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ORIGINAL ARTICLE

Unreinforced concrete masonry under compression: Brazilian statistics, reliability analysis and code calibration

Alvenarias de concreto não-armadas sob compressão: estatísticas brasileiras, análises de confiabilidade e calibração de norma

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Abstract: Although the new Brazilian code is considered an advancement, it still lacks a reliability-based calibration of partial safety factors. Therefore, this paper presents a comprehensive set of reliability analyses oriented to structural masonry members under compression in Brazil. This study was based on structural reliability theory, which allowed a safety assessment through the reliability index (β). Walls with hollow concrete blocks of three classes (A, B and C) and two modular sizes (M-15 and M-20), two ratios between the prism strength and the block compressive strength (η = 0.5 and 0.6), and three ratios between effective height and effective width (λ = 14, 19 and 24) were evaluated. Considering that each one of the 36 elements is described by five ratios between live and dead loads and the five ratios between wind and dead loads, 900 configurations were analyzed. Reliability procedures, the structural configurations and the calibration methodology were implemented and solved in the StRAnD – Structural Reliability Analysis and Design software. Results showed that the reliability indexes are greater for unreinforced masonry walls under compression with modular size M-20 than modular size M-15. It can be noted that the block compressive strength not only represents the greatest impact on the masonry's safety level, but is also the most relevant random variable. Based on the reliability-based calibration of the Brazilian code, it is recommended that unreinforced masonry presents different partial safety factors (γ_m) than reinforced masonry. This work represents a stepping stone in evaluating the safety of Brazilian masonry structures, indicating a possible path in terms of achieving a more uniform reliability in masonry design.

Keywords: reliability, safety, structures, masonry, concrete blocks.

Resumo: Embora a nova norma brasileira seja considerada um avanço, ainda carece de uma calibração baseada em confiabilidade dos fatores parciais de segurança. Portanto, este artigo apresenta um conjunto abrangente de análises de confiabilidade voltadas para membros estruturais de alvenaria sob compressão no Brasil. Este estudo foi baseado na teoria da confiabilidade estrutural, que permitiu avaliar a segurança através do índice de confiabilidade (β). Paredes com blocos vazados de concreto de três classes (A, B e C) e duas famílias modulares (M-15 e M-20), duas relações entre a resistência do prisma e a resistência à compressão do bloco (η = 0,5 e 0,6), e três razões entre altura efetiva e largura efetiva (λ = 14, 19 e 24). Considerando que cada um dos 36 elementos é descrito por cinco relações entre cargas móveis e permanentes e por cinco relações entre vento e cargas permanentes, foram analisadas 900 configurações. Os procedimentos de confiabilidade, configurações estruturais e metodologia de calibração foram implementados e resolvidos no software StRAnD – Structural Reliability Analysis and Design. Os resultados mostraram que os índices de confiabilidade são maiores para paredes de alvenaria não armada sob compressão com tamanho modular M-20 do que com tamanho modular M-15. Pode-se notar que a resistência à compressão do bloco não só representa o maior impacto no nível de segurança da alvenaria, mas também é a principal fonte de incerteza. Com base na calibração baseada em confiabilidade da norma brasileira, recomenda-se que a alvenaria não armada apresente fatores parciais de segurança (γ_m) diferentes da alvenaria armada. Este trabalho representa um marco inicial

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Data Availability: The full dataset that supports the findings of this study is available in [17]-[24].

na avaliação de segurança de estruturas de alvenaria brasileira, indicando um caminho para alcançar uma confiabilidade mais uniforme em projetos de alvenaria.

Palavras-chave: confiabilidade, segurança, estruturas, alvenaria, blocos de concreto.

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1 INTRODUCTION

1.1 Background

The first structural masonry building in Brazil was built in 1966 with hollow concrete blocks. The project's success and the enhancement of structural block fabrication techniques enabled the dissemination of structural masonry in the following years.

During the 1980s the structural masonry reached its peak in the country, driven by the construction of housing projects for low-income families. The deficiency in the Brazilian code that guided the design of masonry resulted in pathological manifestations in many buildings, which reflected in a deceleration in the use of this constructive system.

However, construction companies wagered on the economic advantages of structural masonry and its return was accompanied by a technical committee focused on updating the code concepts. This effort culminated in the development of the Brazilian code NBR 15961-1:2011 [1], which was recently replaced by the NBR 16868-1:2020 [2].

Although the new code was considered an advancement, it still did not incorporate a reliability-based calibration of partial safety factors. To aggravate the scenario, the Brazilian code NBR 8681:2003 [3] has not been revised to consider the specificities of structural masonry. This scenario reinforces the relevance of reliability analysis that demonstrates the safety of structural masonry designed and built in Brazil.

In this context, this paper aims to study the safety of unreinforced concrete masonry members under compression. The work is based on structural reliability theory, which allows a safety assessment through the reliability index (β).

Walls with hollow concrete blocks of three classes (A, B and C) and two modular sizes (M-15 and M-20), two ratios between the prism strength and the block compressive strength (η = 0.5 and 0.6), and three ratios between effective height and effective width (λ = 14, 19 and 24) are evaluated.

The study also considered five ratios between live and dead loads (L_n/D_n= 0.0, 0.5, 1.0, 1.5 and 2.0), and five ratios between wind and dead loads (W_n/D_n= 0.0, 0.5, 1.0, 1.5 and 2.0). In total, 900 configurations are analyzed (36 elements times 25 load ratios).

Based on the current literature, this paper presents a comprehensive set of reliability analyses oriented to structural masonry problems in Brazil, even though there are still unexplored areas for future research (reinforced masonry under compression, masonry beam with and without reinforcement, etc.).

The paper is structured as follows: material and methods are described in Section 2, results and discussions are presented in Section 3 and concluding remarks in Section 4.

1.2 Literature review

Due to the uncertainties presented in a structural project, it is common to notice discrepancies between the real behavior of a structure and its theoretical one. Thus, the main purpose of a reliability analysis is to quantify the safety of an engineering problem from the perspective of the structure's intrinsic uncertainties.

Turkstra and Ojinaga [4] conducted a series of reliability analyses on structural masonry to improve the Canadian masonry design code. However, the suggestions were not adopted at that time.

A reliability evaluation of masonry structures for the United States of America was performed by Ellingwood [5]. Test data for masonry walls under axial compression were used in the study.

Turkstra [6] expanded those studies and reviewed some aspects of limit states design for masonry structures. As a result, new partial safety factors for the Canadian code were established for masonry design.

Statistical and probabilistic analyses of masonry structures in Australia were conducted by Lawrence [7]. The results represented one of the first initiatives on the safety assessment of Australian masonry.

Drake et al. [8] continued the studies related to masonry structures in Australia and developed preliminary techniques to estimate the reliability of masonry walls under compression loading.

In the study conducted by Stewart and Lawrence [9], the structural reliability of masonry in compression was calculated based on wall test data and probabilistic models for material properties and loads in Australia.

Zhai and Stewart [10] developed the first probabilistic model to calculate the structural reliability of masonry walls under compression designed in accordance with applicable Chinese codes.

A structural reliability analysis was performed on concrete masonry under compression by Kazemi et al. [11]. Over 300 masonry compressive strength values were considered from North American investigations, mostly Canadian.

Moosavi and Korany [12] evaluated the reliability of masonry structures under axial compression to assess the safety level of the current and previous editions of the Canadian masonry design code.

A preliminary reliability investigation about masonry walls with ceramic blocks produced in the Brazilian northeast region was performed by Souza et al. [13]. Experimental measurements and tests provided the necessary data.

Zięba and Skrzypczak [14] presented the reliability of compressive masonry structures designed with the recommendations of the Eurocodes. In this work, the walls were made of silicate blocks.

Either in Brazil or elsewhere, few works conducted into the reliability analysis of masonry structures have been reported. To the author's best knowledge, there is no record of any similar survey carried out in Brazilian masonry such as the one proposed in this paper.

Among the most popular methods used in the reliability analyses of masonry structures, stands out the First Order Reliability Method – FORM, Hasofer and Lind [15]. Although classic, this method proved to be adequate, due to its processing speed and accuracy when dealing with problems that present few non-linearities.

The starting point of the FORM method is constructing a joint probability distribution based on the marginal distributions involved in the problem. This method also requires the transformation of random variables vector X, with any distribution, in a set Y of normal random variables with null mean and a standard deviation of one, Melchers and Beck [16].

The joint probability distribution in standard Gaussian space, also called multivariate or multidimensional standard Gaussian distribution, allows finding the reliability index that corresponds to the point on the failure domain with the highest probability of occurrence.

Therefore, the solution is usually reached by solving an optimization problem via numerical methods based on algorithms such as Hasofer, Lind, Racksitz and Fiessler (HLRF), since the reliability index corresponds to the distance between the design point and the origin.

FORM also highlights the relative importance of each random variables that integrate the problem through the sensitivity factors (α). These factors, defined as directional cosines in the hyperplane, correspond to the ratio between the gradient vector and its modulus.

2 MATERIAL AND METHODS

2.1 Brazilian statistics

This item presents the statistical properties of the main random variables of strength and load related to Brazilian unreinforced concrete masonry members under compression. The probability distributions were fitted and the random variable not available in the literature passed goodness-of-fit tests - Chi-Square, Kolmogorov-Smirnov and Anderson-Darling - after the exclusion of spurious results.

2.1.1 Strength variables

Three strength variables associated with unreinforced concrete masonry walls under compression built in Brazil were collected: hollow concrete block compressive strength (f_b), thickness (t) and resistance model uncertainties (θ_R), as shown in Table 1.

Random variables f_b and t were compiled from results of axial compression tests and dimensional analysis performed in more than six thousand concrete blocks manufactured between 2011 and 2016 in Brazil.

The blocks were divided into three classes (A, B and C) and two modular sizes (M-15 and M-20). Class A blocks had characteristic compressive strength greater than 6 MPa, while class B and class C blocks had characteristic compressive strength greater than 4 and 3 MPa, respectively. Modular size M-15 blocks had a width of 14 cm, while modular size M-20 blocks measured 19 cm.

The dataset was built from 14 manufacturers: six from the Northeast region, five from the Southeast region and three from the South region. Only four manufacturers were not certified by the ABCP (Brazilian Association of Portland Cement) quality program. Further details are given in Santiago and Beck [17], and Almeida et al. [18].

Table 1. Brazilian strength random variables.

Random Variable			Distribution	Mean	COV	References
f_b	A	M-15	Normal	$1.358.f_{bk}$	0.239	Santiago and Beck [17] and Almeida et al. [18]
		M-20	Normal	$1.364.f_{bk}$	0.197	
	B	M-15	Normal	$1.592.f_{bk}$	0.279	
		M-20	Normal	$1.328.f_{bk}$	0.248	
	C	M-15	Normal	$1.566.f_{bk}$	0.294	
		M-20	Normal	$1.419.f_{bk}$	0.221	
t	A	M-15	Normal	$1.003.t_n$	0.005	Santiago and Beck [17] and Almeida et al. [18]
		M-20	Normal	$1.002.t_n$	0.003	
	B	M-15	Normal	$1.003.t_n$	0.025	
		M-20	Normal	$1.002.t_n$	0.010	
	C	M-15	Normal	$1.005.t_n$	0.021	
		M-20	Normal	$1.001.t_n$	0.003	
θ_R			Normal	1.940	0.137	This work

The random variable θ_R, which expresses the difference between the experimental strength of masonry elements under compression ($f_{m,exp}$) and the predicted value by the design model prescribed on NBR 16868-1:2020 [2] ($f_{m,the}$), was derived from axial compression tests results, executed in concrete masonry members.

Tests performed on approximately four dozen walls with lengths above 99 cm were considered, as reported by Izquierdo [19], Lopes [20], Fortes et al. [21], Nunes [22], Lima [23] and Pinheiro [24].

The studies were conducted in Brazilian laboratories, with national manufactured blocks, varying in length and height, of unreinforced hollow concrete masonry walls. The accuracy of the wall resistance predictive model used in Brazil is not precise, hence the importance of considering studies oriented to national reality.

As it can be noted, Normal and Lognormal probability distributions fitted well to the data, possibly due to the relatively small sample size. Figure 1 shows box-and-whiskers representation for the ratios between experimental and theoretical results, in order to highlight the lower and upper boundaries used to exclude outliers.

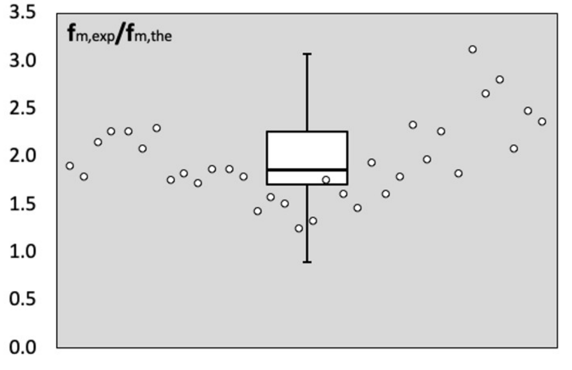


Figure 1. Box-and-whiskers representation for the samples.

For comparative purposes, Table 2 presents the distributions according to the literature. It can be inferred that Brazilian parameters are similar to the ones related to other countries.

The variation observed on bias and COV of θ_R is mainly due to the size of the database, the specificities of the designed model, the properties of the materials (block and mortar), the equipment and testing procedures, etc.

Stewart and Lawrence [9], for instance, considered tests performed on 87 concentrically loaded concrete walls, a predictive model based on a nominal characteristic compressive strength of the masonry units provided by the manufacturer, and a common mortar type.

Table 2. International strength random variables.

Random Variable	Distribution	Mean	COV	References
f_b	Normal	$1.41.f_{bk}$	0.17	Kezemi et al. [11]
	Normal	$1.46.f_{bk}$	0.205	Moosavi and Korany [12]
t	Normal	t_n	0.02	Stewart and Lawrence [25]
	Normal	t_n	0.01	Moosavi and Korany [12]
θ_R	Lognormal	2.18	0.20	Stewart and Lawrence [9]

More than 40 years ago, Ellingwood et al. [26] already stated that variations in these estimates, among individual sets of data, are naturally to be expected, which does not invalidate these values for reliability analyses.

It is important to notice that, in Tables 1 and 2, the means of the random variables were expressed as a function of their respective characteristic or nominal values (f_{bk} and t_n).

2.1.2 Load variables

Six load variables, related to general structures built in Brazil, were collected: dead load (D), arbitrary point in time live load (L_{apt}), 50-year extreme live load (L_{50}), annual extreme wind load (W_1) and 50-year extreme wind load (W_{50}) and load model uncertainties (θ_L) as shown in Table 3.

Table 3. Brazilian load random variables.

Random Variable	Distribution	Mean	COV	References
D	Normal	$1.06.D_n$	0.12	Santiago [27]
L_{apt}	Gamma	$0.21.L_n$	0.76	Costa et al. [28]
L_{50}	Gumbel	$0.92.L_n$	0.25	Costa et al. [28]
W_1	Gumbel	$0.33.W_n$	0.47	Beck and Souza [29]
W_{50}	Gumbel	$0.90.W_n$	0.34	Beck and Souza [29]
θ_L	Lognormal	1.00	0.05	JCSS [30]

The random variable D reflects the accuracy with which engineers estimate the self-weight of structural and non-structural elements. It was based on results sent by different structural engineers that determined the weight of the same building based on the return given by the construction companies that hired them, as stated in Santiago [27].

Random variables L_{apt} and L_{50} reflect the building occupancy as a random process in time. Both of them were obtained from a stochastic model proposed in the JCSS [30] properly adapted to the Brazilian reality. The main point of the adaptation was the decomposition of live loads in two parcels, with different behaviors regarding their temporal variability. Details of the simulation procedure are presented in Costa et al. [28].

The random variables W_1 and W_{50} reflect the wind effect on structures as a random process over time. Both variables were based on wind speed data obtained in meteorological stations across the country, as reported by Beck and Souza [29].

Random variable θ_L, which expressed the load uncertainties in the structure, was based on definitions of the JCSS [30].

It is important to emphasize that, in Table 3, the means of those random variables were expressed as a function of their respective nominal values (D_n, L_n and W_n).

2.2 Reliability analysis and code calibration

This item presents the reliability procedures, the structural configurations and the calibration methodology. The study was implemented and solved in the StRAnD – Structural Reliability Analysis and Design software, developed by the research group of Professor Beck [31].

2.2.1 Reliability procedures

Reliability analysis was performed by limit state functions derived from the combination equation proposed by the Brazilian code NBR 8681:2003 [3] and Turkstra [32], as presented in Equation 1.

$$\begin{cases} g_1(X) = R(f_b, t, \theta_R) - L(D, L_{50}, W_1, \theta_L) = 0 \\ g_2(X) = R(f_b, t, \theta_R) - L\big(D, W_{50}, L_{apt}, \theta_L\big) = 0 \end{cases} \tag{1}$$

It is important to mention that the failure probability and the reliability index, for each limit state, were obtained according to Equation 2.

$$p_f = \int_{g(x)\leq 0} f_X(x)dx \approx \Phi(-\beta) \tag{2}$$

where $g(x) \leq 0$ is the failure domain, $f_X(x)$ is the joint probability distribution of the random variable vector $X = \{f_b, t, \theta_R, D, L_{apt}, L_{50}, W_1, W_{50}, \theta_L\}$, and β is the distance between the design point and the origin. It should be noted that $f_X(x)$ was obtained as the product of marginal distributions, once the random variables were assumed independent.

Since the limit state functions proved to be linear for these variables, and the marginal probability distributions were not excessively non-Gaussian, accurate estimates of the failure probability were obtained using FORM.

This method maps Equation 1 to the standard Gaussian space, solves an optimization problem to find the design point, and linearizes the limit state function at the design point. FORM also provided the sensitivity factors α, as sub-products of the solution, given by Equation 3.

$$\alpha = \{\alpha_i\}_{i=1,\ldots,n}^{t} = \frac{\nabla g_u(u^*)}{\|\nabla g_u(u^*)\|} \tag{3}$$

where n is the number of random variables, u^* is the design point, and ∇g_u is the gradient of the limit state function in standard Gaussian space. The sensitivity factor shows the relative contribution of each random variable towards the failure probability.

2.2.2 Structural configurations

This paper considered unreinforced masonry under compression with hollow concrete blocks of three classes and two modular sizes, two ratios between the prism strength and the block compressive strength (η), and three ratios between effective height and effective thickness (λ). A subtotal of 36 elements was analyzed.

To measure the influence of concrete masonry units, three classes (A, B and C) and two modular sizes (M-15 and M-20) were adopted. Characteristic compressive strength equals to 6, 4 and 3 MPa were considered for the blocks of classes A, B and C, respectively.

To evaluate the mortar's impact, two ratios between the prism strength and the block compressive strength (η = 0.5 and 0.6) were contemplated. It is worth mentioning that the compressive strength of masonry depends, among other things, on the mortar's strength.

Aiming to investigate the slenderness effect, three ratios between effective height and effective thickness (λ = 14, 19 and 24) were examined. It should be noted that the slenderness ratio affects the capacity of masonry to resist compressive loading.

The limit state functions used in the reliability analysis proposed in this study, and related to unreinforced concrete masonry members under compression, are expressed in Equation 4.

$$\begin{cases} g_1(X) = \theta_R \cdot \left\{0.7 \,.\, \eta.\, f_b.\, l.\, t. \left[1 - \left(\frac{\lambda}{40}\right)^3\right]\right\} - [\theta_L \cdot (D + L_{50} + W_1)] = 0 \\ g_2(X) = \theta_R \cdot \left\{0.7 \,.\, \eta.\, f_b.\, l.\, t. \left[1 - \left(\frac{\lambda}{40}\right)^3\right]\right\} - \left[\theta_L \cdot \left(D + W_{50} + L_{apt}\right)\right] = 0 \end{cases} \tag{4}$$

where l is the length, assumed 200 cm. Since the two limit state equations represented loading cases for the same failure mode of the structural member, only the smallest reliability index was considered.

Based on predefined cross-sectional properties, the design strength (R_D) was evaluated according to the partial safety factor γ_m. Consequently, the nominal dead load (D_n) was evaluated by Equation 5.

$$D_n = \frac{R_D(\gamma_m)}{\max\begin{bmatrix}\gamma_D+\gamma_L.(L_n/D_n)+\gamma_W.\psi_W(W_n/D_n)\\ \gamma_D+\gamma_W.(W_n/D_n)+\gamma_L.\psi_L.(L_n/D_n)\end{bmatrix}} \tag{5}$$

where γ_D is the partial safety factor for dead load; γ_L and γ_W are partial safety factors for the main variable load, and ψ_q and ψ_W are load combination factors for the secondary variable load in the combination.

With D_n evaluated, L_n and W_n were obtained from the specified load ratios on Equation 6. From the data in Table 3, the probability distributions of the loads were readily evaluated.

$$\begin{aligned} L_n/D_n &= \{0.0, 0.5, 1.0, 1.5, 2.0\}; \\ W_n/D_n &= \{0.0, 0.5, 1.0, 1.5, 2.0\}. \end{aligned} \tag{6}$$

The selection of this set was related to the likelihood of different load situations for masonry, as presented in Table 4 derived from Ellingwood et al. [26].

Table 4. Weights for different load rations.

L_n/D_n or W_n/D_n	Weight (%)
0.0	36
0.5	36
1.0	20
1.5	6
2.0	2
3.0	0
5.0	0

Reference: Ellingwood et al. [26].

2.2.2 Code calibration

The code calibration was the process of finding the partial safety factor γ_m that minimizes the variations of the reliability indexes of different structural configurations, with respect to the target reliability index (β_{target}).

The study was formulated as a typical reliability-based design optimization (RBDO) problem, which represents the intrinsic uncertainties as random variables through Equation 7.

find: γ_m
that minimizes:
$$D_f = \sum_{i=1}^{m}\sum_{j=1}^{n}\left\{\left[\beta_{target} - \min_k\left(\beta_{ijk}(\gamma_m)\right)\right]^2 . w_{ij}\right\} \tag{7}$$

where m and n are the load ratios considered, $\beta_{ijk}(\gamma_m)$ is the reliability index calculated for the load ratios ij, w_{ij} is the weight of each load ratio in the combination, according to the relative importance of that design case (Table 4), and k is the critical limit state among the cases considered.

The target reliability index was set as $\beta_{target} = \{3.5, 3.6, 3.7, 3.8, 3.9\}$, since it expresses distinct costs of increasing or decreasing the existing safety level for the same type of structural element and consequence of failure.

The optimization problem was solved through the Particle Swarm Optimization algorithm (PSO), which is a meta-heuristic algorithm able to identify the global minimum in non-convex design spaces as given by Kennedy and Eberhart [33].

The algorithm is based on the movement of particles in the multidimensional search space, with the speed and position of each particle in the swarm being updated iteratively. Table 5 presents the main PSO parameters used in the scheduling strategy.

It is important to highlight that each β_{target} resulted in a single value of γ_m after five iterations, meaning that thousands of reliability analyses were done.

Table 5. PSO parameters.

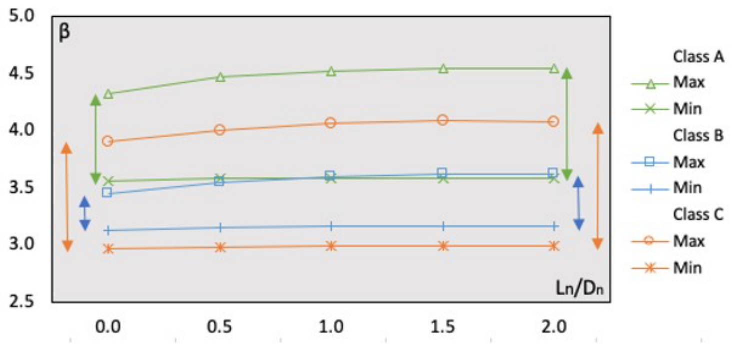
Parameter	Value
Population number (Np)	60.0
Inertia weight (Wi)	0.9
Cognitive learning factor (C1)	2.0
Social learning factor (C2)	2.0
Maximum velocity (Vmax)	1.0

3 RESULTS AND DISCUSSIONS

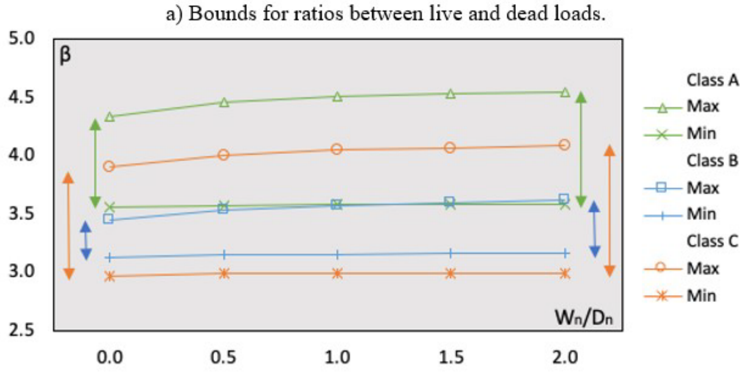
3.1 Reliability index (β)

Results obtained for concrete masonry under compression with partial safety factor $\gamma_m = 2.0$ currently indicated in NBR 16868-1:2020 [2] are presented in this section. The average reliability index that summarizes the results is 3.67.

Figure 2 illustrates the maximum and minimum reliability indexes obtained for all structural configurations, once the lines become too many when all results are combined. Tangible discrepancies between load ratios are not observed because the parameters of the load variables are similar, regardless of the combination.



a) Bounds for ratios between live and dead loads.



b) Bounds for ratios between wind and dead loads.

Figure 2. Reliability index envelope for unreinforced concrete masonry walls under compression.

The distance between curves in Figure 2 reflects the quality of concrete blocks for the three studied classes. Greater differences in the characteristics of the random variables f_b for both modular sizes – mainly the coefficient of variation – result in more distant curves. Figure 2 illustrates the envelope of βs, but hides areas without points due to the differences observed in modular-sized blocks' properties.

Figure 2 shows that the values for reliability indexes fall within 2.97 and 4.55, which are slightly smaller than those found by Stewart and Lawrence [25]: 3.6 to 5.0. This is due to the design model prescribed on NBR 16868-1:2020 [2] and also to additional consideration of wind loads. In general, the reliability indexes vary between 3.0 and 4.0 in civil structural problems according to Moosavi and Korany [12].

Even though there is a scarcity of publications about the reliability of Brazilian masonry, Souza et al. [13] found reliability indexes that varied from 2.58 to 5.56 for structural ceramic blocks produced in the state of Alagoas.

Considering other types of structures in Brazil, the reliability indexes found herein are greater than those found by Santiago et al. [34] for concrete columns under compressive loading: 2.22 to 3.99; and similar to those found by Oliveira et al. [35] for composite columns under compression: 2.4 to 4.6.

The reliability index envelopes obtained for unreinforced masonry under compression related to the modular size are presented in Figure 3. The reliability indexes found herein are greater for modular size M-20 blocks than modular size M-15 blocks. The range between both groups of curves in this figure reflects the quality of modular-sized concrete blocks.

The reliability indexes range from 3.48 to 4.55 for modular size M-20, while the reliability indexes fall between 2.97 and 3.79 for modular size M-15. CSA S408 [36] suggests reliability indexes between 2.8 and 4.5 for sudden failure. AS 5104 [37] indicates reliability indexes that vary from 3.1 up to 4.2 according to the safety measures. JCSS [30] proposes reliability indexes between 3.1 and 4.7, depending on the consequence of failure.

Although the safety is greater for unreinforced masonry walls under axial compression with modular size M-20 than modular size M-15, both groups present results in line with the literature. The findings emphasize the importance of a rigorous quality control during manufacturing and receiving processes of blocks to reduce discrepancies associated with modular size.

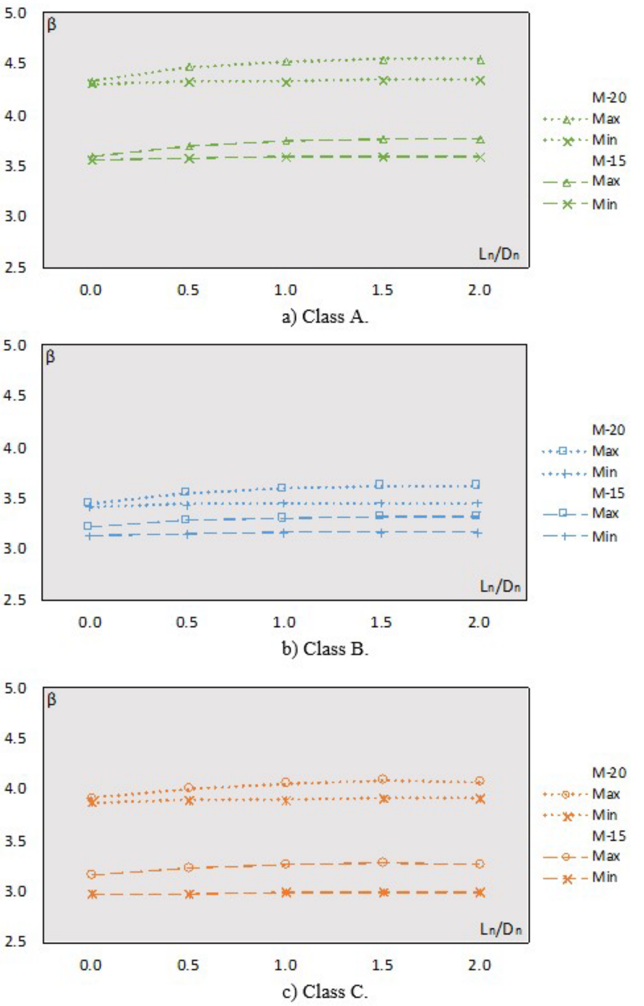


Figure 3. Reliability index envelope for unreinforced concrete masonry under compression according to the modular size.

Figure 4 exhibits the reliability indexes variation for unreinforced masonry under compression according to the ratios between the prism strength and the block compressive strength (η). In masonry with blocks of lower resistance classes, the safety tends to decrease with an increase in the prism resistance.

The reliability indexes are higher for unreinforced masonry walls under axial compression with η = 0.5 than η = 0.6, mainly in classes B and C. This result emphasizes the importance of a rigorous quality control in the production of mortar and in the execution of masonry.

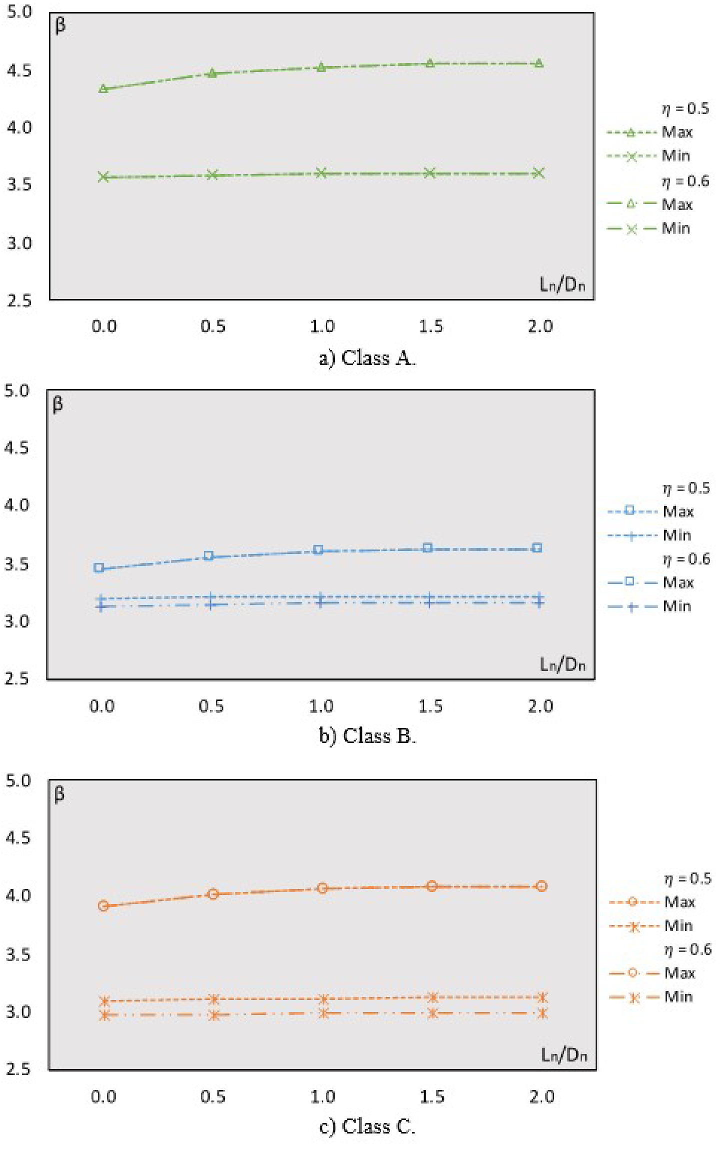


Figure 4. Reliability index envelope for unreinforced concrete masonry under compression conforming to the ratios between the prism strength and the block compressive strength (η).

The maximum and minimum reliability indexes calculated for unreinforced masonry under compression according to the ratios between effective height and effective thickness (λ) are presented in Figure 5. The reliability indexes found herein are contrary to expectations, since slenderness does not impact the problem.

The safety of unreinforced masonry walls under axial compression is not affected by the slenderness. Works such as Turkstra et al. [38] and Bartlett [39] also demonstrated that slenderness does not significantly change the reliability index.

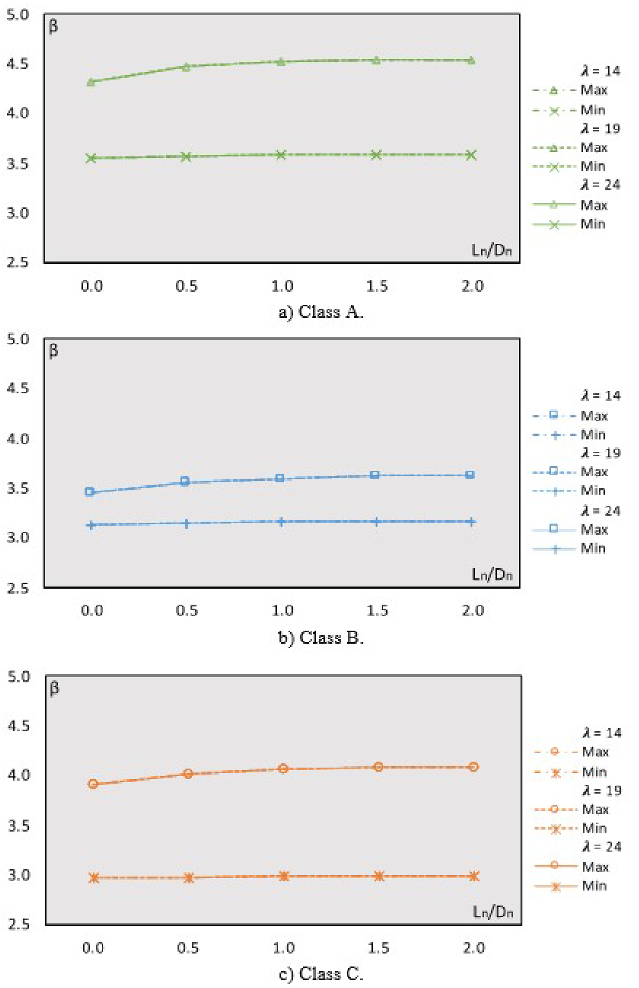


Figure 5. Reliability index envelope for unreinforced concrete masonry under compression according to the ratios between effective height and effective thickness (λ).

Load ratio had a minor influence on the reliability indexes for all the variations considered in the present study (group of modular size, ratio between the prism strength and the block compressive strength, and ratio between effective height and effective thickness), as it can be inferred by the small amplitude between curves. These results are in line with the findings of Allen [40] and Kazemi et al. [11].

3.2 Sensitivity factors (α)

Figure 6 exhibits sensitivity factors α of the random variables for unreinforced concrete masonry members under compression designed with class B Blocks, modular size M-15, the ratio between the prism strength and the block compressive strength of 0.6, ratio between effective height and effective width equal to 19, the ratio between live and dead loads of 1.0 and ψ_L equal to 0.5.

The observation of these factors, highlighted in Figure 6, is essential to interpret the results of reliability analysis. The concrete block compressive strength is the most relevant random variable, followed distantly by thickness and resistance model uncertainties; although live load (L_{50}) and wind load (W_{50}) also impact the results.

Although f_b presented a higher COV (0.197 to 0.279), θ_R COV value is also significant (0.137); therefore, their product has a combined effect that impacts the results. Perhaps the reliability problem can be converted into a single random variable problem without significant changes in results.

Even though Figure 6 represents a single point among the cloud of points between the curves in Figure 2, it still expresses a valid trend for all cases. The findings agree with the results described by Zięba and Skrzypczak [14], since it was observed that the greatest impact on the masonry’s safety level is the concrete block's compressive strength.

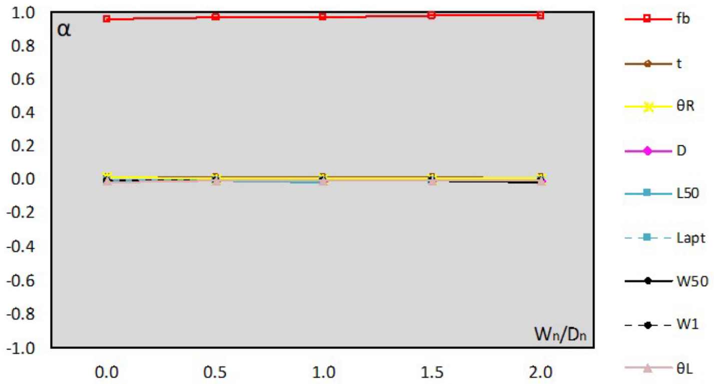


Figure 6. Sensitivity factors for unreinforced concrete masonry under compression.

3.3 partial safety factor (γ_m)

The capacity reduction factor γ_m calibrated for the studied set of β_{target} are presented in Figure 7. Under the current γ_m value of 2.0 proposed in NBR 16868-1:2020 [2], the target reliability index is equivalent to 3.67 for unreinforced concrete masonry subjected to axial compression. It is important to clarify that Figure 7 does not show the dispersion of reliability indexes, which illustrates a partial picture of the problem.

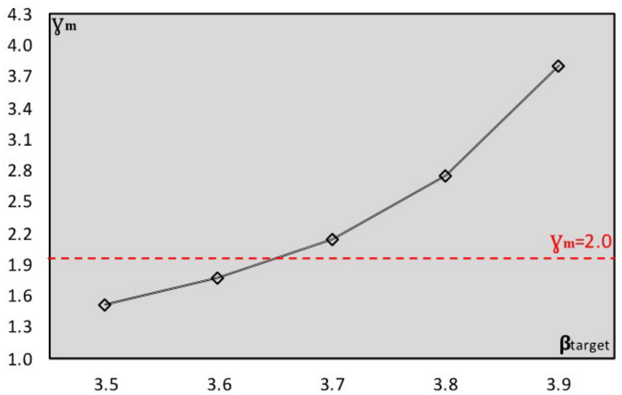


Figure 7. Partial safety factor for unreinforced concrete masonry under compression.

Moosavi and Korany [12] adopted $\beta_{target} = 3.9$ in their work, Stewart and Lawrence [25] assumed $\beta_{target} = 3.8$ in their research, Zhai and Stewart [10] considered $\beta_{target} = 3.7$ in their study, and Thamboo et al. [41] used $\beta_{target} = 3.6$ in their inquiry.

Considering that unreinforced masonry lacks ductility, it is reasonable a material factor different from reinforced masonry. Using different γ_m is common in many masonry standards as the Australian AS 3700:2018 [42] and the British PD 6697:2019 [43].

A target reliability index above 3.8 would have a significant economic impact on Brazilian masonry projects, but values between 3.7 and 3.8 may indicate a possible path in terms of achieving more safety.

It is also important to highlight that a higher value of γ_m would increase the level of safety of unreinforced concrete masonry under compression by reducing the dispersion of reliability indexes, as shown in Figure 8.

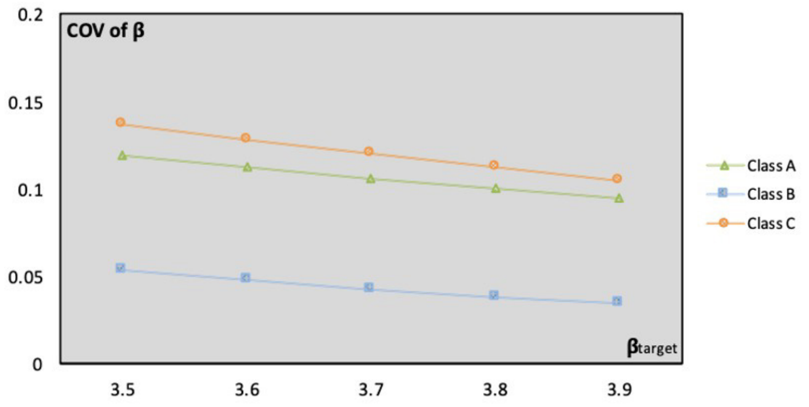


Figure 8. Dispersion of reliability indexes for unreinforced concrete masonry under compression.

On one hand, the reduction in the dispersion of reliability indexes is the main consequence of using Equation 7 in calibration, on the other hand, the reduction could be even greater if each class and modular size had its own γ_m.

Although results suggest that it is sensible to employ different partial safety factors for different classes and modular sizes, it is not common practice in major international codes used as references. Regardless of the value of γ_m, one way to reduce discrepancies in the safety level of masonry under compression in Brazil is through rigorous quality control during the manufacturing and receiving processes of blocks.

In general, the results presented reiterate that a high partial safety factor should be applied in NBR 16868-1:2020 [2]. A γ_m between 2.15 ($\beta_{target} \approx 3.7$) and 2.5 ($\beta_{target} \approx 3.76$) appears to be adequate for unreinforced masonry design in Brazil.

This work is a clear contribution towards the progress of Brazilian design codes. The general trends observed herein are valid, indicating a pathway in terms of achieving more uniform reliability in structural masonry built in the country.

4 CONCLUSIONS

This paper addressed a set of reliability analyses of Brazilian unreinforced concrete masonry members under compression. The work included a compilation of statistics on the strength of materials, modeling uncertainties and loading either derived from or adjusted to reflect the reality in Brazil.

Reliability analysis and the calibration process were done using the StRAnD software, which was developed in the Department of Structural Engineering at the School of Engineering of São Carlos.

The study revealed that strength and load variables, adjusted for structural masonry members with hollow concrete blocks built in Brazil, are consistent with the findings reported in the international literature.

The work showed that the reliability indexes are greater for unreinforced masonry walls under compression with modular size M-20 than modular size M-15, regardless of the strength class. The study also demonstrated that the compressive strength of concrete blocks is the most relevant random variable.

In performing this extensive research, covering a variety of structural configurations, it became clear that limit state specifications for masonry design in Brazil have not been developed from reliability-based calibration methods.

The actual level of safety of Brazilian masonry structures is not properly known. Furthermore, the situation is escalated by the fact that the strength properties of masonry are highly variable.

Despite the limitations previously mentioned, we believe that this work represents a stepping stone to the knowledge about the safety of masonry in Brazil, indicating a possible path in terms of achieving a more uniform reliability in masonry design.

The results presented reiterate the importance of a rigorous quality control during manufacturing and receiving processes of structural concrete blocks, as well as the relevance of the unreinforced masonry have γ_m different from reinforced masonry.

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