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A study of brazilian concrete strength (non-)compliance and its effects on reliability of short columns

Estudo da (não-)conformidade de concretos produzidos no Brasil e sua influência na confiabilidade de pilares curtos

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Abstract

This paper presents a study of the strength (non)compliance of structural concretes produced in Brazil, and a study of the detrimental effects of non-compliance in reliability of short columns subject to axial compression. The investigation is based on experimental results of over five thousand concrete samples from different parts of Brazil. Results show that a significant part of these concretes do not reach the characteristic strength specified in design. As a consequence, these concretes should be considered non-compliant, and mitigation measures should be adopted (design revision, further testing, structural reinforcement, load restrictions and demolition/reconstruction). The study also investigates the impact of concrete strength non-compliance on the reliability of short columns subject to axial compression, when mitigation measures are not adopted. In reinforced concrete, short columns are the structural elements whose resistance most directly depends on the compressive strength of concrete. One consequence of concrete strength non-compliance is that the theoretical equation relating mean concrete resistance to specified concrete strength does not apply. Using an alternative expression, derived from the experimental results, a significant reduction in reliability of short columns is observed, due to the noncompliance of concretes produced in Brazil. These results testify to the importance of a rigorous control in the reception of concretes at construction sites, as well as in the control of the required mitigation measures, when noncompliant concrete is received.

Keywords: non-compliance of concrete strength, concrete structures, structural safety, structural reliability, short columns, axial compression.

Resumo

Este artigo apresenta um estudo da (não-)conformidade dos concretos estruturais produzidos no Brasil, bem como do impacto da não-conformidade na redução da confiabilidade de pilares curtos submetidos a compressão simples. Esta investigação tem como base ensaios de resistência de mais de cinco mil corpos-de-prova de diferentes localidades do Brasil. Estes ensaios mostram que parte significativa dos concretos atualmente produzidos no Brasil não atingem a resistência característica (fck) especificada em projeto. Como resultado, estes concretos deveriam ser considerados não-conformes, e medidas de mitigação deveriam ser aplicadas. O trabalho investiga ainda o impacto da não-conformidade dos concretos produzidos no Brasil na confiabilidade de pilares curtos de concreto armado submetidos a compressão simples, quando medidas de reforço e recuperação não são adotadas. Em concreto armado, pilares curtos são os elementos estruturais cuja resistência mais diretamente depende da resistência à compressão do concreto. Uma consequência da não-conformidade dos concretos é que a equação teórica de norma, que relaciona a resistência média com o valor característico especificado em projeto, não pode ser utilizada na análise de confiabilidade. Utilizando equação equivalente, determinada a partir dos resultados experimentais, verifica-se uma redução significativa da confiabilidade dos pilares curtos em função da não-conformidade dos concretos. Estes resultados reforçam a necessidade de um controle rigoroso no recebimento do concreto, bem como na fiscalização das medidas de mitigação no caso dos concretos não-conformes.

Palavras-chave: não-conformidade da resistência do concreto, estruturas de concreto, segurança das estruturas, confiabilidade das estruturas, pilares curtos, compressão simples.

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1. Introduction

This article presents a study of the strength of plant concretes produced in Brazil, and of the (non)compliance to specified strengths. The study also investigates how the non-compliance affects the reliability of short columns subject to axial compression. The study is based on results of more than five thousand specimens produced in loco at different construction sites in different parts of Brazil.
Structural reliability theory is employed to analyze the impact of concrete strength non-compliance in the safety of reinforced concrete structures produced in Brazil. The study consists in establishing reliability indexes for short columns designed following Brazilian codes, but produced using the concretes effectively delivered at Brazilian construction sites. The study shows that a significant part of these concretes does not comply with design-specified strengths. The safety analysis considers different reinforcement ratios and different ratios of dead to live loads. The effect of concrete confinement by reinforcing steel is not taken into account. The analysis is made for four characteristic values of concrete strength – C20, C30, C40 e C50.
This study is motivated by the long range goal of performing the reliability-based calibration of Brazilian design codes. Initiatives in this regard have already been taken in Beck and Doria [1], Beck et al. [2], Chaves et al. [3] and Beck and Souza Jr. [4].

2. Context and relevance

Plant concrete which is delivered today at construction sites in Brazil do not always reach the required strength as specified in design [5]. Concrete strength noncompliance results, naturally, in structures whose reliability level is not the same as specified in design codes.
The problem of concrete strength non-compliance permeates the whole civil construction production chain: structural design offices and professionals, concrete plants, and technical quality control laboratories. The relevance of the subject can be evaluated from a recent round-table "concrete strength: is your facility safe?" promoted by SINDUSCON-BA on the 18th may 2010 in the city of Salvador [6]. The theme importance can also be grasped from the creation of specific study groups like the "technical committee on concrete strength compliance" from the Brazilian Association of Structural Engineering (ABECE) [7].
Concretes with non-compliant strengths result in economical losses, which include the necessity of design reevaluation, further testing, execution of reinforcements and also demolition and reconstruction. Further, significant losses arise from the time lost and from the prejudice to the image of involved companies[5].
Brazilian code NBR 12655:2006 [8] specifies how the statistical control of concrete should be performed upon reception at a construction site. Brazilian code NBR 6118:2003 [9]
describes how to proceed in case a non-compliant concrete is detected. If the characteristic strength obtained in testing coupons molded in-loco (f_{ckest}) results less than the specified strength (f_{ck}) - case of non-compliant concrete -, corrective actions must be adopted, as specified in NBR 6118:2003 [9] as detailed in [10]:

1. revision of structural design, considering the characteristic strength found in testing the coupons molded in-loco with the delivered concrete;
2. if non-compliance remains, extract coupons from the actual structure, following NBR 7680:2007 [11]; from these coupons, obtain a new estimate of f_{ck} and make a new verification of the project; if necessary, specify load restrictions for the structure;
3. if the noncompliance persists, chose to:
   a – provide reinforcement of the structural member produced with the noncompliant concrete;
   b – decide for the partial or complete demolition.

Results presented in this article do not refer to one particular constructed facility, much to the contrary: this article illustrates the problem of non-compliance for general facilities produced with the concretes actually delivered at building sites in Brazil. Because the study doesn´t address particular facilities, the effect of adopting measures of reinforcement and recuperation cannot be considered. Hence, the study investigates the impact of concrete strength non-compliance in general constructed facilities when mitigation measures are not adopted. Also, the study presented herein cannot be used to justify the non-adoption of mitigation measures when non-compliance is detected at a given facility: these measures must follow guidelines of the technical codes [8] and must consider the actual value of characteristic strength obtained for the concrete delivered at the site and employed in the given facility.

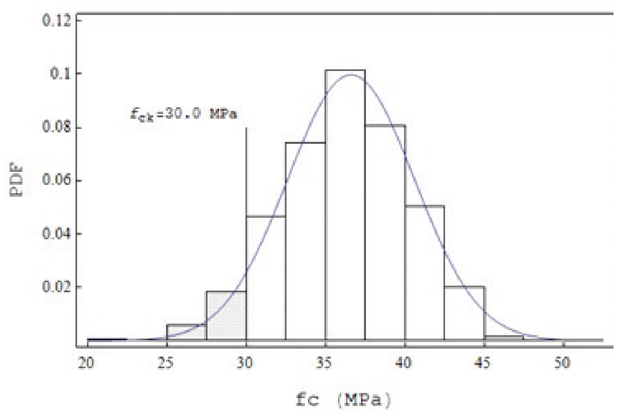
3. Characteristic strength of concrete (f_{ck}) in compression

Concrete strength is random by nature. Uncertainty and variability of concrete strength originates in the non-homogeneity of materials, in the variability on the properties of constituting materials and in mixture imprecision. Hence, n specimens of the same batch of concrete, when tested, will result in n different results for the concrete strength. Variability within specimens of the same batch will generally be smaller than variability between specimens of different batches. but cannot be ignored.
Uncertainty in concrete resistance increases as the scope of concretes considered increases. It is understood that a coupon extracted from a given location of a structure represents with good precision the resistance of that concrete (excluding possible technical problems in coupon extraction). The concrete from one batch is delivered to a region of the structure, and has it´s resistance measured indirectly from coupons molded in loco upon reception of the concrete. Concrete from a whole structure is delivered in different batches, hence will present a variability in properties which is larger than for a single batch. Concretes produced in different plants will have even larger variability. Concretes mixed in loco have, generally speaking, larger variability than plant concrete. Finally, concrete considered in a design code must reflect the variability of all concretes (of the same class) produced in the country(ies) where the code applies. Herein, "design code concrete" refers to models (equations) used in design codes to:

1. specify the reference strength for the mixture;
2. specify the procedures for reception and control of concretes in construction sites [8];
3. specify what the design code considers to be a compliant concrete [8, 10].

The random nature of concrete strength demands for a probabilistic approach. Figure 1 shows a histogram (obtained by simulation) of the compressive strength of a code-compliant concrete of class

Figure 1 – Results obtained by simulation of a strictly compliant C30 concrete



C30. Such concrete strength, as any other, can be characterized by a mean strength (f_{cm}), a standard deviation (σ) and a probability distribution function. Well known international results [12, 13] show that a Normal distribution is appropriate to describe compressive strength of concrete. A Normal distribution, together with moments (or parameters) f_{cm} and σ describe concrete strength completely. However, a probabilistic description is not viable for the daily professional practice of structural design. Hence, to simplify the practice of structural design, it is chosen to work with a reference value, which is the characteristic value of concrete strength (f_{ck}). By convention, the characteristic value is chosen such that 95% of the tested specimens, or 95% of a given volume of concrete, has strengths larger than the characteristic value. This convention, together with the observation that concrete strength follows a Normal distribution, leads to the well-known design equation:

$$f_{ck} = f_{cm} - 1{,}65.\sigma \tag{1}$$

In the histogram (Figure 1), concretes whose resistance is below the characteristic value appear shaded: these correspond to the 5% accepted by the codes. Design codes accept these 5% of concretes whose resistance is below f_{ck} because there is no other way around the random nature of concrete strength. If one considers safety in structural design, 5% is too much, therefore a partial safety factor on concrete resistance is used. The design value of concrete strength, following Brazilian codes [9], is $f_{cd}= f_{ck}/1.4$.

Equation (1) can be used for:

1. evaluation the characteristic strength of a batch of concrete, for a minimum number of 20 specimens [9] from which f_{cm} and σ are evaluated;
2. evaluate the third parameter, when two are already known (example: to evaluate the reliability of a structure produced with compliant concrete, mean value can be evaluated from specified f_{ck} and known σ);
3. establish the reference strength for the mixture (f_{cm}), so that produced concretes result compliant [8] when tested after 28 days.

When used to specify the reference strength for the mixture [8], equation (1) is written as:

$$f_{cm} = f_{ck} + 1{,}65.\sigma_d \tag{2}$$

where f_{ck} is the characteristic strength specified by the designer, f_{cm} is the mean resistance expected for testing at 28 days and σ_d is the design value of the standard deviation. σ_d is either code-specified [8], or the value obtained by a given plant in previous deliveries of the same concrete. For a condition of best quality control (condition A), Brazilian code [8] specifies σ_d=4 MPa. Hence, following Eq. (2), if plant history is not taken into account, reference resistance for the mixture should be at least 6,6 MPa larger than the characteristic resistance specified by the designer.

Once the concrete is produced, it is delivered to the construction site and poured in the structure. Quality control, executed by molding in loco cylindrical specimens which are tested after 28 days, has the objective of verifying if the produced concrete effectively reached the characteristic strength specified by the designer. The Brazilian code for reception and control of concrete [8] allows total or partial sampling controls. In total sampling control, two specimens are molded out of each batch of concrete. Resistance to be considered is the largest value between these two specimens. Preferably, the places where the concrete is poured in the structure should be tracked, in order to allow for design reevaluation is case the concrete is found to be non-compliant. In the control with partial sampling, at least 6 specimens are molded for each 50 to 100 m³ of concrete poured into the structure [8]. When quality control is by partial sampling, and when the number of specimens is greater or equal to twenty, Eq. 1 is used to estimate the characteristic resistance [8]. In case of total sampling control and for more than twenty specimens, characteristic strength is estimated based on the 0.05 percentile of the samples. Testing results are ordered such that $f_1 < f_2 < f_3 < \cdots <$, and characteristic strength is given by [8]:

$$f_{ckest} = f_{int[0.05n]} \tag{3}$$

where int[] represents the integer part.

At the end of the quality control, a batch or batches of concrete are considered compliant when the estimates value of characteristic strength (f_{ckest}) satisfies [8]:

$$f_{ckest} \geqslant f_{ck} \tag{4}$$

4. Objectives and experimental database

Results presented in this article are based on compression tests

Table 1 – Quantification of speciments forming the concrete strength database

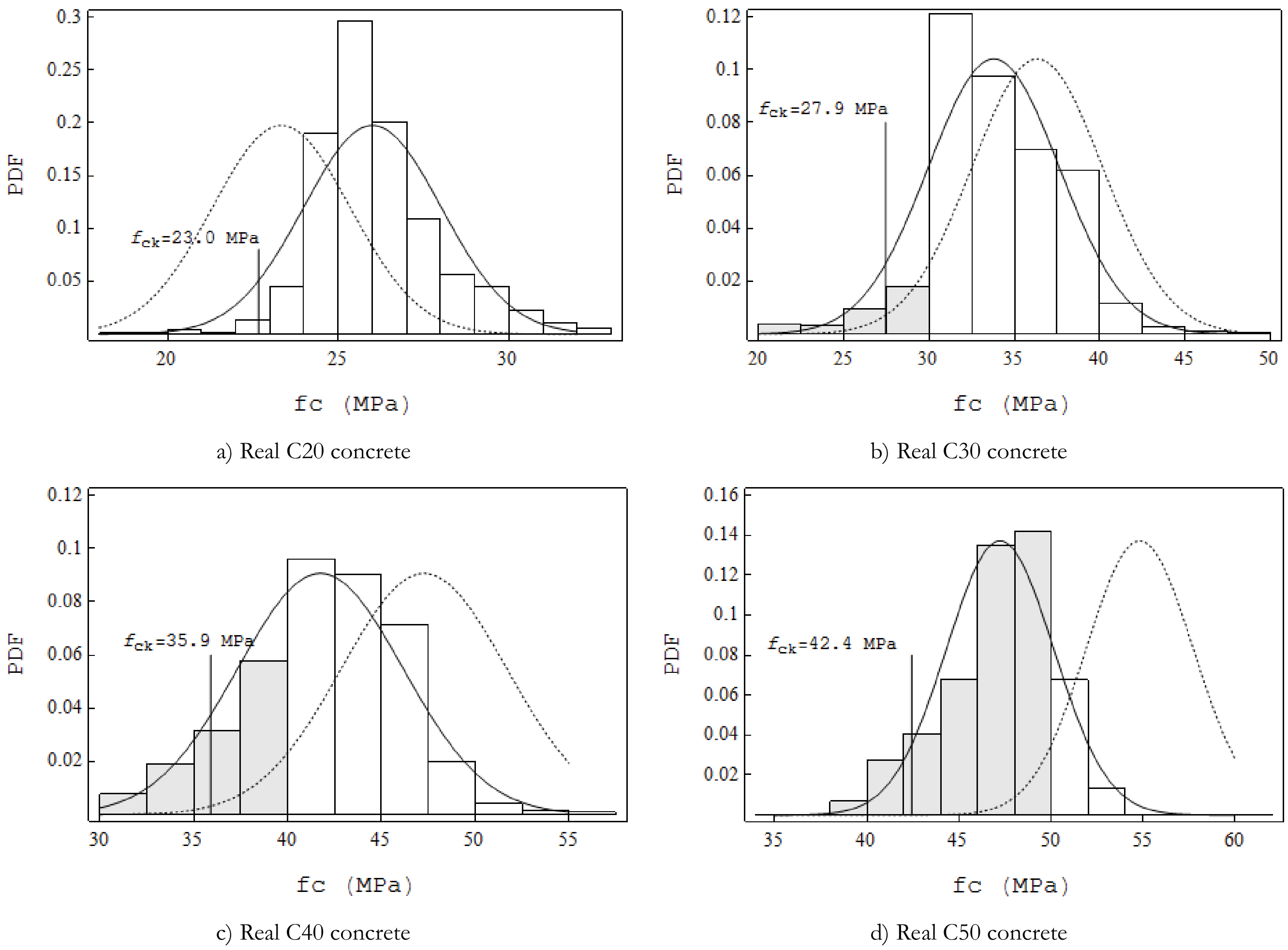
Class	Number of specimens
C20	896
C30	1052
C40	3742
C50	148

of over five thousand specimens, molded in loco at different construction sites in different parts of Brazil, and tested after 28 days of molding. Table 1 describes the distribution of specimens in terms of concrete classes. The database contains results obtained in nine Brazilian states – Alagoas, Bahia, Ceará, Distrito Federal, Maranhão, Minas Gerais, Paraíba, Rio de Janeiro and São Paulo – and was obtained from structural designers and offices, as follows: Antonio Nereu Cavalcanti Filho of TECNOCON, Cesar Pinto of CSP Projetos e Consultoria em Estruturas, Luiz Felipe Ferreira Mello of SILCO Engenharia, Renato Trindade of AJL Engenharia, Otávio Luiz do Nascimento of CONSULTARE, Egydio Herve Neto of Ventuscore Solucoes em Concreto and Marcos Carnaúba. This database does not cover all of Brazil, but a significant part of the country.

Certainly, the authors would like to dispose of results for other parts of Brazil, in order to increase representativeness of the database and reevaluate results presented herein. In order to achieve this objective, the authors call upon structural designers and consulting offices to make their data available.

Given this observation about the limitations of database representativity, an in order to simplify the discussion to follow, the present database is considered to be representative of the situation of concretes produced throughout the country.

Figure 2 – Comparison of histograms (and distribution functions) for real concretes



a) Real C20 concrete

b) Real C30 concrete

c) Real C40 concrete

d) Real C50 concrete

Table 2 – Results obtained from the statistical analysis of the concrete strength database

Class	f_{ckest} (MPa) (Eq. 1)	f_{ckest} (MPa) (Eq. 3)	f_{cm} (MPa)	σ (MPa)	C.V.	f_{ckest}/f_{ck}	Percentage on non-compliant samples
C20	23,0	23,7	26,0	1,847	0,071	1,15	1
C30	27,9	28,0	33,8	3,554	0,105	0,93	9
C40	35,9	33,6	41,7	3,570	0,090	0,90	30
C50	42,4	41,1	47,2	2,910	0,062	0,85	84

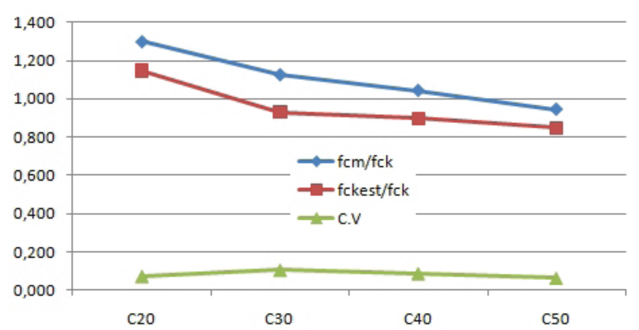
Objectives of the present study can be divided in:

1. Identify trends and eventual misconduct of plant concretes produced in Brazil, which result in concrete non-compliance. In case of non-compliant concrete, Eqs. 1 and 2 don´t apply.
2. Establish equations equivalent to Eqs. 1 and 2, which reflect the reality of concretes actually produced in Brazil, and which may be used in reliability analyses.
3. Quantify the effect of concrete non-compliance in the safety (reliability) of structures produced in the country, when mitigation measures are not adopted.

A (further) long range goal, which will be subsidized by results presented herein, is to perform the reliability-based calibration of partial safety factors of Brazilian structural design codes (NBR8681 and NBR6118) [4].

Objectives 1 and 2 listed above differ significantly from the scope of the quality control code for concrete [8]. This code addresses types of control (partial or total sampling), differentiates between mixing conditions (A, B or C), establishes minimum numbers of specimens and the conditions under which concrete compliance is evaluated for individual batches of concrete. The objective of this study is to obtain a probabilistic description which is representative of all plant concretes produced in the country. Hence, it is not important to know if batch A of plant B resulted non-compliant, because the following reliability analysis refers to design codes which apply in the whole country (same partial safety factors are used everywhere). Moreover, the way specimens are molded and the testing procedure is the same for all concretes, regardless of type of quality control or mixture condition. Therefore, data originated from different plants and batches can be grouped and analyzed in block, for a given geographical location. In this study, concrete data is categorized into regions (south, south-east, northeast) and local statistics are evaluated. National statistics are then evaluated as weighted means of local statistics, where the weight is given by the total numbers of samples of each concrete class in each geographical region.

Figure 3 – Results from the statistical analysis of concrete strength database



5. Strength and (non-)compliance of concretes produced in Brazil

Although statistical quality control is being performed upon reception of concrete in Brazil, it is widely known that a significant part of these concretes presents non-conforming strengths [5, 6, 7]. Table 2 summarizes results obtained in this study, from the database described in Table 1. These results are presented graphically in Figures 2 and 3.

Figure 2 shows the histograms obtained from compression tests of cylindrical specimens molded in loco and tested after 28 days, with monotonic loading in standard velocity. The shaded portion of the histograms shows those samples whose resistances resulted less than the characteristic strength. The corresponding percentiles are shown in the last column of Table 2. The figure also shows the characteristic strength estimated from the experimental results using Eq. 1. The continuous line shows the Normal probability distributions adjusted to the histogram data. The dotted line shows the hypothetical probability distributions that these results should follow, if the concretes where strictly compliant ().

Table 2 shows results in terms of f_{ckest}, f_{cm}, σ , coefficient of variation (σ / f_{cm}), ratio (f_{ckest} / f_{ck}) and the percentile of non-compliant concretes in each class. Figure 3 illustrates the change in these parameters as a function of concrete class. It can be observed that results are quite consistent, despite the in-homogeneity of the database in terms of concrete classes and geographical locations. It is also observed that, although the number of samples for C50 is quite small, curves of f_{ckest} / f_{ck} and f_{cm} / f_{ck} follow the same tendency as for the other concretes.

In Table 2, f_{ckest} is evaluated from experimental results using Eqs. 1 and 3. As should be expected, there is large agreement between these results. Equations 1 and 3 are equivalent, but Eq. 1 assumes a Normal distribution, whereas Eq. (3) assumes an empirical distribution of the data ($F_i = i/n$). Evaluation of other parameters (like f_{ckest} / f_{ck}) follows results of Eq. 1.

It can be observed in Figure 2, as well as in Table 2, that $f_{ckest} < f_{ck}$ for concretes of classes C30, C40 and C50. Hence, the set of experimental results for these concretes, as a whole, can be considered non-compliant. The order of non-compliance can be evaluated qualitatively from Figure 2, from the difference between dotted and continuous probability distribution lines. The order of non-compliance can be quantified from the percentile of samples whose strength is below the specified characteristic strength, in the last column of Table 2. By virtue of design code this percentile should be limited to 5%. However, much larger percentiles are observed for C30, C40 and for C50 concretes. It becomes evident that the problem of concrete non-compliance is more significant for concretes of larger nominal resistance.

Results obtained for σ show that the standard deviation specified in the code to obtain the reference resistance for the mixture (σ_d=4 MPa for preparation condition A and best quality control [8]) is slightly conservative, which serves as a safety margin so that concretes mixed following Eq. (2) result compliant.

However, results presented herein show that the reference strength mixing equation is not being respected by (some) Brazilian plants: for C40 and C50 concretes, **mean** strength was found to be close to the **characteristic strength** specified in design. This confirms observations by Grandiski (in the discussion which is part of reference [5]) that plants have been centering the reference mixing strength on the desired characteristic strength, and not on the mean (which is roughly 6,6 MPa larger, as commented). This result certainly has a negative impact on the reliability of reinforced structures produced in Brazil. One measure of this impact is the ratio f_{ckest}/f_{ck}, which is around 0.9 for the C40 concrete (largest experimental dataset). Given this reality, not taking mitigation measures (by hypothesis) is equivalent to design reinforced concrete structures with partial safety coefficient γ_c=1.4·0.9=1.26. Quantification of this impact, however, requires reliability analyses, as presented in the sequence.

Results presented herein show that concretes of classes C30, C40 and C50 produced in Brazil are resulting non-compliant. Hence, Eqs. 1 and 2 cannot be used in reliability analyses. In this type of analysis, it is common to reconstruct the statistics of concrete strength from the specified characteristic strength. Hence, for the reliability analysis of a general structure produced in Brazil with the actual concretes delivered by the plants, Eq. (5) should be used, based on results presented in Table 2.

$$
\begin{aligned}
&\text{C20:}\quad f_{cm} = 1{,}15.f_{ck} + 1{,}65.\sigma\\
&\text{C30:}\quad f_{cm} = 0{,}93.f_{ck} + 1{,}65.\sigma\\
&\text{C40:}\quad f_{cm} = 0{,}90.f_{ck} + 1{,}65.\sigma\\
&\text{C50:}\quad f_{cm} = 0{,}85.f_{ck} + 1{,}65.\sigma
\end{aligned}
\tag{5}
$$

Use of Eq. 5, together with the experimental standard deviations presented in Table 2, allows the statistics of real strengths for concretes produced in Brazil to be reconstructed.

6. Structural reliability analysis

The non-compliance of concrete strengths, verified through experimental results, negatively affects the safety of reinforced concrete structures produced in Brazil, if mitigation measures are not adopted. In this section, the effect of concrete non-compliance of the reliability of short columns is investigated, by comparing reliability results for compliant (design code) and non-compliant (real) concretes.

Within all structural elements in reinforced concrete, resistance of short columns has the strongest dependence on concrete strength. Hence, in evaluation of the effect of concrete non-compliance, only short columns subject to centered normal loads are considered herein. Design specifications [9] require a minimum eccentricity to be considered in the design of short columns. Eccentricities cause second order moments which may lead to tensile stresses in portions of the columns cross-section. However, the largest effects of concrete strength non-compliance are observed when the whole

Table 3 – Random variables considered in reliability analysis

Variable			Distribution	Mean	C.V.	Reference
f_c	Compliant	C20	normal	f_{ck} + 1,65.σ	0,080	This article
		C30			0,099	
		C40			0,078	
		C50			0,053	
	real (non-compliant)	C20	normal	Eq. (5)	0,072	This article
		C30			0,105	
		C40			0,090	
		C50			0,062	
C			normal	0,003.b (mm)	$\frac{4\text{mm} + 0{,}006.b}{0{,}003.b}$	(16)
f_s			lognormal	1,12.f_{sk}	0,050	(16)
D			normal	1,05.D_n	0,100	(12)
L			Gumbel	1,00.L_n	0,250	(12)

cross-section is under compressive stresses. Hence, no eccentricities are considered herein.
Columns are linear elements, usually vertical, whose function is to receive actions acting on different levels of the structure and transmitting them to the foundations. Together with the foundations, columns are the most important structural elements in a construction, since the collapse of a single column can lead to global damage and even to overall progressive structural collapse [14, 15].

6.1 Resistance variables

Steel and concrete strengths are the most significant random variables affecting the resistance of short columns. Moments and probability distributions for these variables are presented in Table 3. Reinforcing steel of class CA-50 with characteristic yield stress of 500 MPa was considered. Parameters of steel resistance were obtained from the literature [16],
Concrete strengths were evaluated from Eq. (2), when the concrete is assumed compliant (for comparison) and from experimental results (Table 2 and Eq. 5) for real concretes. In both cases, the experimental standard deviations reported in Table 2 were used.
Another source of uncertainty in column strength is cross-section dimensions, arising from imperfections in form work. Square cross-section columns are considered, with sides b=30 cm. Random variable C, which quantifies dimensional uncertainties from nominal dimensions, is taken from the *Probabilistic Model Code* [16] and is presented in Table 3.

6.2 Design equations and load variables

In order to represent expected actions on a real structure, random variables dead load (D) and live load (L) are also considered. Nominal values of these actions, D_n and L_n, are evaluated following the corresponding design code provisions [9, 17].
In evaluating reliability of a general short column (that is, without modeling a particular building), the conventional design order is inverted. Instead of designing a column cross-section in order to support a specified loading, the cross-section is pre-defined and code rules are used to evaluate the maximum loading that could be imposed on that column. Hence, once the characteristic strength of concrete (f_{ck}), cross-section dimensions (b x b) and reinforcement ratio ρ are selected, design strength of the short column is given by:

$$R_D = \frac{0{,}85.[b^2 - A_s].f_{ck}}{\gamma_c} + A_s.f_s \quad (6)$$

where coefficient 0.85 accounts for strength reduction due to the Rüsch effect, γ_c=1.4 is the concrete partial safety factor (NBR6118:2003 [9]), f_s is the steel stress corresponding to the limit concrete deformation (2,0‰) and A_s is the steel area. Once the design resistance is evaluated, it is made equal to the design load:

$$R_D = S_D = \gamma_D D_n + \gamma_L L_n \quad (7)$$

where γ_D=1,4 and γ_L=1,4 are the partial safety factors on loads, given by NBR6118:2003 [9] and NBR8681:2003 [17]. These values, hence the results that follow, correspond to the design of type 2 buildings (those for which live actions do not exceed 5kN/m²), following NBR8681:2003 [17].
Dividing Eq. (7) by D_n and rearranging terms, one obtains:

$$D_n = \frac{R_D}{\gamma_D + \gamma_L L_n / D_n} \quad (8)$$

For a chosen load ratio (L_n/D_n), one finds the nominal value of the dead load (D_n), and hence the nominal value of live load (L_n). From these nominal values, and using known parameters and probability distributions [12] (Table 3), the random variable dead and live loads (D and L) are reconstructed.
Eight load ratios are considered herein: L_n/D_n={0,1; 0,4; 0,7; 1,0; 1,3; 1,6; 1,9; 2,2}. Following Ellingwood and Galambos [12], the typical range of load ratios for concrete structures is $0.5 \le L_n/D_n \le 1.5$. Following Szerzen and Nowak [18], the typical range of load ratios for reinforced concrete columns is $0.1 \le L_n/D_n \le 1.5$. Hence, the range of load ratios considered herein is slightly larger than typical values. In the interpretation of results, one should remember that, for type 2 buildings, the load ratio is usually $L_n/D_n \le 1.0$.

6.3 Reinforcement ratios

It is important that the reliability analyses cover the range of design conditions expected in practice. Hence, four classes of concrete resistance and eight load ratios (L_n/D_n) are considered. In addition, three values of reinforcement ratios are considered: ρ={ρ_{min}; 2,0%; 4,0%}. The minimum reinforcement ratio (ρ_{min}) follows specifications for the design of short columns [9].

6.4 Limit state equation

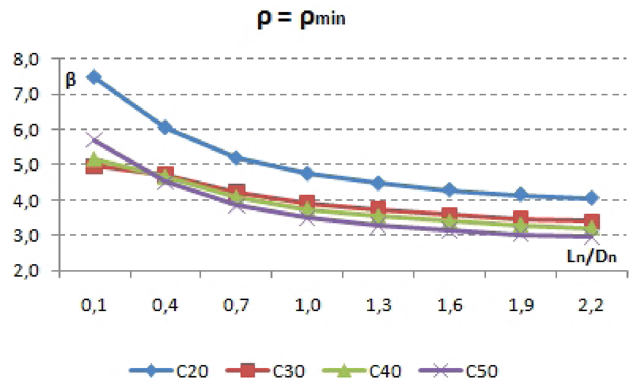
For a short concrete column of square cross-section (nominal dimensions b x b) and steel area A_s, the ultimate capacity limit state equation is:

$$g(X) = 0{,}85.[(C+b).(C+b) - A_s].f_c + A_s.f_s - D - L \quad (9)$$

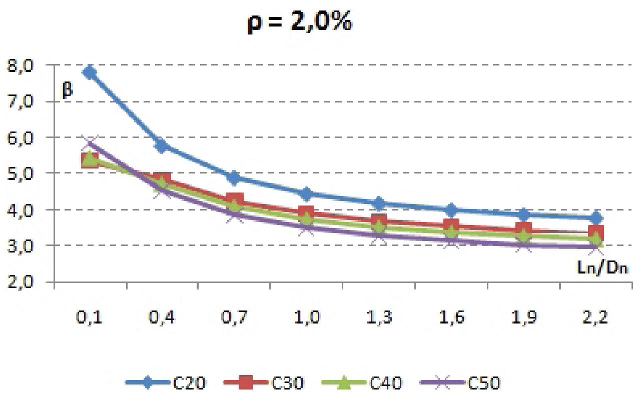
where:
C is the random variable which quantifies dimensional variability from nominal dimensions () [16];
f_c is (random variable) concrete strength;
f_s is (random variable) steel stress corresponding to limit concrete deformation (2,0‰);
D is (random variable) dead load;
L is (random variable) live load.
The limit state equation (Eq. 9) and design equations (Eqs. 6 and 7) are similar; however it is important to note that Eq. (9) involves random variables and unitary partial safety factors, whereas design equations (Eqs. 6 to 11) involve characteristic values of material strengths, nominal values of loads and non-unitary safety factors.

In order to study the effect of concrete non-compliance in the reliability of short columns, two sets of results are obtained herein: considering the theoretical equation (Eq. 2, which assumes compliant concretes) and the real relation (Eq. 5, which reflects experimental results for actual concretes). Reliability indexes are evaluated through the first order reliability method [19, 20] using computational code StRAnD: Structural Reliability Analysis and Design developed by Beck [21].

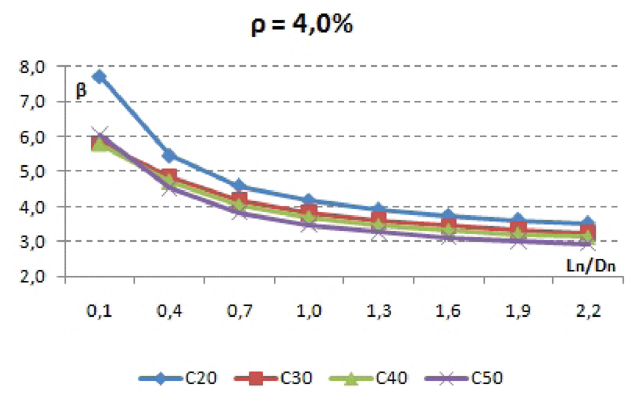
Figure 4 – Reliability index results for real concretes in terms of load ratio L_n/D_n and concrete class



a) Results for $\rho = \rho_{min}$

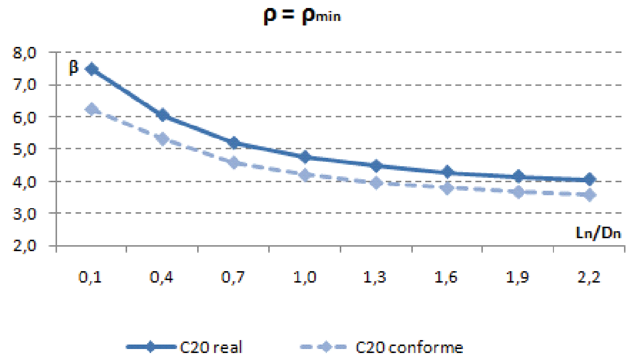


b) Results for $\rho = 2{,}0\%$

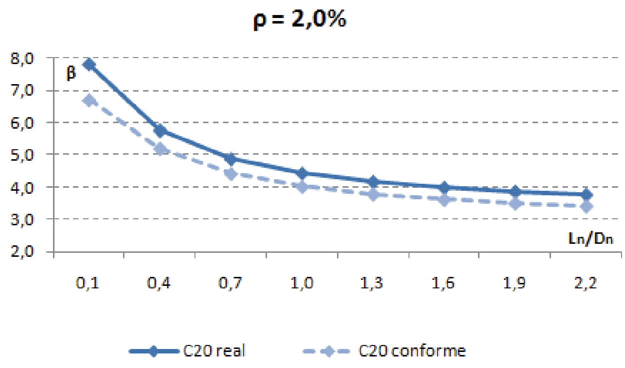


c) Results for $\rho = \rho_{máx}$

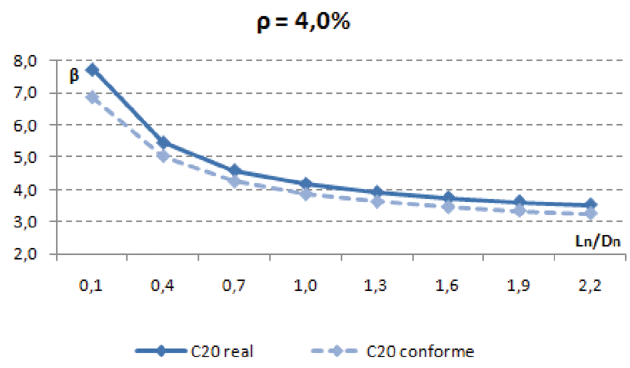
Figure 5 – Reliability index results for real and compliant C20 concretes in terms of load ratio L_n/D_n



a) Results for $\rho = \rho_{min}$



b) Results for $\rho = 2{,}0\%$



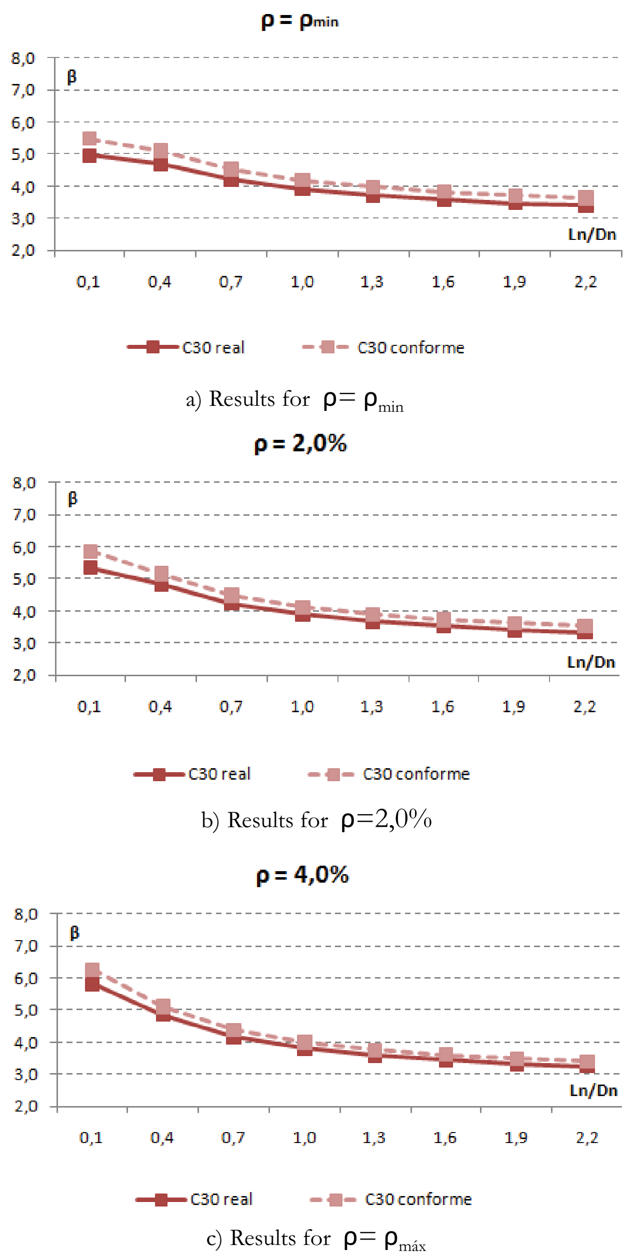
c) Results for $\rho = \rho_{máx}$

7. Reliability analysis results

Reliability indexes obtained for short columns with different reinforcement ratios and for four strength classes are presented in

Figure 4. Results show that reliability indexes decrease as load ratios are increased. This behavior is well-known, and is known to be a consequence of using constant partial safety factors for loads (g_D and g_L), regardless of load ratios. Since the coefficient of variation (c.v.) of live load (0,25) is much larger than the c.v. of dead load (0,10), as the live load increases proportionally to the dead load, contribution of live load increases and reliability indexes are reduced.

Figura 6 – Reliability index results for real and compliant C30 concretes in terms of load ratio L_n/D_n



a) Results for $\rho = \rho_{min}$

b) Results for $\rho = 2{,}0\%$

c) Results for $\rho = \rho_{máx}$

In Figure 4, the distance between curves reflects the effect of concrete strength class on reliability indexes. The Figure shows that, for load ratios $L_n/D_n > 0{,}4$ reliability indexes decrease slightly as concrete strength increases, especially for lower reinforcement ratios. This result may appear contradictory, but it is a consequence of designing the admissible load for a predefined column cross-section, and also of the dominating role of live load L (as will be shown in the sequence). This is also a consequence of concrete strength non-compliancy, which is aggravated for concretes of greater nominal resistances. For load ratios $L_n/D_n < 0{,}4$, concrete strength is the dominating random variable, and the observed behavior is inverted: reliability indexes increase as concrete strength increases. Only the (real) C20 concrete does not follow this tendency, because it is "better than a strictly compliant concrete", as seen in Table 2.

Figure 5 illustrates reliability indexes obtained for C20 concretes with different reinforcement ratios. Two sets of results are presented in this Figure. The dotted line shows results for strictly compliant concretes, whereas the continuous line shows results for the real, "more than strictly compliant concretes". The Figure makes evident that reliability of C20 concretes short columns is not affected by concrete non-compliancy. Actually, C20 concretes produced in Brazil are, following results of our database, better than strictly compliant. This increases reliability indexes for the real C20 concretes.

Reliability indexes for C30 concretes and different load ratios are shown in Figure 6. In this figure, it can be observed that concrete strength non-compliancy affects column reliability in a moderate way, As expected, effects of non-compliancy are larger for low load ratios, when concrete strength plays a more significant role in column reliability.

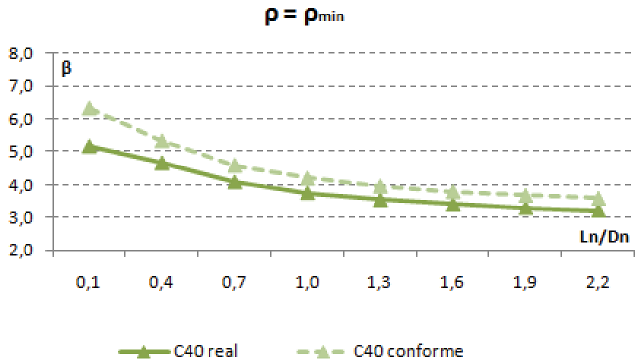
Reliability indexes obtained for C40 concretes and different load ratios can be observed in Figure 7. It is noted that for C40 concretes the problem of non-compliancy has a greater effect in the reduction of reliability indexes. The reduction is quite significant, as reliability indexes are dropped below target reliability levels. Following the EUROCODE, reliability index for columns should be at least β=3.8. In Figure 7 it can be observed that, for compliant concretes, this value is always achieved for $L_n/D_n < 1{,}0$. However, for the non-compliant concretes delivered at Brazilian construction sites, reliability indexes drop below this target level. Hence, the effect of concrete non-compliancy is quite significant for C40 concretes.

Figure 8 illustrates similar results for C50 concretes. Non-compliancy affects reliability indexes in an even more significant way for these concretes.

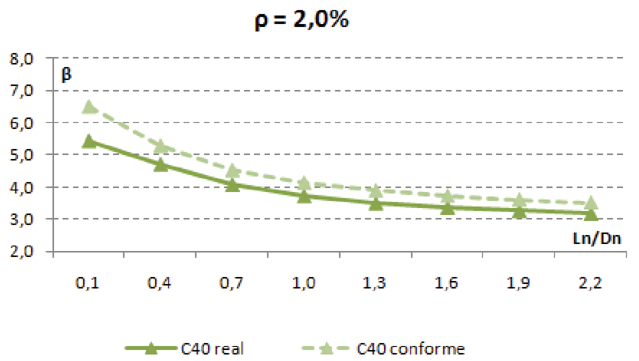
In general terms, it is possible to note in Figures 6, 7 and 8 that, as concrete resistance increases, the distance between dotted and continuous lines increases, revealing the effect of non-compliancy increases as concrete class increases. This is a result of the increasing percentiles of non-compliant concretes observed in the studied database, as concrete characteristic strength increases. In a similar way, it is observed that as the reinforcement ratio is reduced, effects of non-compliancy increase, as the contribution of concrete strength to column strength increases. Finally, it is observed that the largest effects of non-compliancy occur for regions of small load ratios ($L_n/D_n < 0{,}4$), where the importance of concrete strength is larger.

Sensitivity coefficients of the problems random variables are shown in Figures 9, 10 and 11, in terms of load ratios. Sensitivity coefficients are the direction cosines of the geometrical reliability index, and they show the relative contribution of each random variable towards the failure probability. Load variables (L and D) appear as negative coefficients, whereas resistance variables (f_c, f_y, C) appear as positive coefficients. Figures 9, 10 and 11 show that uncertainty in the live load plays an increasing dominating role as the load ratio L_n/D_n increases, which should be expected. Less evident is the increasing importance of live load when reinforcement ratios increase. Concrete strength, is the second most important random variable, especially for columns with low reinforcement ratio and executed with lower strength concretes. As reinforcement ratios increase, the contribution of concrete strength is reduced, but the importance of live load increases. Concrete strength becomes the most important random variable when load ratios are small (L_n/D_n<0,4).

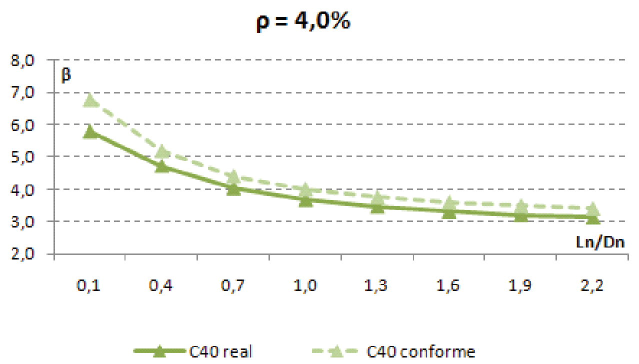
Figure 7 – Reliability index results for real and compliant C40 concretes in terms of load ratio L_n/D_n



a) Results for $\rho = \rho_{min}$

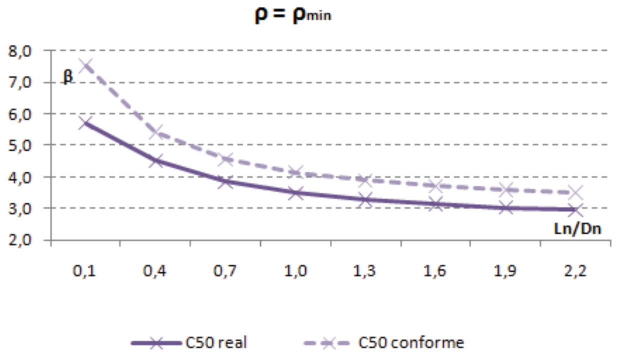


b) Results for $\rho = 2,0\%$

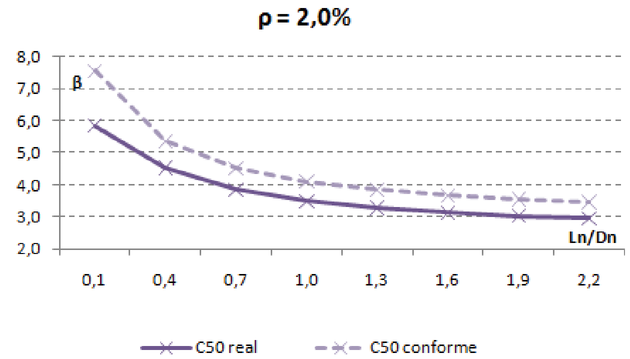


c) Results for $\rho = \rho_{máx}$

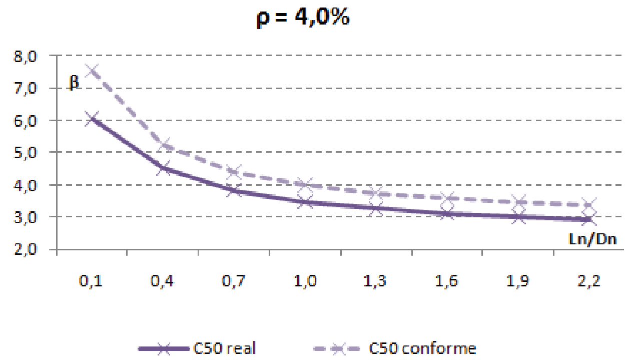
Figure 8 – Reliability index results for real and compliant C50 concretes in terms of load ratio L_n/D_n



a) Results for $\rho = \rho_{min}$



b) Results for $\rho = 2,0\%$



c) Results for $\rho = \rho_{máx}$

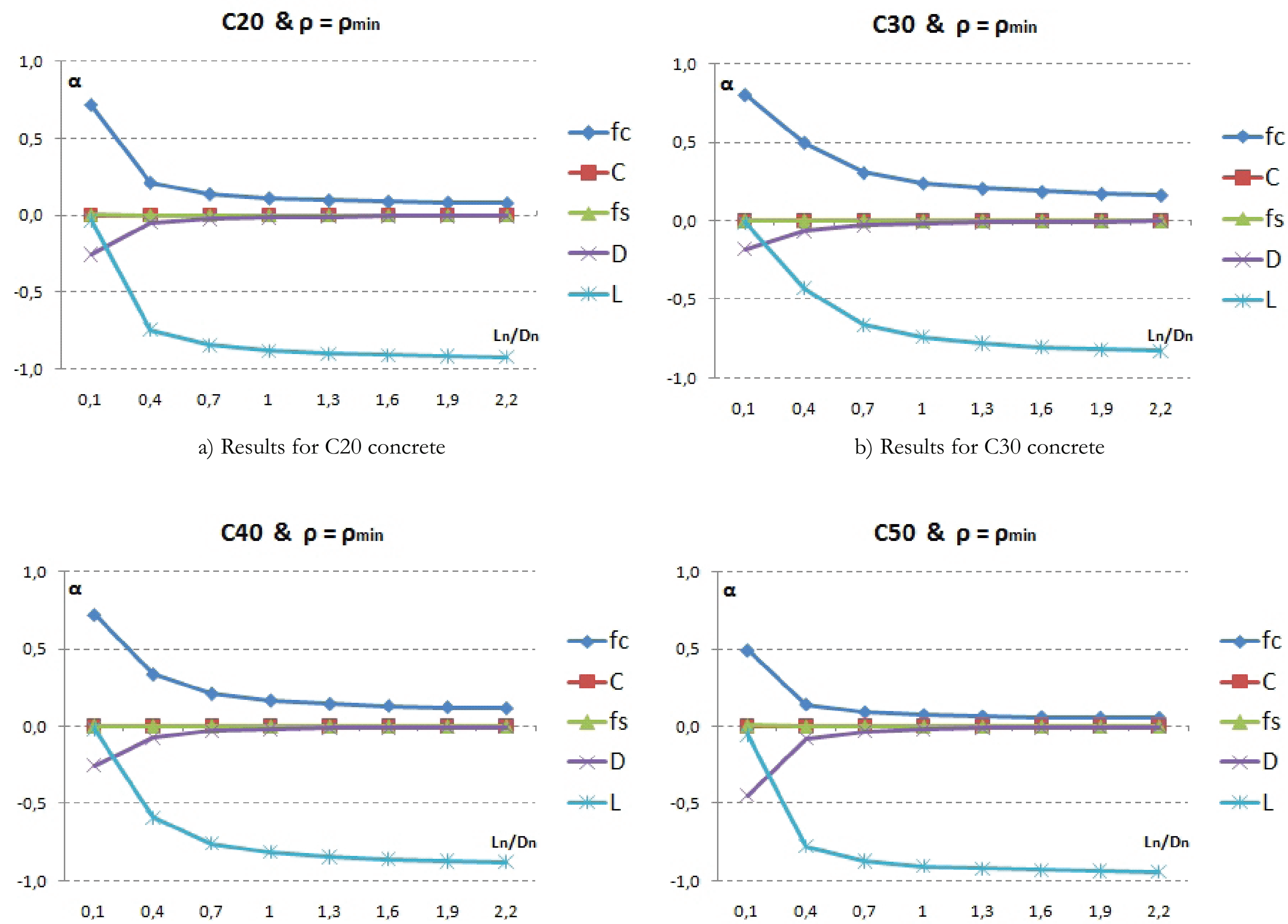
8. Concluding remarks

This article presented an investigation of the strength of plant concrete produced in Brazil, covering classes C20, C30, C40 and C50, and based on a database of over five thousand specimens molded in loco upon reception. It was verified that a significant part of these concretes to not reach the characteristic strength specified in design, and hence should be considered non-compliant. Non-compliancy increases as the class of concrete resistance increases. For C40 concretes, mean resistance of over three thousand specimens was found to be slightly above the specified characteristic strength (f_{ck}=40 MPa specified; f_{cm}=41,7 MPa and f_{ckest}=35,9 MPa, obtained experimentally). Results presented herein indicate that, for higher strength concretes, plants have been centering the reference mixture strength on the characteristic strength, and not on the mean strength, as required [8]. Such a practice should be strongly opposed by constructors and designers, who pay the highest losses resulting from concrete non-compliance.

For the C40 concrete (largest experimental dataset), the ratio of delivered (f_{ckest}) to specified f_{ck} was found to be around 0.9. In terms of design, in the absence of mitigation measures, this would be equivalent to designing reinforced concrete structures using a partial safety factor of γ_c=1.4·0.9=1.26.

The investigation also showed how the non-compliance of concretes produced in Brazil would affect the safety of reinforced concrete structures, if mitigation measures would not be adopted. The study covered an ample range of design situations, including three reinforcement ratios, eight live-to-dead load ratios and four concrete strength classes. It was found that C20 concretes are actually better than strictly compliant. For C30 concretes, a moderate reduction of reliability indexes was observed. For C40 and C50 concretes, a significant reduction was observed. For customary load ratios (L_n/D_n<1,0), reliability indexes for compliant concretes are always above the recommended value for columns (β=3.8). For the real concretes, reliability levels fall significantly below this level. Hence, the detrimental effect on column safety is not acceptable. Reduction is reliability levels was found to be more drastic for lower reinforcement rations and small load ratios.

Figure 9 – Sensitivity coefficients for $\rho = \rho_{min}$ in terms of load ratio L_n/D_n



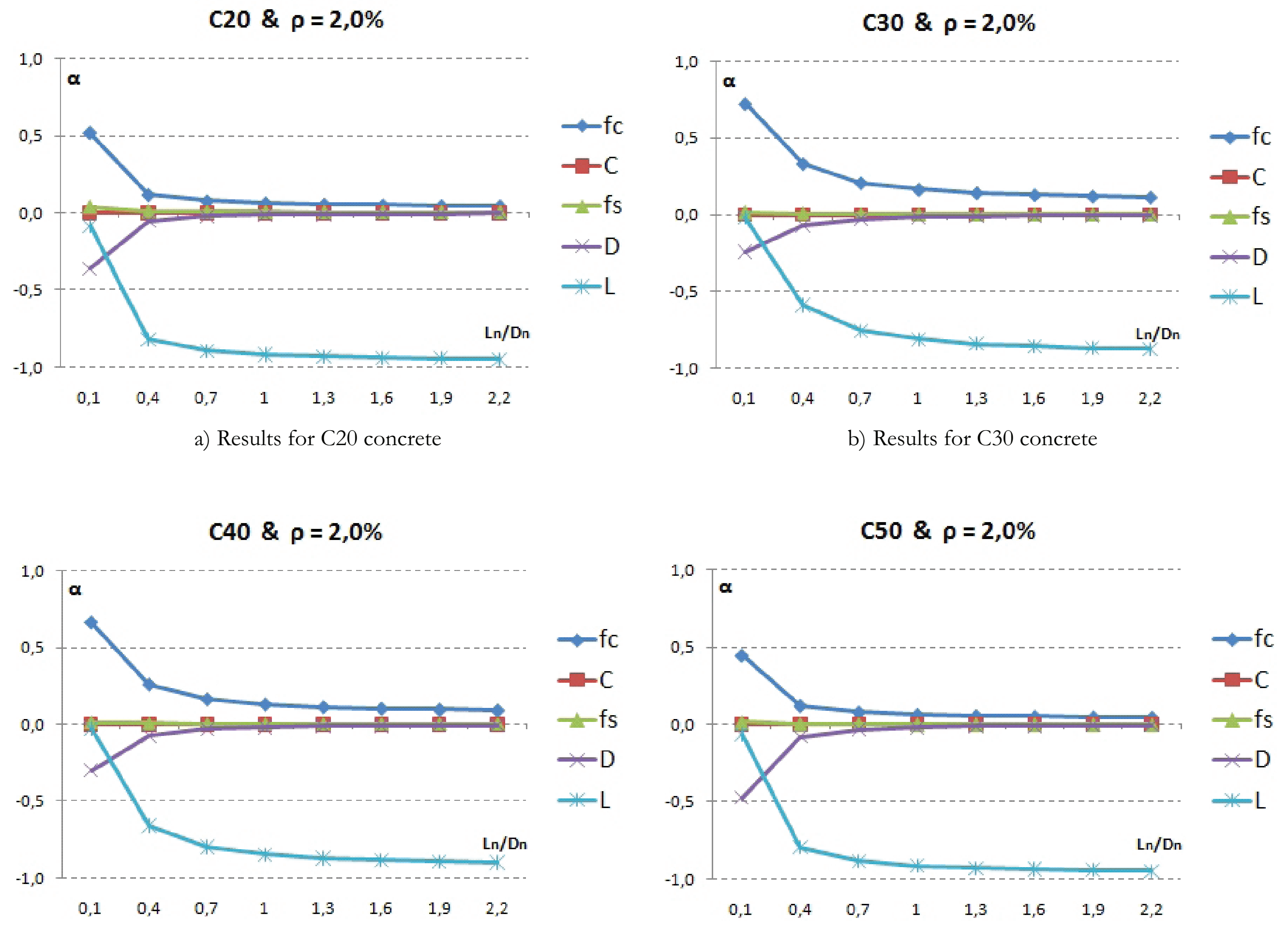
a) Results for C20 concrete

b) Results for C30 concrete

c) Results for C40 concrete

d) Results for C50 concrete

Figure 10 – Sensitivity coefficients for $\rho = 2{,}0\%$ in terms of load ratio L_n/D_n



a) Results for C20 concrete

b) Results for C30 concrete

c) Results for C40 concrete

d) Results for C50 concrete

The authors believe that there is ample space for improvements of structural design codes in Brazil. However, changes can only be proposed if the current codes of practice are strictly followed. In particular, the research group of the authors is working on a long range project whose objective is to perform the reliability-based calibration of partial safety factors for Brazilian structural design codes [4]. The study of concrete strength compliance is fundamental to achieve this objective.

Results presented herein illustrate aspects of the problem of concrete strength non-compliancy in Brazil. Since the study has not addressed particular buildings, effects of reinforcement, load limitations and reconstruction could not be considered. The investigation showed that the safety of reinforced concrete structures in Brazil would be significantly affected if measures of reinforcement, load limitations or reconstruction where not adopted in a particular construction where non-compliancy of concrete strength is identified.

9. Acknowledgment

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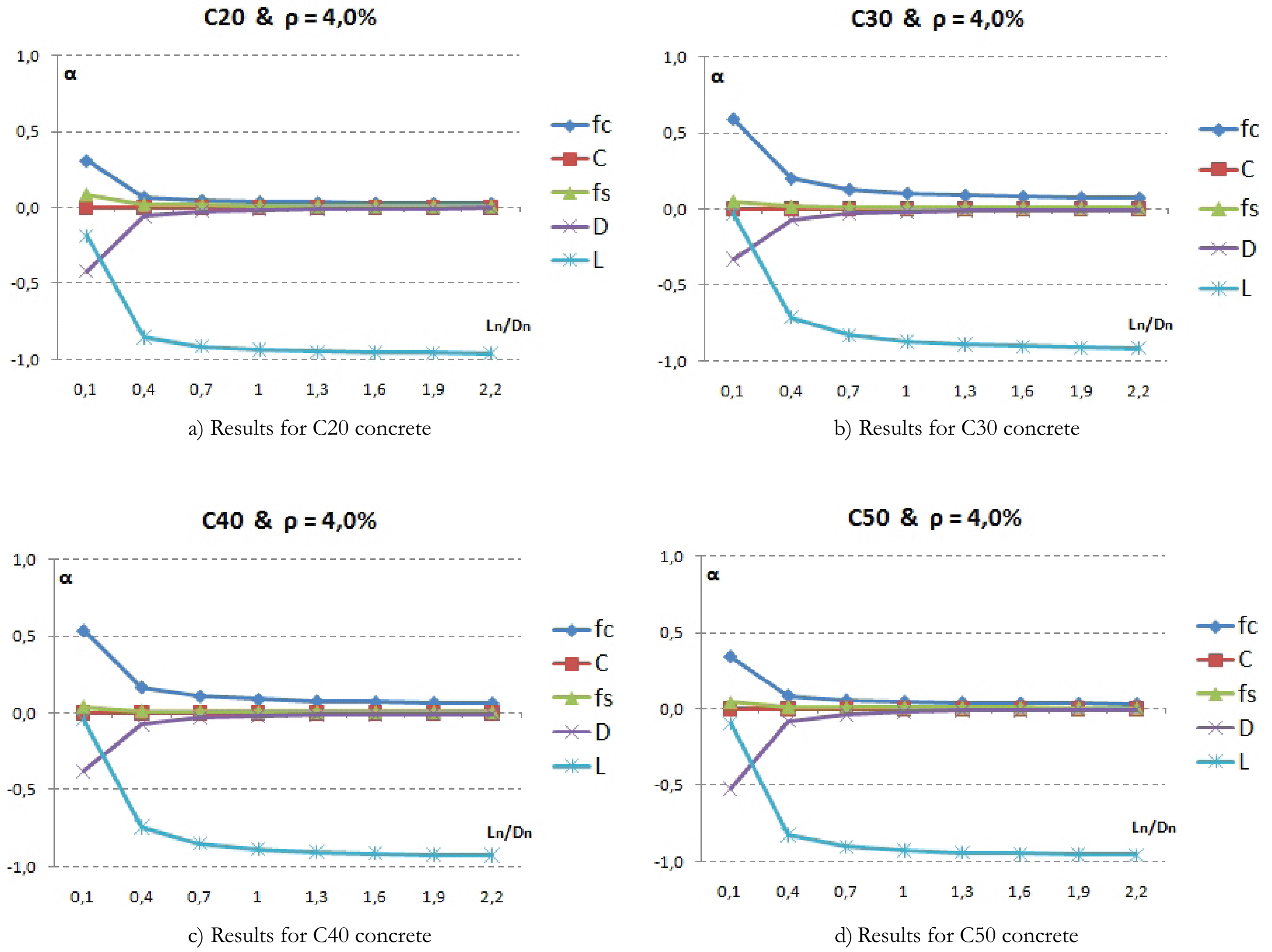
Figure 11 – Sensitivity coefficients for $\rho = \rho_{máx}$ in terms of load ratio L_n/D_n



a) Results for C20 concrete

b) Results for C30 concrete

c) Results for C40 concrete

d) Results for C50 concrete

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