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# Safety evaluation of circular concrete-filled steel columns designed according to Brazilian building code NBR 8800:2008

# *Avaliação da segurança de pilares mistos preenchidos de seção circular projetados segundo a norma NBR 8800:2008*



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## Abstract

This paper presents an investigation of the safety of circular concrete-filled steel columns designed according to Brazilian building code NBR 8800:2008. The investigation is based on a comparison of code provisions to column strength obtained in 32 experimental tests, and on a comparison to provisions of the equivalent American and European codes. The modeling error of theoretical code resistance models is evaluated. An analysis of covariance is performed in order to identify tendencies of these models. The study reveals that the resistance model used in the Brazilian code is compatible with foreign codes, in terms of bias and variances. The study reveals an additional safety margin of the order of 10% for the NBR8800 code, when partial safety factors are removed. Reliability analysis is performed for 3888 column configurations, resulting in reliability indexes that cover the building codes application spectrum. The study shows that the Brazilian code presents reliability indexes which are compatible with foreign codes.

***Keywords:*** Concrete-filled columns, experimental analysis, safety, reliability.

## Resumo

Este artigo apresenta uma investigação da segurança de pilares mistos preenchidos de seção circular, dimensionados segundo a norma brasileira NBR 8800:2008. Esta investigação é feita com base na comparação de previsões normativas com a resistência obtida em 32 ensaios experimentais, bem como na comparação com as normas americana e européia correspondentes. O erro de modelo das equações teóricas de resistência destas normas é determinado. Uma análise de covariância é realizada para verificar tendências das equações de resistência. O estudo mostra que o modelo de resistência utilizado na norma brasileira é compatível com as normas estrangeiras, em termos de tendenciosidade e variância. O estudo revela uma margem de segurança adicional da ordem de 10% na NBR8800, quando removidos os coeficientes parciais de segurança. Uma análise de confiabilidade é realizada para 3888 configurações de pilar, resultando em índices de confiabilidade que refletem a segurança do universo de pilares cobertos pelas normas. O estudo mostra que a norma brasileira apresenta índices de confiabilidade compatíveis com as normas estrangeiras.

***Palavras-chave:*** NBR 8800, pilares mistos preenchidos, análise experimental, segurança, confiabilidade.

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## 1. Introduction

This paper presents an evaluation of the safety of circular concrete filled columns, designed according to Brazilian design code (NBR 8800:2008 [1, 2]). The investigation is based on 32 experimental results and on a comparison to the corresponding North-American and European design codes (ANSI/AISC 360:2005 [3] e EN 1994-1-1:4:2004 [4]). This research is part of an extensive experimental program about steel-concrete composite structures developed at the São Carlos School of Engineering, University of São Paulo.
In recent years, steel-concrete composite design has been experiencing increasing applications worldwide. In some countries of Asia and Oceania, in particular, the choice for concrete-filled steel is based on the excellent resistance of this composite to seismic loading. This property is credited to the confinement effect of concrete filling by steel tube, to the ductility provided by outer steel and to the great energy absorption capacity of the composite [5].
In addition to the sound mechanical properties, concrete-filled steel does not require formwork during construction and allows for a reduction of cross-sections when compared with reinforced concrete. Due to these advantages and increasing usage worldwide, steel concrete composites have been subject of intense research in recent years [6 to 14].

## 2. Experimental analysis

The present study is based on experimental results obtained for 32 circular concrete-filled steel tube columns of different configurations. The main parameter varied in these tests was the length-to-diameter ratio $L/D$. Slenderness ratios of $L/D$ = {3, 5, 7 and 10} where tested. The columns where composed of steel tubes of $D$ = 114.3 mm (external diameter) with wall thickness of $t$ = {3.35 and 6.0} mm.
The columns where filled with concretes of mean compressive strengths of $f_{cmean}$ = {32.68, 58.68, 88.78 and 105.45} MPa. These concretes were produced with commonly encountered materials and usual techniques of mixture and cure. Compressive strengths were obtained by testing cylindrical specimens (100 × 200) mm, 28 days after production. The Young modulus was evaluated by means of the normative prescriptions.
Yield strength of the tubes was also evaluated by testing, following ref. [15]. For the tube with wall thickness $t$ = 3.35 mm, a mean strength of 287 MPa was found, with a c.o.v. of 5.6%. For the $t$ = 6.0 mm wall thickness tube, mean strength was 343 MPa and the c.o.v. was found to be 3.36%.
Testing of the columns was performed at the Structures Laboratory, São Carlos School of Engineering, University of São Paulo. An Instron 8506 testing machine was used for this purpose. This electronically controlled equipment allows static tests of up to 2500 kN nominal load in structural elements of reasonable proportions.
The tested columns were instrumented with four linear variable differential transducers (LVDT), which were placed outside the columns with help of metal collars. Longitudinal displacements where measured as the mean of four transducers. Additionally, two strain gauges were placed outside each column, and two were placed in a steel bar inside the concrete, allowing one to measure strains in both steel and concrete components of the composite column.
Centered axial loading was applied to the concrete and steel section of the columns and tests were conducted until failure. The failure of short columns was characterized by shear fracture of the concrete core, which was still confined by the steel tube. Slender columns failed due to global plastic buckling with no signs of local buckling of the steel tube.
Experimental results of ultimate load obtained for the 32 specimens are presented in Tables 1 and 2, second column. The first column in these tables identifies the specimens. Code P1-80-5D-E, for example, identifies tube wall thickness (P1 for $t$ = 3.35 mm, P2 for $t$ = 6mm), 80 identifies resistance class of the concrete, 5D is the length (with respect to diameter), and E means that the axial load was applied simultaneously on both concrete and steel sections (all cases presented in this paper).

## 3. Design code resistance models

In this section, a brief description of the main design equations for steel-concrete composite columns is presented, according to the Brazilian, North-American and European design codes. Detailed description of the design procedures can be found in the corresponding code documents [1, 3, 4] or in ref. [16].
The design codes considered in this study provide different expressions for the evaluation of a columns compressive resistance. However, they are all based on a sum of the contributions of steel and concrete to the resisting capacity of the columns.
The design strength o columns subject to axial compressive loads, according to NBR 8800:2008, is given by equation (1). Since, in this investigation, no reinforcement bars where used inside the columns, the corresponding contribution terms were removed from design equations presented herein.

$$N_{Rd} = \chi_{NBR} \cdot N_{Rd,p\ell} \tag{1}$$

In eq. (1), $\chi_{NBR}$ is a resistance reduction factor to account for column instability, and $N_{Rd,p\ell}$ is the cross-section resistance, given by eq. (2).

$$N_{Rd,p\ell} = \frac{f_y \cdot A_a}{\gamma_a} + \alpha \cdot \frac{f_{ck} \cdot A_c}{\gamma_c} \tag{2}$$

In eq. (2), $A_c$ is the concrete cross-section area; $f_{ck}$ is the characteristic compressive strength of concrete; $\gamma_c$ is a partial safety factor for concrete resistance (equal to 1.40), $\gamma_a$ is a partial safety factor for steel (equal to 1.10) and α is a cross-section factor (α=0,85 for rectangular or square and α=0,95 for circular cross-sections, respectively). In the comparison to experimental results, all partial safety factors are taken as unitary, and all resistance properties are considered at their mean values.
In the European code (EC4), the design resistance for circular concrete-filled steel tube columns is given by:

$$N_{pl,Rd} = \eta_a \cdot \frac{f_y \cdot A_a}{\gamma_a} + \frac{f_{ck} \cdot A_c}{\gamma_c} \cdot \left[1 + \eta_c \cdot \left(\frac{t}{D}\right)\frac{f_y}{f_{ck}}\right] \tag{3}$$

where $\eta_c$ is a concrete-confinement factor and $\eta_a$ is a steel resistance reduction factor. The design resistance is given by:

$$N_{Ed} = \chi \cdot N_{pl,Rd} \quad (4)$$

The resistance reduction factor $\chi$ for axially loaded columns is given by four design equations, based on experimental results (curves a, b, c or d). The type of cross-section and the instability axis define the curve to be used in a given design.
In the North American (ANSI/AISC) design code, the resisting capacity of a column is given by:

$$P = \phi_c \cdot P_n \quad (5)$$

where $\phi_c$ is a partial safety factor applied to member resistance - LRFD – Load and Resistance Factor Design (equal to 0.75 for columns). Instability of slender columns is given by $P_n$, which is evaluated from equation (6):

$$P_n = \begin{cases} P_0 \cdot \left[0,658^{\left(\frac{P_0}{P_e}\right)}\right], & \text{se } P_e \geq 0,44 \cdot P_0 \\ 0,877 \cdot P_e, & \text{se } P_e < 0,44 \cdot P_0 \end{cases} \quad (6)$$

In equation (6), $P_e$ is the Euler load for elastic instability, and $P_0$ is the elastic resistance of the cross-section, given by:

$$P_0 = A_s \cdot f_y + C_2 \cdot A_c \cdot f'_c \quad (7)$$

In equation (7), $C_2$=0,95 for concrete-filled circular cross-sections.
For all columns tested in the experimental part of the investigation, the strength predicted by code resistance models was determined. Results are compared in Tables 1 and 2, columns 3, 4 and 5.
In the comparison to experimental results and in the reliability analysis to follow, equations (1) to (7) are used with unitary partial safety factors. In order to distinguish this situation from the design situation (where a safety margin is desired), the axial compressive strength for unitary safety margins will be referred to as $N_{Rs}$, with “s” standing for “safety analysis”.

## 4. Model error

In order to compare theoretical design code provisions with experimental ultimate load results, a model error ($M_e$) variable is introduced:

$$M_e = \frac{F_{exp}}{N_{Rs}} \quad (8)$$

Samples of this random variable were obtained from the 32 available experimental results, for each of the design code provisions, as shown in Tables 1 and 2. Histograms of these results are presented in Figure 1. A probability distribution fit to the cumulative histograms was performed, resulting in the distributions and parameters shown in Table 4 and in Fig. 1. The lognormal distribution is appropriate to describe resistance model error, since it preserves log-normality of material strength distributions [17]. Other probability distributions, such as translated Rayleigh, translated lognormal and triangular were actually found to provide better fits to the available data, but where discarded as the lower and/or upper limits where not justifiable.
The mean of a model error random variable ($M_{e\ mean}$) is known as bias factor. Ideally, this mean should be unitary, reflecting an unbiased resistance model. Moreover, a perfect resistance model would result in zero variance. Clearly, this is not the case of the data shown in Table 4. Results show that the ANSI and NBR design codes provide resistances that are smaller than the experimental ($M_{e\ mean}$>1), resulting in conservative bias. The European code has a tendency for over-predicting column resistance, resulting in unconservative model bias $M_{e\ mean}$<1).
Ideally, the bias should be removed from the resistance model. This can be done by an investigation of the model, which identifies bias sources. When this is not possible, a mean bias correction can be adopted. A corrected resistance model is obtained as:

$$N_{RS}^{corr.} = M_{e\,mean} \cdot N_{RS}$$
$$M_e^{corr.} = \frac{M_e}{M_{e\,mean}} \quad (9)$$

The corrected model error random variable, $M_e^{corr.}$, is also obtained by dividing $M_e$ by $M_{e\ mean}$. The mean Unitary (the corrected model presents no bias) and the standard deviation becomes:

$$M_{e\ s.dev.}^{corr.} = \frac{M_{e\ s.dev.}}{M_{e\ mean}} \quad (10)$$

The standard deviation is related to the aleatory error of the model, that is, to the models incapacity of correctly predicting column resistance in all design situations. This uncertainty is incorporated in the reliability analysis to follow.
In order to identify possible sources of bias of the design code resistance models, a covariance analysis was performed. In this analysis, the model error is studied in term of those parameters that where varied in the experiments: $f_{ck}$, slenderness (L/D) and tube thickness $t$. The analysis reveals tendencies of the theoretical model in terms of the variables considered. The correlation index, $\rho_{xy}$, is evaluated as:

Table 1 – Experimental results and code predicted column resistance, wall thickness t=3,35mm

| Column | $F_{exp}$ (kN) | NBR (kN) | EC4 (kN) | ANSI/AISC (kN) |
|---|---|---|---|---|
| P1-30-3D-E | 737 | 615,36 | 813,17 | 615,50 |
| P1-30-5D-E | 739,5 | 611,02 | 753,44 | 611,40 |
| P1-30-7D-E | 631,5 | 604,57 | 700,25 | 605,31 |
| P1-30-10D-E | 599,3 | 591,10 | 634,35 | 592,57 |
| P1-60-3D-E | 952 | 838,16 | 1035,67 | 838,51 |
| P1-60-5D-E | 902,9 | 830,64 | 971,05 | 831,61 |
| P1-60-7D-E | 868,5 | 819,48 | 908,10 | 821,36 |
| P1-60-10D-E | 809,2 | 796,28 | 838,25 | 800,00 |
| P1-80-3D-E | 1136,2 | 1095,57 | 1296,06 | 1096,26 |
| P1-80-5D-E | 1180,7 | 1083,47 | 1224,15 | 1085,37 |
| P1-80-7D-E | 1198,3 | 1065,57 | 1151,97 | 1069,24 |
| P1-80-10D-E | 1111,6 | 1028,51 | 1077,09 | 1035,74 |
| P1-100-3D-E | 1453,1 | 1237,91 | 1441,21 | 1238,83 |
| P1-100-5D-E | 1407,1 | 1222,89 | 1363,74 | 1225,42 |
| P1-100-7D-E | 1375,8 | 1200,70 | 1287,77 | 1205,57 |
| P1-100-10D-E | 1319,9 | 1154,88 | 1209,55 | 1164,46 |

Table 2 – Experimental results and code predicted column resistance, wall thickness
Experimental results and code predicted column resistance, wall thickness t=6,0mm

| Column | $F_{exp}$ (kN) | NBR (kN) | EC4 (kN) | ANSI/AISC (kN) |
|---|---|---|---|---|
| P2-30-3D-E | 1075,4 | 951,32 | 1302,30 | 951,43 |
| P2-30-5D-E | 1016,6 | 944,33 | 1186,81 | 944,62 |
| P2-30-7D-E | 1057,1 | 933,93 | 1085,47 | 934,49 |
| P2-30-10D-E | 872,2 | 912,22 | 967,29 | 913,34 |
| P2-60-3D-E | 1329,1 | 1152,68 | 1499,05 | 1152,97 |
| P2-60-5D-E | 1263,2 | 1142,75 | 1377,68 | 1143,56 |
| P2-60-7D-E | 1190 | 1128,02 | 1267,11 | 1129,58 |
| P2-60-10D-E | 1120,6 | 1097,35 | 1147,25 | 1100,45 |
| P2-80-3D-E | 1496 | 1385,45 | 1728,88 | 1386,02 |
| P2-80-5D-E | 1448,1 | 1371,54 | 1601,84 | 1373,10 |
| P2-80-7D-E | 1400,6 | 1350,94 | 1480,41 | 1353,96 |
| P2-80-10D-E | 1442,4 | 1308,18 | 1359,01 | 1314,16 |
| P2-100-3D-E | 1683,4 | 1514,21 | 1856,99 | 1514,96 |
| P2-100-5D-E | 1607,4 | 1497,84 | 1724,28 | 1499,90 |
| P2-100-7D-E | 1622,5 | 1473,61 | 1599,62 | 1477,59 |
| P2-100-10D-E | 1574,3 | 1423,42 | 1477,31 | 1431,27 |

$$\rho_{xy} = \frac{\mathrm{cov}(x,y)}{\sigma_x \cdot \sigma_y} \qquad (11)$$

where $\sigma_x$ and $\sigma_y$ are the standard deviation of variables x and y, respectively, and cov(x,y) is the covariance operator. A near zero correlation index indicates that there is no linear tendency between the variables.

Results are presented in Figure 2 and in Table 5. Figure 2 shows the correlation between model error and the studied variables for the 32 tested columns. Figure 2a reveals a negative correlation between model error and slenderness ratio, (L/D), for the Brazilian code. Since model error has a positive bias in this code, this covariance shows that bias is reduced as slenderness increases, and the model shifts towards the correct result (unitary model error mean) as slenderness increases. This occurs because the design equation converges to the exact Euler solution for large slenderness.

In terms of concrete strength, $f_{ck}$ (Figure 2b), for NBR, there is no strong linear tendency between $f_{ck}$ and model error. Results, however, tend to become more conservative when $f_{ck}$ is extrapolated. Hence the Brazilian codes resistance model tends to become more conservative for higher strength concretes. The covariance analysis between $M_e$ and t (Figure 2c) reveals that model error tends to get closer to the unitary mean for larger values of tube thickness.

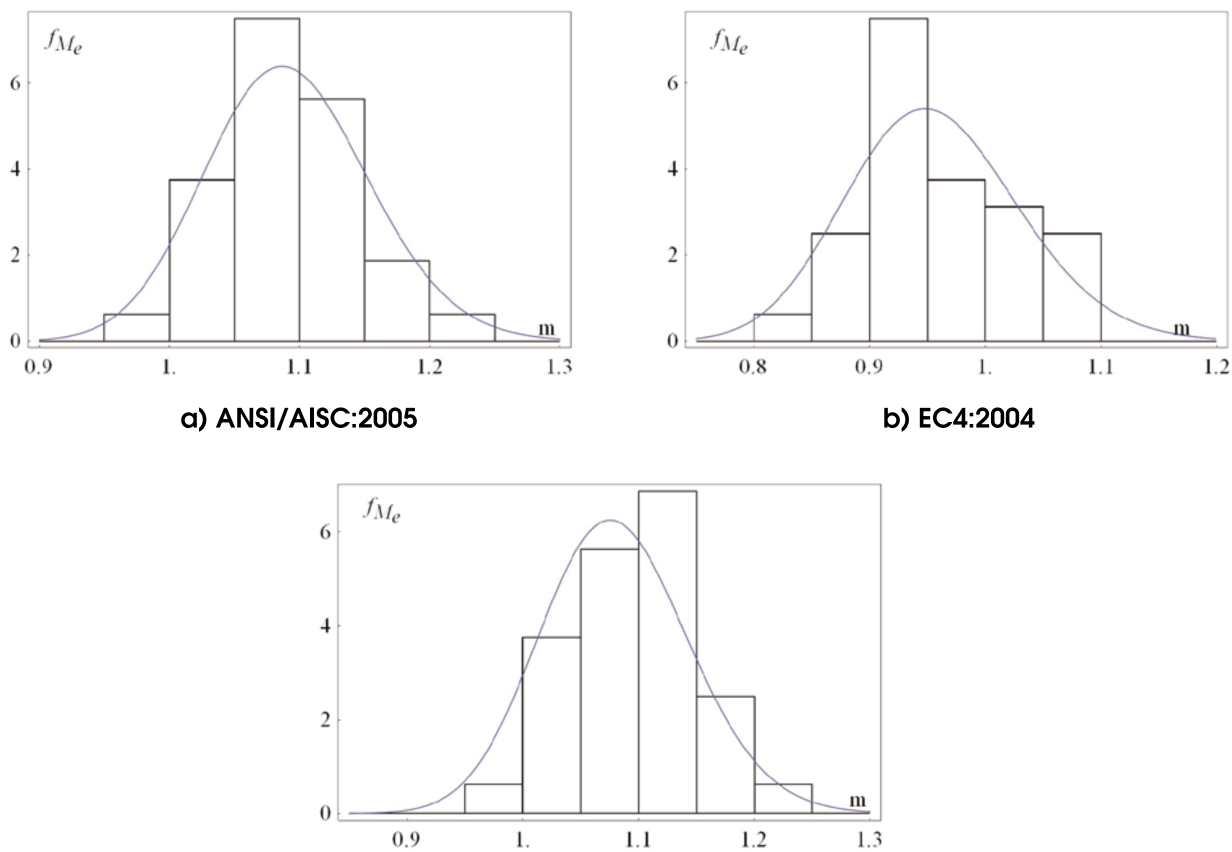
## 5. Reliability analysis

A reliability analysis is performed, taking as reference the experimental results, and assuming that experimental results are representative of the actual resistance of code-designed columns. In this analysis, column strength is evaluated by code provisions ($N_{Rs}$) and corrected by means of model error mean.

**Table 3 – Partial safety factors for actions and resistances, for the three design codes**

| Code | Resistance | | | Actions | |
|---|---|---|---|---|---|
| | Steel | Concrete | Member | Permanent | Variable |
| ANSI/AISC | - | - | 0,75 | 1,2 | 1,6 |
| EC4 | 1,00 | 1,5 | - | 1,35 | 1,5 |
| NBR | 1,10 | 1,4 | - | 1,4 | 1,5 |

**Figure 1 – Histograms of model error random variable and fitted lognormal distributions for three design codes**




a) ANSI/AISC:2005

b) EC4:2004

c) NBR 8800:2008

Initially, the relevant resistance and action random variables are identified.

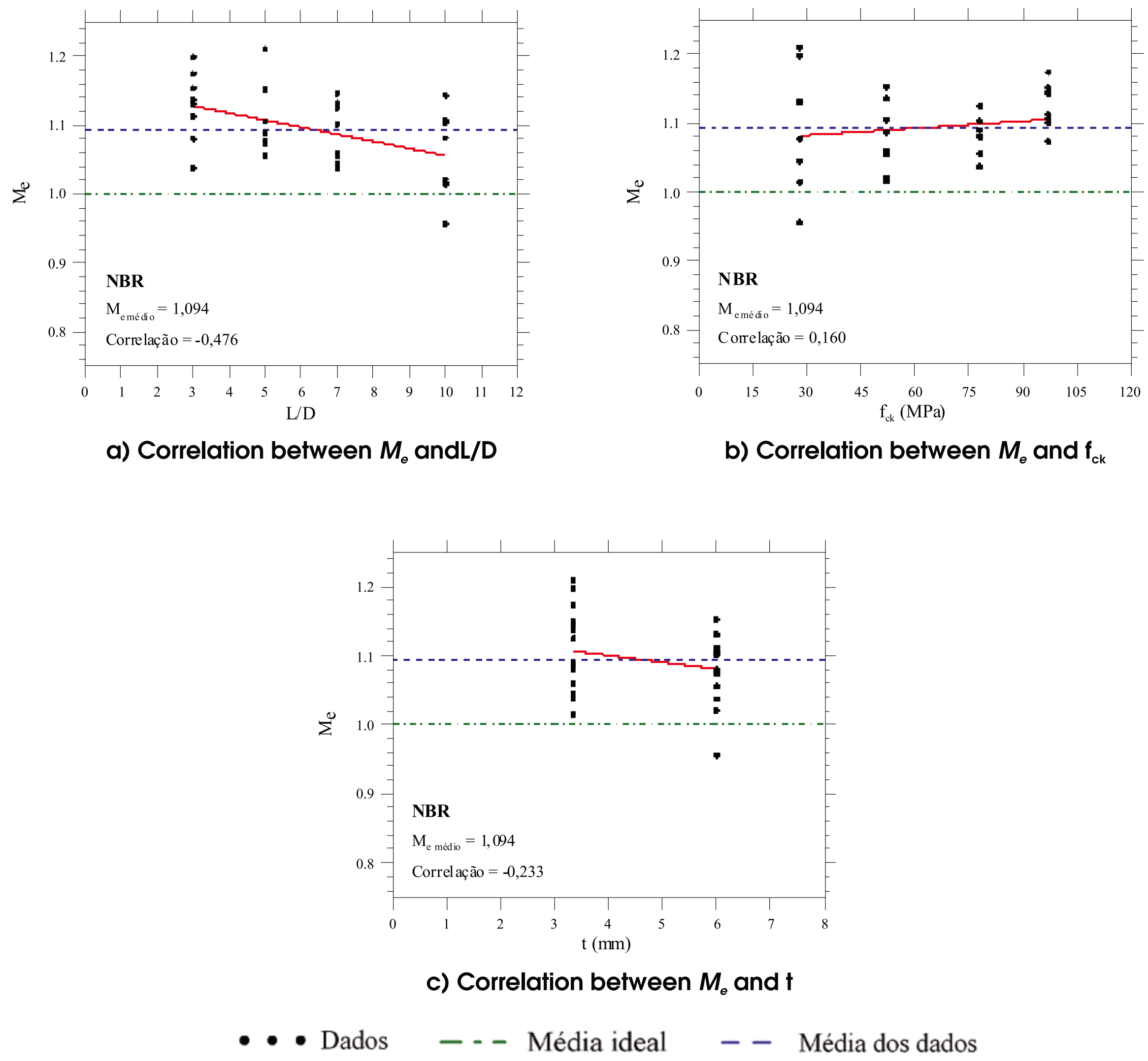
**Table 4 – Statistics of uncorrected model error random variable**

| Model error $M_e$ | Distribution | Mean | C.V. |
|---|---|---|---|
| ANSI/AISC | lognormal | 1,092 | 0,057 |
| EC4 | lognormal | 0,956 | 0,078 |
| NBR | lognormal | 1,094 | 0,057 |

## 5.1 Resistance variables

The uncorrected model error variables, $M_e$, and the corresponding moments, are presented in Table 4. The corrected model error variables, $M_e^{corr.}$, are presented in Table 6.
Other random variables that affect column resistance are yielding stress of steel and compressive strength of concrete. Moments and distributions are presented in Table 6. Parameters for steel yield strength were taken from literature [18]. Nominal values considered in the reliability analysis are: $S_{yk}$ = {250, 300, 350} MPa.
Parameters of concrete compressive strength were obtained by experimental testing. Figure 3 shows the histograms corresponding to 4 concretes used in this study, as well as the fitted probability distributions (Table 6). The histograms correspond to

**Figure 2 – Covariance analysis between model error and other column parameters, based on experimental results (NBR 8800)**




a) Correlation between $M_e$ andL/D

b) Correlation between $M_e$ and $f_{ck}$

c) Correlation between $M_e$ and t

{9, 11, 9 and 16} samples of concrete with characteristic resistances of {28, 52, 78 and 97} MPa. Characteristic resistances are obtained from:

$$f_{ck} = f_{c,mean} - 1,65 \cdot S_d \quad (12)$$

where $S_d$ is the standard deviation obtained in testing of the concrete coupons.
Other variables could also have been included in the reliability analysis, but are considered of lesser importance.

### 5.2 Load variables

In order to evaluate reliability of the columns in a service condition, the permanent action D and variable action L are incorporated in the analysis, as well as the corresponding uncertainty. Nominal values of these actions, $D_n$ and $L_n$, are evaluated from the design resistance of the columns, following each design code:

**Table 5 – Correlation coefficients between model error ($M_e$)and other columns parameters, according to experimental results**

| Code | L/D | $f_{ck}$ | t |
|---|---|---|---|
| ANSI/AISC | 0,505 | 0,141 | -0,220 |
| EC4 | 0,584 | 0,549 | -0,299 |
| NBR | 0,476 | 0,160 | -0,233 |

$$N_{Rd} = \gamma_D D_n + \gamma_L L_n \quad (13)$$

were $\gamma_D$ and $\gamma_L$ are partial safety factors for permanent and variable actions, respectively. Equation (13) is solved for the nominal values $D_n$ and $L_n$ by fixing a ratio between these actions. In this paper, six load ratios are investigated: $L_n/D_n$= {0.5; 1.0; 1.5; 2.0; 2.5; 3.0}. Parameters and distributions of random variables $D$ and $L$ are taken fro the literature [18] and are presented in Table 6.

### 5.3 Other problem parameters

In order for the reliability analysis to reflect the spectrum of design situations covered by a design code, it is important to consider a spectrum of variation for the design parameters. As shown in the paragraphs before, 3 values for steel yielding stress, 4 values for concrete resistance and 6 values for load ratio ($L_n/D_n$) are considered in this study. Besides these, 6 values of column slenderness are considered: $L/D$={3, 6, 10, 15, 20 and 25}; 3 steel tube thicknesses: $t$={4, 6, 8}$mm$ and 3 values of external diameter: $D$={100, 150 and 200} $mm$. In total, 3888 column configurations are considered in the reliability analysis.

### 5.4 Limit state equation

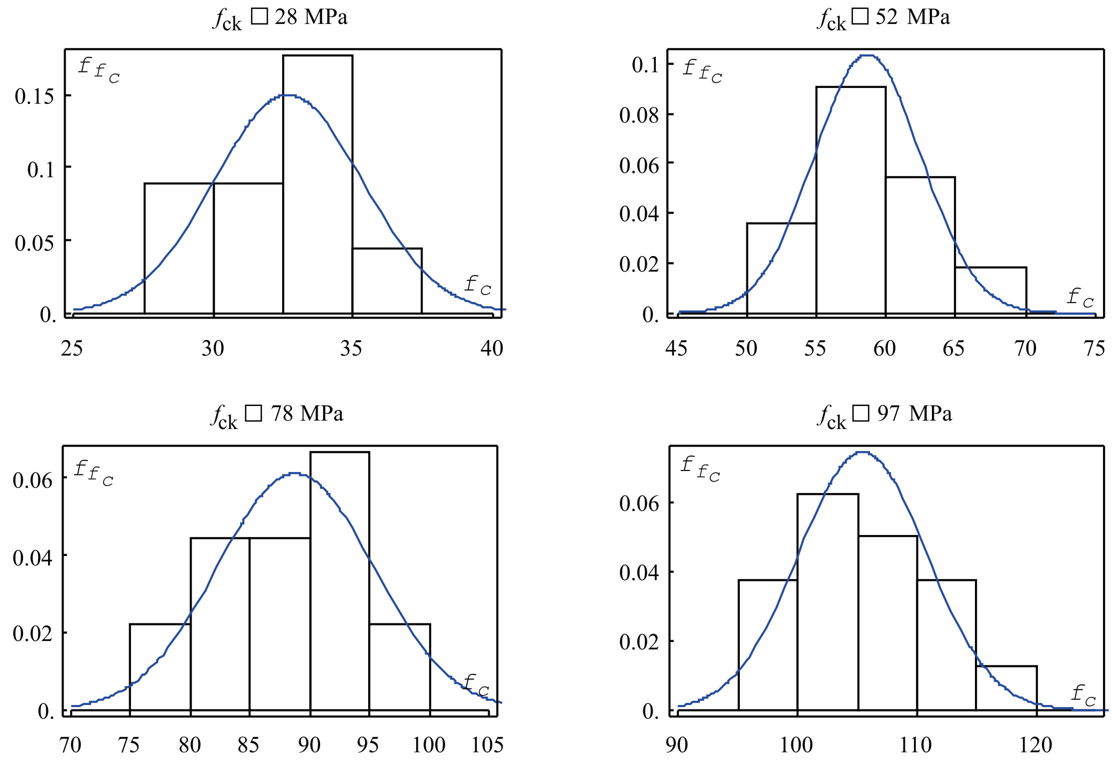
The limit state equation for the reliability analysis is:

$$g(x) = N_{Rd}(X_1, X_2) \cdot X_3 - X_4 - X_5 \quad (14)$$

**Table 6 – Random variables considered in reliability analysis**

| Variable | | Code | Distribution | Mean | C.V. |
|---|---|---|---|---|---|
| Steel yielding stress | $X_1$ or $S_y$ | - | lognormal | 1,08 $S_{yk}$ | 0,050 |
| Concrete compressive strength | $X_2$ or $f_c$ | - | normal | 32,68 | 0,081 |
| | | - | normal | 58,68 | 0,066 |
| | | - | normal | 88,78 | 0,074 |
| | | - | normal | 105,45 | 0,051 |
| Model error | $X_3$ or $M_e^{corr.}$ | NBR | lognormal | 1,00 | 0,059 |
| | | EC4 | lognormal | 1,00 | 0,077 |
| | | ANSI | lognormal | 1,00 | 0,057 |
| Permanent action | $X_4$ or D | - | normal | 1,05 $D_n$ | 0,100 |
| Variable action | $X_5$ or L | - | Gumbel | 1,00 $L_n$ | 0,250 |

Figure 3 – Histograms of experimental results for concrete strength and fitted probability distributions



were:
$X_1$ is the steel resistance;
$X_2$ is the concrete strength;
$X_3$ is the model error variable, corrected or not;
$X_4$ is the permanent action;
$X_5$ is the variable action.
These variables change according to column configuration and design code considered.
For each of the 3888 column configurations and for each design code (3 codes), column strength is evaluated by design equations (equations 1, 4 and 5), actions are determined from equation (14) and the reliability index is evaluated. Three sets of results are obtained: without model error, with model error and with model error and bias correction. In total, 34992 reliability analysis are performed. Reliability indexes are evaluated via First Order Reliability Method [19, 20], using a computational program developed by Beck [21].

## 6. Results of reliability analysis

Reliability indexes of columns designed according to the design codes are shown in Figure 4. Three distinct result sets are presented in the figure. The dashed line shows results without model error. The dotted line shows results when model error is included in the safety analysis, but not used in resistance model bias correction. The continuous line shows results when model bias is corrected, and model error is included in the analysis.
Minimum and maximum reliability indexes (β) among the 3888 column configurations are shown in Figure 4 and in the figures to follow. Hence, the distance between the upper and lower curves reflects the variations of β in the spectrum of column configurations analyzed.
For the Brazilian (NBR) and American (ANSI/AISC) design codes, Figure 4 shows that considering model error in the analysis increases reliability, because these design codes are conservative even when partial factors are set to unity. This additional safety margin only appears when model error is included in the analysis. Results also show that, for these design codes, the mean effect of model error (bias) is larger than the effect of model error variance, which always reduces reliability. Resistance model bias correction for these codes is detrimental to safety, since this removes the additional safety margin.
For the European code (EC4), incorporation of model error in safety analysis leads to a significant reduction on reliability indexes. This reduction is due to model error mean (resistance model is unconservative with unitary partial factors) and due to model error variance. Resistance model bias correction leads to a partial recovery of safety levels.
Results that ignore model error (dashed lines) are theoretical, and don't benefit from the extensive experimental results presented herein. On the other hand, the mean model correction proposed

in section 4 is not ideal, since preferably the resistance models should be investigated for possible sources of bias, which could then be removed or reduced. Correlation coefficients presented in Table 5 could be useful in this purpose.

Use of a given design code should ideally result in uniform and sufficient reliability indexes for the spectrum of structural elements covered by that design code [18]. Hence, the quality of a particular design code can be measured in terms of sufficient and uniform reliability requirements.

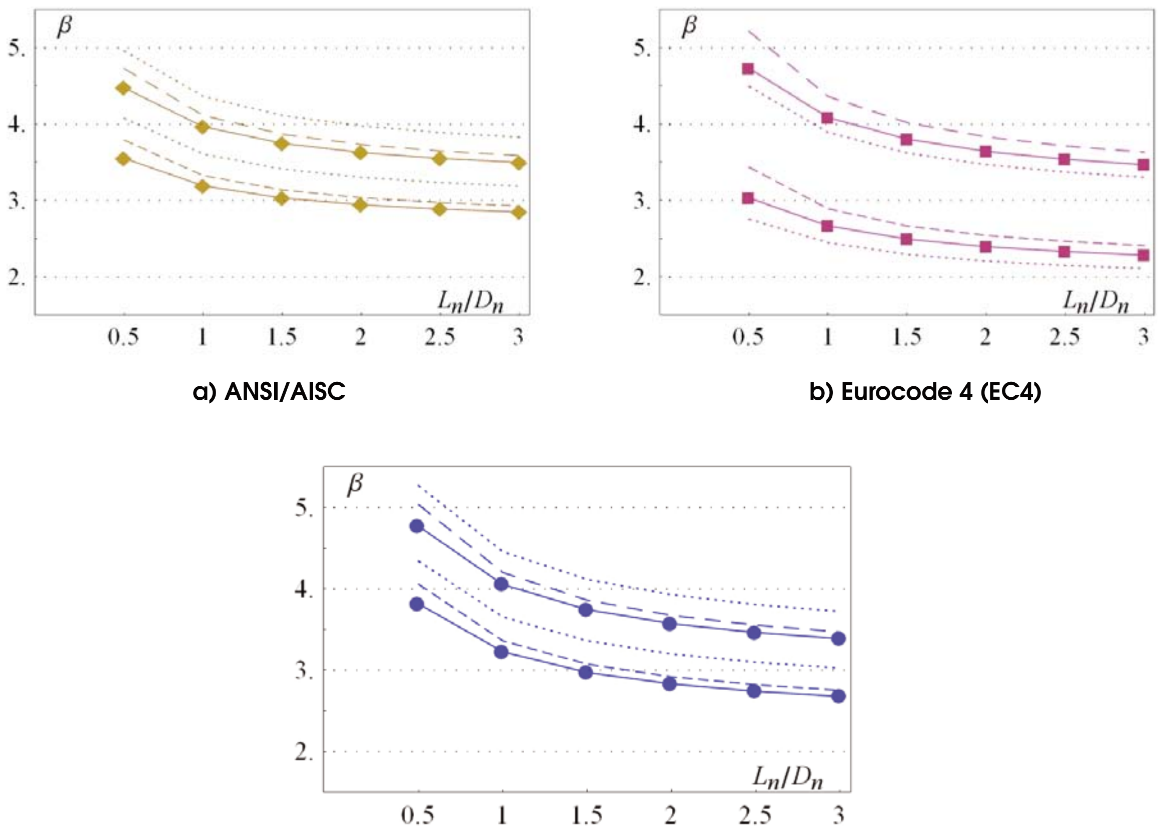
Results in Figure 4 show a clear dependency of reliability indexes with load ratio. This behavior is typical, and is known to be a consequence of adopting constant partial factors $\gamma_D$ and $\gamma_L$ regardless of load ratio. Since the coefficient of variation of the variable action (0.25) is much larger than the c.o.v. of permanent action (0.10), as the variable load increases (proportionally to permanent loads), reliability indexes are reduced.

Figure 5 shows reliability index results in terms of column slenderness, for a fixed load ratio ($L_n/D_n$=1). The same three sets of results are presented. The figure shows some reduction of reliability indexes for increasing slenderness. This is true for all design codes, although less noticeable for the Brazilian code. The figure also shows that reliability indexes are reasonably uniform over column slenderness, for the NBR code, and less so for the American and European codes. Slenderness ratios investigated in this study are well within the limits allowed by each design code. The EC4 and NBR codes allow columns with relative slenderness – $\lambda_0$ – smaller or equal to 2. For the columns considered, the largest slenderness L/D = 25, the maximum value is $\lambda_0$=1.15. For the ANSI/AISC code, slenderness limit is given by $KL/r_t$<200. For the columns considered, the maximum value is $KL/r_t$= 80.

Figure 6 shows reliability index results in terms of concrete strength, steel yield stress, steel tube thickness and diameter, for the NBR code. Results show that the NBR design procedure leads to reliability indexes that are approximately constant with respect to these design parameters. Results for concrete strength also suggest that the limit imposed by the Brazilian code ($f_{ck} \leq$ 50MPa) could be revised.

In terms of uniform reliability requirement, Figures 4, 5 and 6 show that none of the codes studied produces uniform reliability. For the Brazilian code, reliability indexes are reasonably uniform over col-

**Figure 4 – Reliability index results for ANSI (a), EC4 (b) and NBR (c) in terms of load ratio; no model error (dashed line), with model error (dotted line) and with model error and model bias correction (continuous line)**

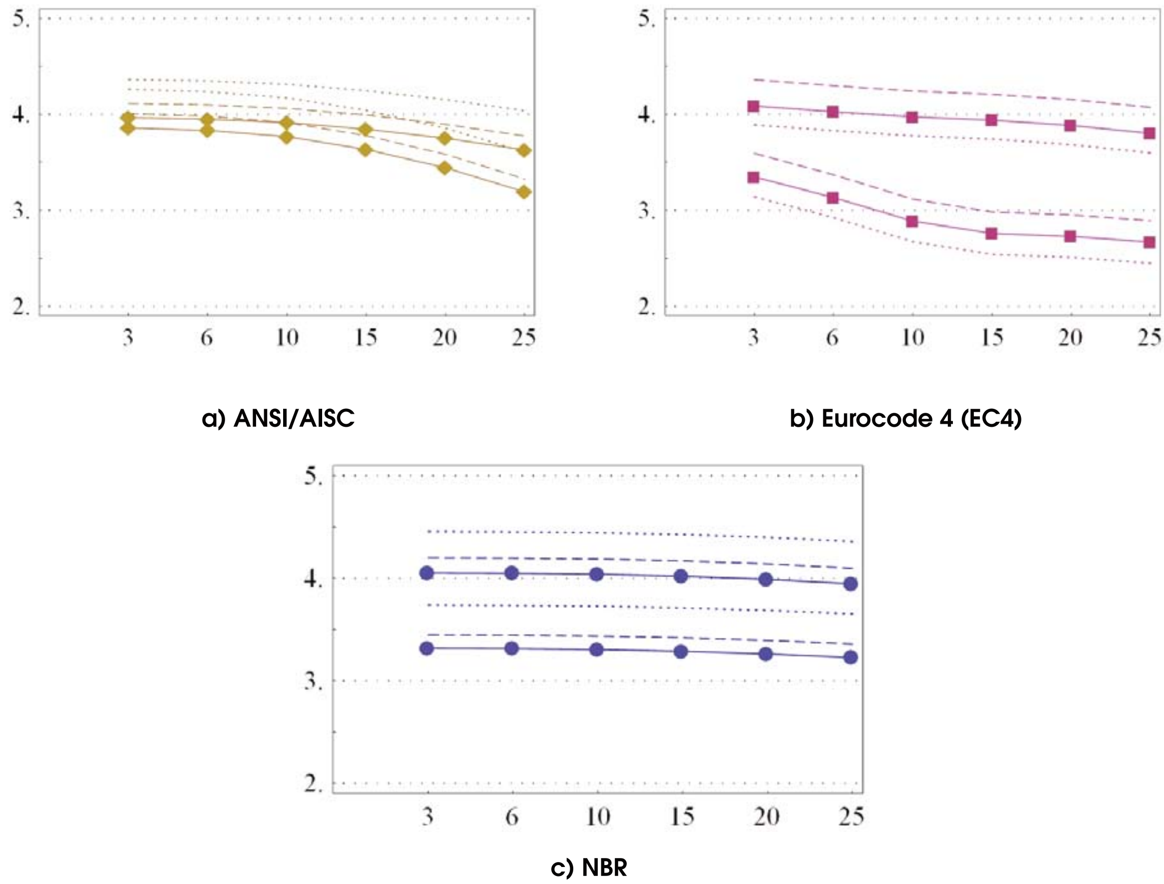



a) ANSI/AISC

b) Eurocode 4 (EC4)

c) NBR

Figure 5 – Reliability index results for ANSI (a), EC4 (b) and NBR (c) in terms of slenderness ratio L/D, for constant load ratio $L_n/D_n$=1,0.; no model error (dashed line), with model error (dotted line) and with model error and model bias correction (continuous line)




umn slenderness, steel and concrete strength, tube thickness and diameter. However, the joint variation of these parameters leads to the bands of reliability indexes seen in Figures 4 to 6.

In terms of sufficient reliability requirement, reliability indexes obtained in this study can be compared to target values used in calibration of partial safety factors for the ANSI and EC4 codes. For the ANSI code and D+L load combination, $\beta_{TARGET}$ =3.0 [18]. Results obtained herein for the NBR and ANSI codes show a conservative margin when compared to this target reliability value, being closer to it for large slenderness and large load ratios.

The European code presents a target reliability $\beta_{TARGET}$ =3.8 for 50 years and medium consequence class (residential and office buildings) [22]. Results show that this target is not met for the larger part of columns considered in this study. Results presented in ref. [23] confirm that Eurocode targets are hardly met in practice.

Finally, sensitivity coefficients (direction cosines) of the problems random variables are shown in Figure 7, in terms of load ratios. The figure shows bands of values, corresponding to the smaller and larger values obtained amongst the 3888 column configurations considered. Sensitivity coefficients show the relative contribution of each random variable in failure probabilities. Load variables appear with a negative sensitivity coefficient, whereas resistance variables have positive coefficients. The figure shows that the uncertainty in the variable action has a role which is increasingly dominant as load ratios $(L_n/D_n)$ increase. The second most important role is due to permanent loads. The third most contributing random variables are concrete strength and model error, especially for small load ratios.

## 7. Conclusions

This paper presented an investigation of the safety of steel-concrete composite columns designed according to design codes NBR8800:2008, ANSI/AISC 360:2005 and EN1994-1-1:4:2004. The investigation was based on 32 experimental results for circular concrete-filled steel columns.

Model error random variables were derived for these design codes, revealing the accuracy of design code provisions in terms of predicting column strength. Results reveal conservative bias of 9.2% and 9.4% for the ANSI/AISC and NBR desing equations, respectively, and an unconservative bias of -4.4% for the EC4 code. The standard deviation of model errors varied between a minimum of

5.7% for the ANSI/AISC and NBR codes and a maximum of 7.8% for the EC4. In a comparison to the foreign codes, the resistance model used in NBR8800 can be admitted to be well calibrated.
The study presented a safety analysis for a wide range of column configurations. This analysis revealed reliability indexes that are considered sufficient for the ANSI and NBR codes, but which are not uniform over the range of design situations. The Eurocode (EC4) presented reliability indexes well below the own target of 3.8, and which in many cases are below the American target of 3.0.
The study revealed that the Brazilian code is well balanced, with a conservative and acceptable bias of 9.4%, which represents an additional safety margin. Reliability indexes are acceptable and reasonably uniform over slenderness ratios, concrete and steel resistances, and steel tube thickness and diameter. Reliability indexes present some lack of uniformity when all these parameters vary simultaneously, and a larger lack of uniformity in terms of the ratio between variable and permanent loads. Results also show that the admitted range of concrete resistances in the Brazilian design code can be amplified in order to allow concretes of greater strength.

## 8. Acknowledgments

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Figure 6 – Reliability index results for NBR 8800 in terms other problem parameters, $L_n/D_n$=1,0; no model error (dashed line), with model error (dotted line) and with model error and model bias correction (continuous line)

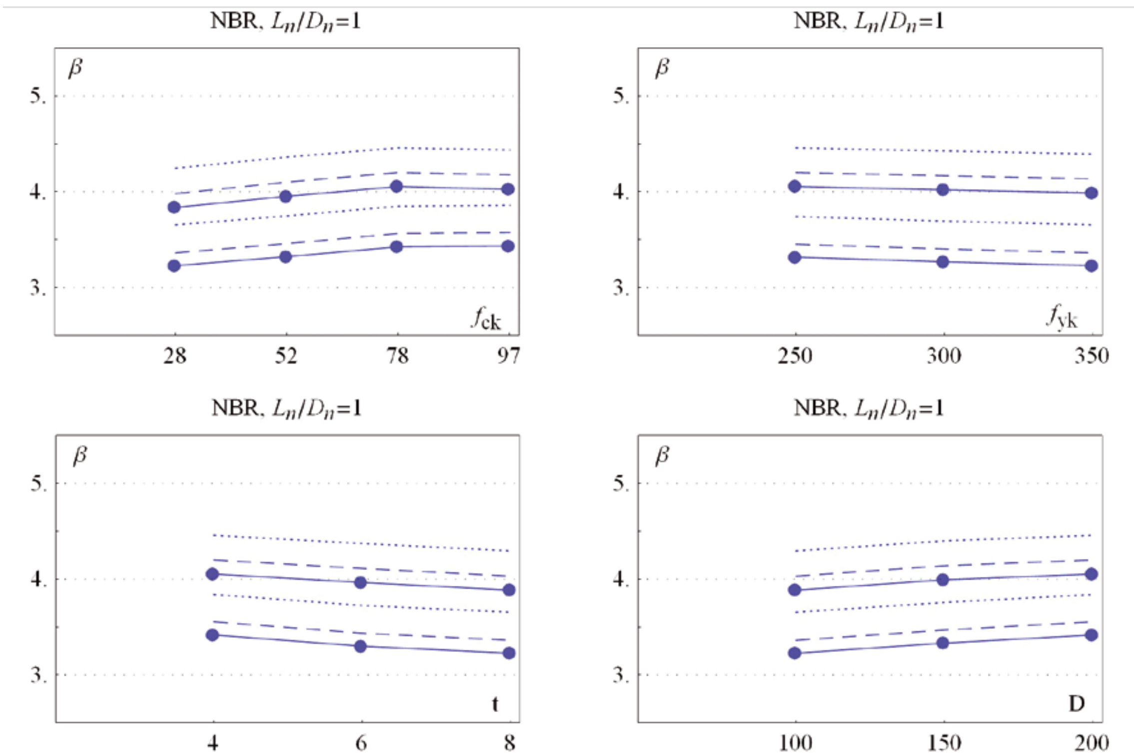
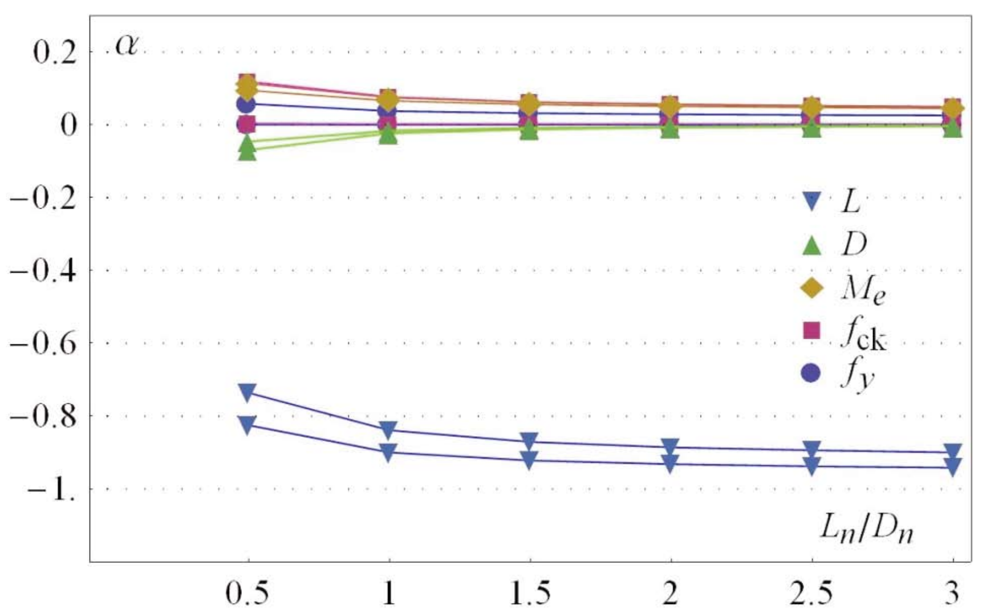


Figure 7 – Sensitivity coefficients of failure probabilities for NBR 8800 in terms of load ratio





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