-55	1.25	1452	200	27	173	2300	575	47.134
S11	MD-55	0.80	912	140	27	113	2500	625	20.910
S12	MD-65	0.80	912	140	32.50	107.5	2500	625	18.519
S13	CE-75	0.80	1112	140	37.49	102.51	3200	800	13.768
S14	CE-75	1.25	1771	140	37.72	102.28	3200	800	21.477
S15	MF-50	0.80	997	140	26.13	113.87	2500	625	15.149
S16	MF-50	1.25	1587	140	26.36	113.64	2500	625	24.063

The MD-55 and MD-65 steel decking were tested by Sieg [19] and Grossi [17] in accordance with EUROCODE 4 Part 1-1:2004 [14], therefore partial safety factor for longitudinal shear (γ_{sl}) adopted was 1.25 for these models. In slabs S13 and S14 was used the CE-75 steel decking, designed by Ferraz [20] according to CSSBI S2-1988:1988 [21]. In S15 and S16 the MF-50 steel decking, tested by Brendolan [22] and designed according to CSSBI S2-2002:2002 [23], was used. So, in S13 to S16 slabs, the adapted partial safety factor used was $\gamma_{sl} = 1.43$.

Table 2 presents the factors m and k used to calculate the longitudinal shear resistance of the slabs in this study. In the steel decking tested by Ferraz [20] and Brendolan [22], adaptations are necessary so that the factor m has the dimension force per unit area, in Newton per square millimeter, that is, the value of m was multiplied by the width b and divided by $A_{F,ef}$.

Table 2. Geometric parameters and empirical factors of steel decking

Steel decking	t_f (mm)	$A_{F, ef}$ (mm²)	m (N/mm²)	k (N/mm²)	Reference
MD-55	1.25	1452	161	-0.0660	Sieg [19]
MD-55	0.80	912	164	-0.0080	Sieg [19]
MD-65	0.80	912	151	-0.0050	Grossi [17]
CE-75	0.80	1112	136.82	0.0017	Adapted from Ferraz [20]
CE-75	1.25	1771	117.80	0.0392	Adapted from Ferraz [20]
MF-50	0.80	997	124.56	-0.0087	Adapted from Brendolan [22]
MF-50	1.25	1587	139.36	-0.0514	Adapted from Brendolan [22]

5 Probabilistic models

The distributions and statistical parameters of each variable were taken from several publications and studies about reliability and composite slabs. To calculate the statistical parameters of the random variable model error for longitudinal shear resistance, an experimental database of seventy slabs was compiled with the results of the following studies: Araújo [24], Brendolan [22], Britto Junior [25], Friedrich [26], Grossi [17], Sieg [19], Ferraz [20] and Silva [27].

The probabilistic models are summarized in Table 3.

Table 3. Probabilistic models and and statistical parameters

Variable	Symbol	Distribution	Mean (μ)	Coefficient of variation	Reference
Dead load	D_k	Normal	1.05 D_k	0.10	[28], [29] and [30]
Live load (50 years)	L_k	Gumbel	1.00 L_k	0.25	[30]
Nominal area of the cross-section of the steel decking	$A_{F,ef}$	Lognormal	1.00 $A_{F,ef}$	0.05	[31]
Total slab height	h_t	Lognormal	0.99 h_t	0.029	[32]
Theoretical span	L	Lognormal	1.00 L	0.05	[31]
Model error for load effect	θ_S	Lognormal	1.00	0.05	[5]
Model error for longitudinal shear resistance	θ_R	Lognormal	1.03	0.06	Authors
Width	b	Deterministic (1000 mm)	-	-	-

Variable	Symbol	Distribution	Mean (μ)	Coefficient of variation	Reference
Distance from the bottom face of the slab to the GC of the effective section of the steel decking	e	Deterministic	-	-	-
"m" factor	m	Deterministic	-	-	-
"k" factor	k	Deterministic	-	-	-

6 Results

The S01 slab was used to compare the influence of the two types of loads partial safety factors with normal combination. According to ABNT 8681:2003 [2] and ABNT NBR 8800:2008 [1], for type 1 building the coefficient (*i.e.* for a live load less than 5.0 kN/m²) for dead load is $\gamma_D = 1.35$ and for live load is $\gamma_L = 1.50$; and for type 2 buildings (*i.e.* for a live load greater than 5.0 kN/m²), the coefficients are $\gamma_D = \gamma_L = 1.40$. Figure 3 presents the results of the reliability indexes for both types of buildings using the S01 cross-section.

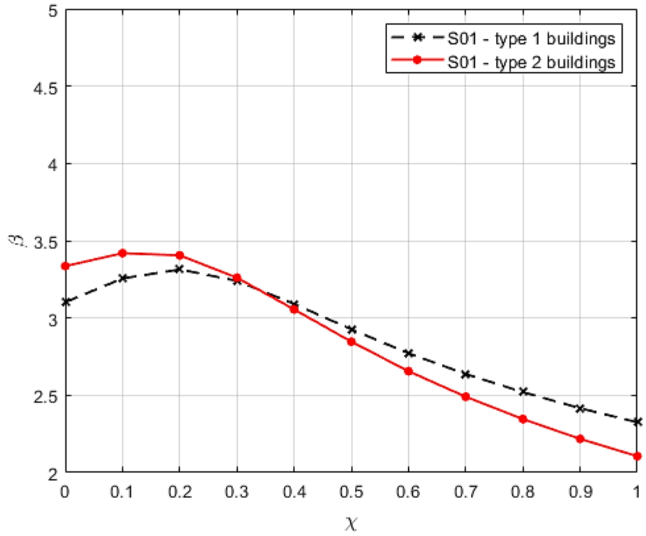

Figure 3. Comparison of reliability indexes obtained considering the loads partial safety factors for type 1 and 2 buildings via SMC

In general, the reliability index decreases with the increase of the live load variable, since the coefficient of variation of L_k is higher than that of D_k. According to Fig.3, for load ratios between 0.0 and 0.3, composite slabs designed with load partial safety factors considering type 2 buildings have higher reliability than type 1. However, this situation reverses to load ratios $\chi \geq 0.35$.

For the most critical usual case, that is, when $\chi = 0.7$, it was obtained $\beta_{MCS} = 2.637$ and $\beta_{FORM} = 2.654$ for type 1 building and $\beta_{MCS} = 2.491$ and $\beta_{FORM} = 2.509$ for type 2 building. These values are close to the target reliability index of 2.50 adopted in this paper, however, by MCS, the value for the type 2 building was slightly lower.

The variation of the reliability indexes with the change of the theoretical span (L), total slab height (h_t), thickness of steel decking (t_f) and steel decking type was analyzed with the partial safety factors for loads considering type 1 buildings. To evaluate the influence of theoretical span, the slabs

S01 to S05 were simulated considering type 1 building. Figure 4 presents the results obtained in this analysis.

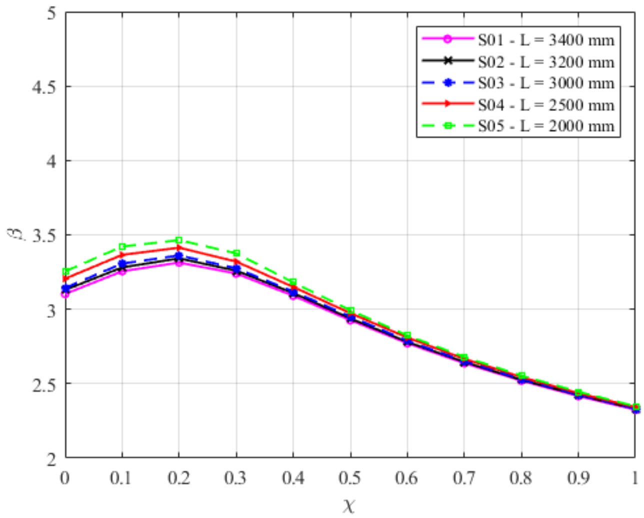

Figure 4. Reliability indexes of S01 to S05 slabs via SMC

From Fig. 4, it is observed that the reliability index increases with the decrease of the theoretical span value, mainly for lower load ratios. However, the variation of this parameter did not significantly change the reliability of the composite steel-concrete slabs. Moreover, for the most critical usual case (*i.e.* the lowest reliability within the practical range of $\chi = 0.1$ to 0.7), the slabs S01 and S05 obtained reliability indexes higher than the target value ($\beta_{target} = 2.5$).

To evaluate the influence of the total slab height (h_t), the slabs S06 to S10 were simulated considering type 1 building. Figure 5 presents the results obtained in these analyzes.

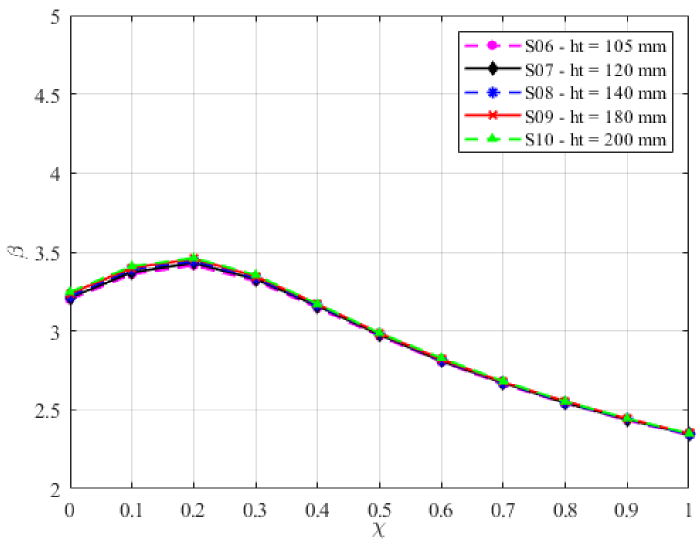

Figure 5. Reliability indexes of S01 to S05 slabs via SMC

From Fig.5, it is observed that the parameter total height of the slab (h_t) has no significant influence on the reliability indices for any load ratio. For the most critical case, the slabs S06 and S10 obtained reliability indexes higher than the target value.

Slabs S04 and S11 were used to verify the influence on the reliability of changing the thickness of the MD-55 steel decking. Figure 6 presents the reliability indexes considering type 1 building obtained in the simulations.

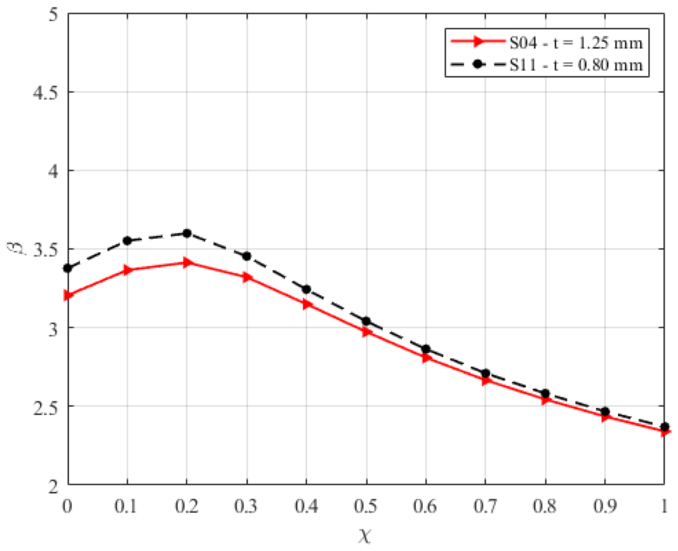

Figure 6. Reliability indexes of S04 and S11 slabs via SMC

Figure 6 shows that the reliability of the S11 slab is higher than that of the S04, mainly for low load ratios, although the longitudinal shear resistance of the slabs with 1.25 mm decks be greater than of the slabs with 0.80 mm deck. The fact that the deterministic variables m and k of the MD-55 steel decking with thickness 0.80 mm, presented by Sieg [19], are higher than of the steel decking of 1.25 mm it might have caused these results.

Furthermore, the m and k factors represent, respectively, the mechanical interlocking and the friction between concrete and steel. They are dependent on several factors, such as the shape of the embossments, the thickness and the height of the steel decking. Besides that, as the thickness variation of the steel decking alters the area of the steel decking cross-section and m and k factors, it is not possible to establish a main reason for these results.

Slabs S11 and S12 were used to verify the influence of height steel decking variation on reliability. Figure 7 presents the reliability indexes considering type 1 building obtained in the simulation.

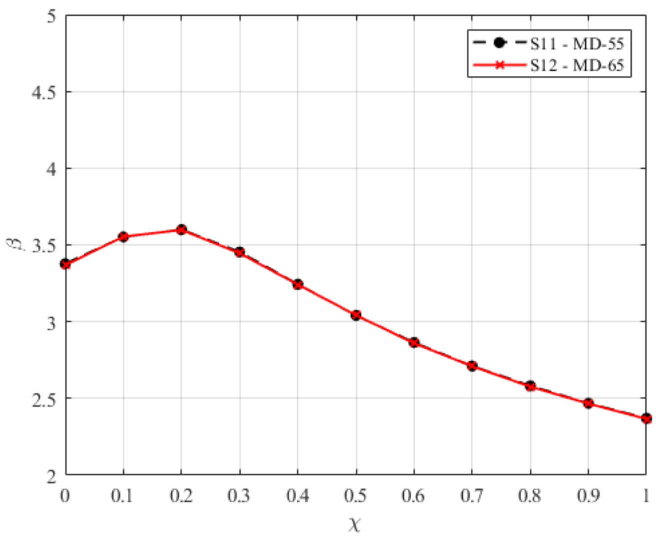

Figure 7. Reliability indexes of S11 and S12 slabs via SMC

Figure 7 proves that changes of the height steel decking did not significantly alter the reliability indexes for any load ratio value.

Slabs S13 to S16 were used to verify the reliability of composite concrete-steel slabs designed with the longitudinal shear partial safety factor proposed by CSSBI S2-2008:2008 [15], equivalent to $\gamma_{sl} = 1.43$. Figure 8 presents the reliability indexes considering type 1 building obtained in the simulations for the S13 and S14 slabs with CE-75 steel decking.

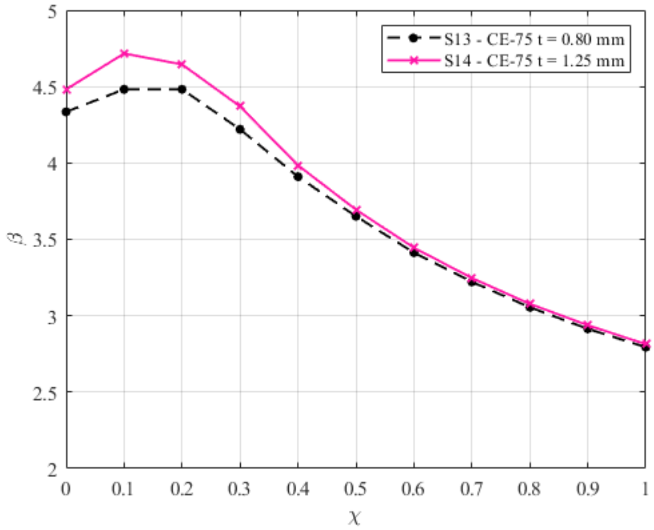

Figure 8. Reliability indexes of S13 and S14 slabs via SMC

Unlike S04 and S11, in composite slabs with CE-75 steel decking the reliability of the slabs with thickness 1.25 mm steel decking is higher than with 0.80 mm. The fact that the deterministic variable m of the CE-75 steel decking of 0.80 mm, presented by Ferraz [20], is lower than the 1.25 mm it might have caused these results.

Figure 9 presents the reliability indices obtained considering type 1 building in the simulations for S15 and S16 slabs with MF-50 steel decking.

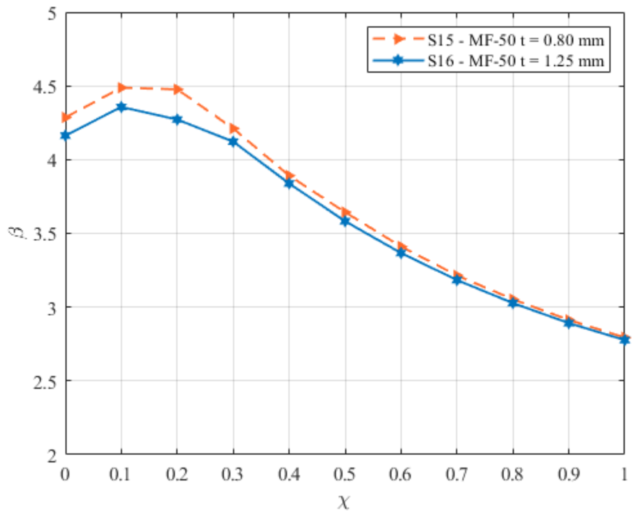

Figure 9. Reliability indexes of S15 and S16 slabs via SMC

As S04 and S11, in the slabs with MF-50 steel decking the reliability decreased when a thicker steel decking was used, even though the longitudinal shear strength of slabs with 1.25 mm steel decking was higher than slabs with 0.80 mm steel decking. The fact that the deterministic variable m of the 0.80 mm MF-50 deck presented by Brendolan [22] is higher than the 1.25 mm deck it might have caused these results.

From the reliability indexes obtained in this paper, it is observed that the reliability of composite concrete-steel slabs design in designed according CSSBI S2-2008:2008 [15] (slabs S13 to S16) is considerably higher than those designed in accordance with EUROCODE 4 Part 1-1:2004 [14] (slabs S01 to S12). Figure 10 illustrates this difference from the reliability indexes found for slabs S11 and S15.

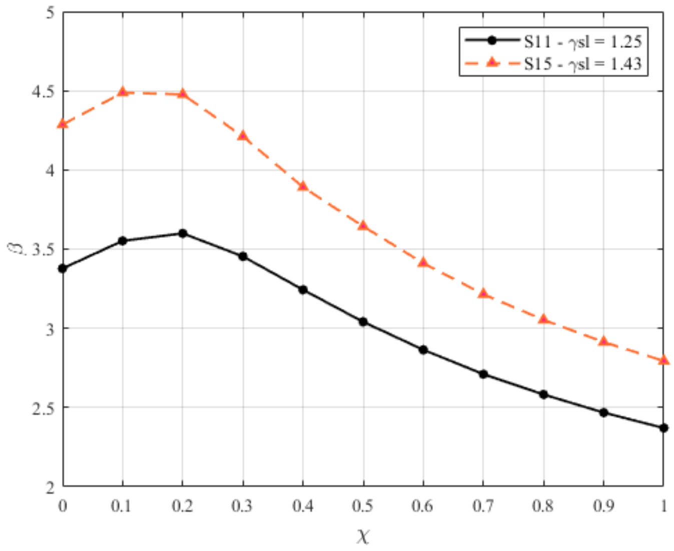

Figure 10. Reliability indexes of S11 and S15 slabs via SMC

To verify the difference of the results between the methods MCS and FORM, the reliability indexes of S01 slab from Monte Carlo Simulation were compared with those obtained by the FORM method, described in the open source FERUM (Finite Element Reliability Using Matlab) [33]. Figure 11 presents the comparison of the results obtained. The maximum difference between the values found was 0.0687 for $\chi = 0.5$.

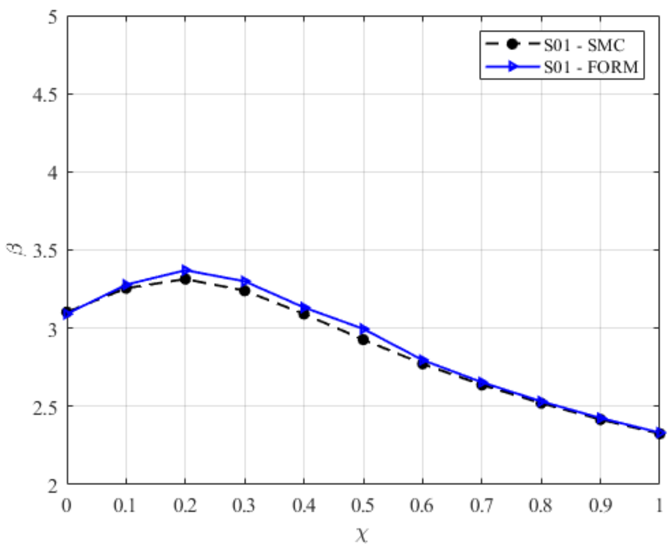

Figure 11. Comparison of S01 slab results via SMC and FORM

Importance factors (ς_i) of slab S01 were obtained via FORM for the load ratios $\chi = 0.3$ and $\chi = 0.7$. Figure 12 and Figure 13 present the results obtained.

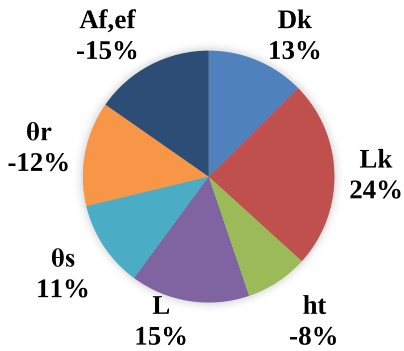

Figure 12. Contribution of random variables in reliability index to $\chi = 0.3$ (S01)

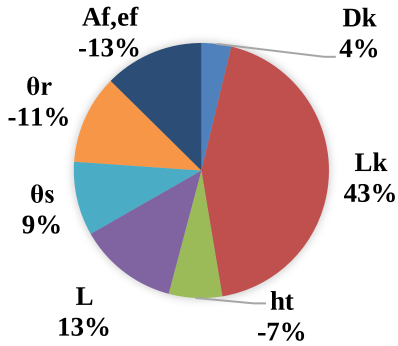

Figure 13. Contribution of random variables in reliability index to $\chi = 0.7$ (S01)

From the results presented in Figure 12 and Figure 13, it is observed that the importance factor for composite slabs is negative when the increase of the value attributed to its associated variable results in higher reliability indexes. However, lower values related to variables with positive importance factors result in increased reliability.

It is observed that the most relevant resistance random variables are the area of the nominal area of the cross-section ($A_{F,ef}$) and the theoretical span (L), followed by the model error for longitudinal shear resistance (θ_R), while the total slab's height (h_t) was the least influential. It is also possible to observe that the relevance of these variables in the system decreases with the increase of the load ratio.

Regarding the random variables of load effect, the results show that the dead load variable is more important when χ is smaller and, on the other hand, the live load variable becomes more influential with the increase of the load ratio. Dead load is the most relevant variable, given its high coefficient of variation, thus being the most decisive variable in determining the probability of failure especially for high load ratios.

7 Conclusion

This paper presents the procedures adopted for the determination of reliability indexes of composite steel-concrete slabs for the longitudinal shear limit state following the requirements of the Brazilian standard ABNT NBR 8800:2008 [1]. This study allowed analyzing through reliability the calculation model using sections and steel decking commonly used in Brazilian buildings. It was also possible to identify the variation in the reliability of composite slabs caused by the change of partial safety factors, geometric parameters and load ratios. Moreover, as the Brazilian code does not present the longitudinal shear partial safety factor, it was possible to compare the reliability indexes using coefficients of two international standards, which were indicated by the Brazilian code.

During a literature review for the determination of probabilistic models and geometric parameters of random variables, no publications were found about the model error for longitudinal shear resistance for composite steel-concrete slabs. Therefore, from the compilation of an experimental database of seventy models, it was possible to characterize this variable, so that when it was inserted in the analyzes, the model uncertainty was considered in the safety level evaluation. As in other publications (as in [5]), the lognormal distribution was adopted. The statistical parameters obtained were $\mu = 1.03$ and $\sigma = 0.06$, similar to those adopted for other structural elements, as in Santos, Stucchi and Beck [5].

To determine the reliability indexes, a MATLAB® code was developed to perform the Monte Carlo Simulation. In addition, for a more complete reliability analysis, the results of the reliability indices by MCS were compared with those obtained by FORM, described in the open source FERUM [33] also in MATLAB®. In both comparisons satisfactory differences were obtained between the two methods.

Regarding the reliability analysis, several analyzes were performed by changing the geometric configurations, the load ratios and the possible load partial safety factors. Seven random variables were considered in this work: the dead load (D_k), the live load (L_k), the total height of the slab (h_t), the theoretical span of the slab in the direction of the ribs (L), the nominal area of the cross-section steel decking ($A_{F,ef}$) and the model error for longitudinal shear resistance (θ_R) and load effect (θ_S).

Reliability indexes generally decreased with increasing load ratio (χ), because the coefficient of variation of the live load variable was higher than that of the dead load variable.

Considering the practical range of load ratios, for all composite slab cross-sections, with the combination of actions for type 1 buildings, reliability indexes were found very close to the established safety level. Only in the MCS of slab S01 for type 2 buildings, considering the most critical usual case of loading $\chi = 0.7$, was obtained $\beta_{MCS} = 2.491$, slightly lower than the target adopted ($\beta_{target} = 2.50$). Therefore, for normal combination in composite steel and concrete slabs, it is recommended to use the dead load safety factor of $\gamma_D = 1.35$ and for live load of $\gamma_L = 1.50$, proposed by ABNT NBR 8681:2003 [2] and ABNT NBR 8800:2008 [1].

The study has also shown that increasing the nominal value of the theoretical span in the direction of the ribs has resulted in lower reliability indexes, especially for lower load ratios. However, the variation of this parameter did not significantly change the reliability.

Regarding the increase in the total slab height, the analysis resulted in an increase of reliability indexes, mainly for low load ratios. However, as the theoretical span, the variation of the total slab height did not significantly change the reliability.

The thickness of the steel decking was the parameter that most significantly influenced the reliability of the composite steel-concrete slabs, since it changed the value of the nominal area of the cross-section steel decking and the m and k factors. For MD-55 and MF-50 steel decking, reliability decreased with increasing thickness. In contrast, using the CE-75 steel decking, opposite results were obtained. Regarding the change in the height of the steel decking, no relevant variations were found in the reliability indexes.

Moreover, the results related to the influence analysis of the variations of the geometric parameters and the loading ratios were confirmed by obtaining the probabilistic importance factors of each variable.

As ABNT NBR 8800:2008 [1] recommends the use of partial safety factor for longitudinal shear from international codes, a composite slabs analysis was performed adopting $\gamma_{sl} = 1.43$ (employed by CSSBI S2-2008:2008 [15]) and $\gamma_{sl} = 1.25$ (value adopted by EUROCODE 4 Part 1-1:2004 [14]). Although simulations with the Canadian code partial safety factors result in considerably higher reliability indexes, the European code is recommended for bending tests to avoid over-conservative constructions.

Analyzing the results obtained in this paper, it is suggested that ABNT NBR 8800:2008 [1] present the procedures related to the bending test and recommend a value for the partial safety factor for longitudinal shear. In addition, the lack of uniformity of reliability indexes for different load ratios demonstrates the importance of calibrating the partial safety factors of the Brazilian code.

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