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ORIGINAL ARTICLE

Reliability analysis of reinforced concrete frames subjected to post-construction settlements

Análise de confiabilidade de pórticos em concreto armado sujeitos a recalques pós-construção

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Abstract: Most papers in the literature address reliability analysis of isolated elements, like beams and columns. However, symmetry and regularity are often exploited in the construction of regular RC frames, resulting in the same or similar designs for all columns of a floor or all beams of a building. This leads to significant differences in member reliability, due to different axial load to bending moment ratios, in different parts of the structure. Moreover, load effects increase, and symmetry is lost under individual support settlements. In this scenario, reliability analyses are performed, for an intact 4-floors and 3-spans RC frame; and considering different settlement conditions. Monte Carlo simulation is performed, considering uncertainties in dead and live loading, and steel and concrete strengths. The results show that a settlement of 10 mm, corresponding to an angular distortion of 1/500, reduced the average reliability of the frame by only 14%, just the same, it reduced the reliability index of several cross-sections of the beams to up to 2.40, value lower than that recommended in the Model Code 2010. It is concluded that the methodology used in this work presents an important tool for the analysis of events not foreseen in the design, supporting the decision making about the need for intervention in the structures.

Keywords: reinforced concrete, reliability analysis, Monte Carlo simulation, foundation settlements.

Resumo: A maior parte dos artigos encontrados na literatura endereça a confiabilidade de elementos isolados, como vigas e colunas. No entanto, simetria e regularidade são frequentemente exploradas na construção de pórticos regulares de concreto armado, o que resulta em projetos iguais ou semelhantes para todas as colunas de um andar ou todas as vigas de um prédio. Isto leva a diferenças na confiabilidade dos elementos, em função das diferentes razões entre carga axial e momento fletor, em diferentes partes da estrutura. Mais ainda, há um aumento dos esforços solicitantes e perda de simetria na presença de recalques de apoios. Neste cenário são realizadas análises de confiabilidade para um pórtico de 4 andares e 3 vãos, na situação intacta e considerando diferentes recalques de apoio. É realizada simulação de Monte Carlo, considerando incertezas nas ações permanentes e de utilização, e na resistência do aço e do concreto. Os resultados mostram que um recalque de 10 mm, correspondente a uma distorção angular de 1/500, reduziu a confiabilidade média do pórtico em apenas 14%, contudo reduziu o índice de confiabilidade de várias seções transversais das vigas para até 2.40, valor inferior ao recomendado no Model Code 2010. Conclui-se que a metodologia utilizada neste artigo se

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apresenta como uma ferramenta importante para a análise de eventos não previstos em projeto, auxiliando na tomada de decisão sobre a necessidade de intervenção nas estruturas.

Palavras-chave: concreto armado, confiabilidade estrutural, simulação de Monte Carlo, recalque de fundações.

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1 INTRODUCTION

In the structural engineering context, uncertainties are related to the inability to predict some characteristics of the structural system, such as loads, material properties, and assumptions of the structural model adopted [1], [2]. Despite these uncertainties, design codes employ design methodologies to obtain resistant, safe and robust structures.

Although the structural design considers these uncertainties in the design variables, it is still possible that a structure will be exposed to a condition not foreseen in the design [3]. Given these new conditions in the service situation, it is necessary to evaluate the new safety level of the structure, verifying the necessity of reinforcements or even the demolition of the system in more severe cases.

In terms of assessing the safety level of an existing structure, reliability theory can be employed for this purpose. Some authors have dedicated themselves to studying and evaluating the safety level of existing structures using such a theory. Works such as Facholli and Beck [4] and Beck et al. [5] have employed reliability theory to evaluate the safety level in structural element loss events. Küttenbaum et al. [6], Mankar et al. [7] and Souza et al. [8] evaluated the variations of the mechanical properties of materials and their impact on the system's reliability in situations of structure use. In Ávilla et al. [9] reliability analysis was applied to verify the safety level of historic buildings in regions susceptible to earthquakes.

In terms of structural design, many engineers still design structures without considering the effects of settlement [10]. Amancio [11] states that such a condition often occurs since predicting settlements in structures is still a complex factor due to the difficulty of obtaining parameters such as soil strength and deformability. Thus, this paper aims to contribute to the soil-structure interaction theme by developing a conceptual study to verify the influence of settlements on the safety of reticulated reinforced concrete structures. Therefore, this work intends to develop an analysis methodology that can contribute to decision-making regarding the maintenance of reinforced concrete structures of multiple floors submitted to actions arising from foundation settlement.

This paper is divided into six sections. The first three sections introduce the initial concepts of beam design and structural reliability. Section 4 discusses the conceptual problem of a frame structure submitted to settlement conditions. Sections 5 and 6 present the results and conclusions about this research.

2 DESIGN OF BEAMS UNDER PURE BENDING

This section presents the concepts related to structural analysis and the format of the limit state equation. The normative used were Brazilian standards ABNT NBR 8681 [12] and ABNT NBR 6118 [13].

The frame analysis was carried out for vertical loads. Equation 1 characterizes the ultimate limit state (g) of the beam's cross-section resistance capacity at stage III due to normal loads, as defined by ABNT NBR 6118 [13].

$$g(f_y, f_c, D, L) = E_R \cdot M_R(f_y, f_c) - M_S(D, L, f_c) \quad (1)$$

$$M_R = A_s \cdot f_y \cdot \left(d - \frac{\lambda}{2} \cdot x\right) \quad (2)$$

$$x = \frac{A_s \cdot f_y}{f_c \cdot b_w \cdot \alpha_c \cdot \lambda} \quad (3)$$

The loading moment M_S indicates the maximum demand of bending moment on the cross-section, determined herein using linear analysis. The resistant moment M_R indicates capacity of the RC cross-section. A_s is the steel area of the cross-section, λ and α_c are factors that depend on the characteristic compressive strength of concrete (f_c). These factors can be consulted in section 17.2.2 of ABNT NBR 6118 [13]. f_y represents the yield strength of the passive reinforcement steel used. d and b_w represent the effective height and width of the RC section. E_R represents model error variable for beam bending.

2.1 Determination of internal loads

The effects of the loads on the frames studied are evaluated by employing a linear-elastic static analysis. The mechanical model is based on the matrix analysis of structures, with frame-type elements (see Figure 1) and three degrees of freedom per node. Static linear analysis is sufficient for an approximate study of the load distribution in frame structures, allowing the redistribution of loads once the equilibrium and ductility conditions of ABNT NBR 6118 [13] are satisfied. However, in this paper, the analyses were performed without considering the redistribution of internal forces.

The nodal displacement vector $\boldsymbol{d}$ is obtained by a system containing the global stiffness matrix $\boldsymbol{K}$ and the external forces vector ($\boldsymbol{f}$). Thus, the system of equations that represents the discretized structural system will be given by Equation 4.

$$\boldsymbol{K} \cdot \boldsymbol{d} = \boldsymbol{f} \tag{4}$$

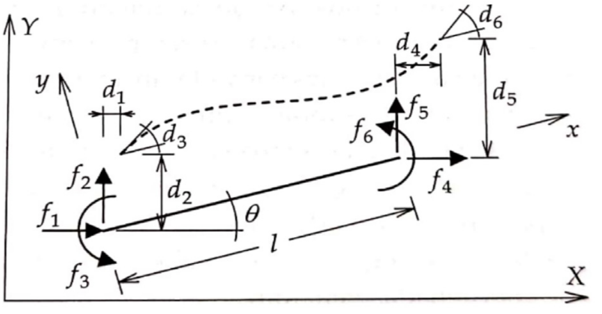

Figure 1. Frame element.

3 STRUCTURAL RELIABILITY AND MONTE CARLO METHOD

In this section, the basic concepts of the reliability evaluation of a structure are presented. The Monte Carlo method, employed herein for reliability analyses, is also presented.

The basic reliability problem is represented by the multiple integral of Equation 5, where p_f represents the failure probability of the structure, $\boldsymbol{X}$ is the n-dimensional vector representing the random variables of the system, $f_{\boldsymbol{x}}(\boldsymbol{x})$ represents the joint probability density function over the failure domain, and $G(\boldsymbol{X})$ is the limit state equation. $G(\boldsymbol{X}) \leq 0$ represents a failure condition.

$$p_f = P(G(\boldsymbol{X}) \leq 0) = \int \ldots \int_{G(\boldsymbol{X}) \leq 0} f_x(\boldsymbol{x})\, d\boldsymbol{x} \tag{5}$$

The probability of failure is a complementary concept to structural reliability. Failure probability measures the propensity of a structure or structural system to fail to satisfy the technical design requirements (function, strength, equilibrium) within a specified design life, respecting the operational and design conditions [2].

Several methods can be applied to solve Equation 5. In this work, the stochastic Monte Carlo method was applied. This algorithm was developed in the 1940s during the end of World War II and the beginning of the Cold War. It was initially employed by the mathematicians Stanislav Ulam and John von Neumann, who were working on developing the hydrogen bomb [14].

Among several variations of the Monte Carlo method, the Crude Monte Carlo was used in this work, which consists of random tests with a certain number of samples. The failure probability calculation is given by an approximation of Equation 5. The estimation of the failure probability using the Monte Carlo method is given by Equation 6.

$$\overline{p_f} = \frac{1}{n_s} \cdot \sum_{i=1}^{n_s} I[G(\boldsymbol{X})] = \frac{n_f}{n_s} \tag{6}$$

$$I[G(\boldsymbol{X})] = 1 \; if \; G(\boldsymbol{X}) \leq 0 \tag{7}$$

$$I[G(\boldsymbol{X})] = 0 \; if \; G(\boldsymbol{X}) > 0 \tag{8}$$

In Equation 6, n_s is the number of samples and n_f is the number of system failure events observed in n_s samples ($I[G(\boldsymbol{X})] = 1$).

The reliability index (β_{MC}) of the structure is obtained by Equation 9, which involves the inverse Standard Gaussian cumulative probability distribution. The numerical solution of this inverse function can be found in Beck [2].

$$\beta_{MC} = -\Phi^{-1}(\overline{p_f}) \tag{9}$$

3.1 Target Reliability Index

The reliability index will define a level of safety for the structure, but the design standards must be calibrated to a minimum level of safety required for any given structure. For existing structures, the minimum required value is given in Table 1.

Table 1. Suggested target reliability indices for existing structures, Model Code 2010 [15].

Limit State	β_{target}	Reference period
SLS	1.50	Service life
ULS	Between 3.10 and 3.80	50 years
	Between 3.40 and 4.10	15 years
	Between 4.10 and 4.70	1 year

These target index values are often used to estimate the partial safety coefficients in structural design standards such as ABNT NBR 8681 [12]. Applications of this calibration process can be seen in Santiago et al. [16].

4 STUDY OBJECT

This section presents the characteristics of the structural model used in the reliability analysis of a frame structure subjected to differential settlement and the numerical method employed in the reliability analysis.

The example used to evaluate structural safety is a four floors plane frame, as described in Facholli and Beck [4]. Figure 2 presents the geometry of the structural frame, which has three spans of 5 meters and a floor height of 3 meters. Table 2 shows the cross-section values for each element represented in Figure 2.

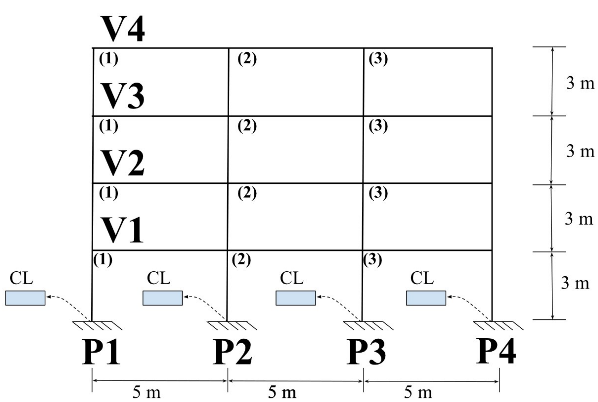

Figure 2. Plane frame in reinforced concrete analyzed and nomenclature of the columns, beams, and column layout (CL).

The structural design of the plane frame elements of Figure 2 was performed according to ABNT NBR 6118 [13]. All elements were considered to be constituted by a concrete with characteristic compressive strength of 30 MPa, and tangent and secant modulus of elasticity according to item 8.2.8 of ABNT NBR 6118 [13] (granite type aggregate). The beams are subjected to a live load to dead load ratio of 0.61 ($D_k = 26.38\ kN/m$, $L_k = 16\ kN/m$) as described in Facholli and Beck [4]. The live loads are established considering a residential building, with the rooms classified as "Pantry and laundry area", with 2.0 kN/m^2. For permanent loads, a total of 3.30 kN/m^2 is considered. The building slabs have a thickness of 0.10 meters [4]. It is worth noting that usual cases of the L/D ratio for reinforced concrete beams of buildings can vary between 0.1 and 0.60 [17].

The geometry of the element sections and the steel area are described in Figure 3. It is worth noting that the analyzed sections refer to beam-column connections (negative bending moment), which are the most loaded sections in a plane frame. For this work, the redistribution of internal loads in the reliability analysis was not considered.

It is also worth mentioning that beam V1 was used as a reference for the design of the typical floor and beam V4 has its own detailing because it is a roof element. Therefore, V1, V2 and V3 have the same structural detailing.

Table 2. Geometric properties of the elements.

Type element	Element ID	b_w *(cm)*	*h (cm)*
Beam	V1 = V2 = V3	20	45
	V4	20	45
Column	P1 = P4	40	20
	P3 = P3	50	20

Table 3. Settling conditions and angular distortion imposed on the foundation.

Load case	ρ-P1 (mm)	ρ -P2 (mm)	ρ -P3 (mm)	ρ -P4 (mm)	*p (Arbitrary)*	γ
ρ_{P1-5}	(i) 5.00	(j) 0	-	-	70%	1/1000
ρ_{P1-10}	(i) 10.00	(j) 0	-	-	25%	1/500
ρ_{P1-50}	(i) 50.00	(j) 0	-	-	5%	1/100
ρ_{P2-5}	-	(i) 5.00	(j) 0	-	70%	1/1000
ρ_{P2-10}	-	(i) 10.00	(j) 0	-	25%	1/500
ρ_{P2-50}	-	(i) 50.00	(j) 0	-	5%	1/100

The imposed frame settlements are shown in Table 3. The created patterns aim to produce an angular distortion ($\boldsymbol{\gamma}$) between columns of the building. The angular distortion is given by Equation 11:

$$\Delta_{ij} = \left|\rho_i - \rho_j\right| \tag{10}$$

$$\gamma = \frac{\Delta_{ij}}{L} \tag{11}$$

where Δ_{ij} indicates the differential settlement between columns, ρ_i and ρ_j are the individual settlements of each foundation and L is the span between foundations i and j.

The probabilities (p) of occurrence of each settlement magnitude in Table 3 were arbitrated but following a pattern which is typical of random phenomena affecting structural performance: the higher the magnitude, the lower the probability of occurrence. This same pattern is observed for extreme wind and live loads, earthquakes, floods, etc. This pattern has led to what is today known as Performance Based Engineering (see [2], [18]–[21]). To obtain more realistic failure probability estimates, actual foundation settlements for particular types of soils should be considered. Traditional bibliographies on the subject can be consulted, such as Skempton and MacDonald [22], Das [23], Burland et al. [24], and Nour [25]. In addition, the angular distortions established in Table 3 range from a less aggressive scale to rotation-to-rotation values which induce severe damage to the structure studied.

Reliability analysis by Crude Monte Carlo simulation (Equation 6) involves a heavy computational burden, due to the repetitive solutions of the numerical models. Due to this complexity, soil-structure interaction effects during settlement were not considered. This is left as a suggestion for future studies.

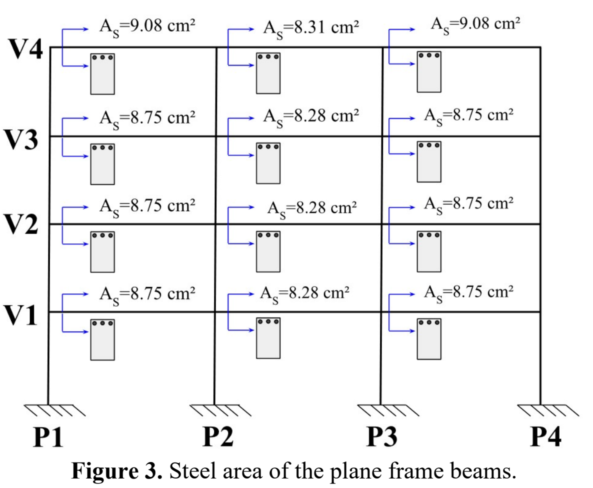

Figure 3. Steel area of the plane frame beams.

Table 4 presents the random variables considered in the reliability study. The other variables of the beam design problem (b_w, d, α_c, λ , and A_s) were considered deterministic.

Table 4. Random variable statistics.

Description	Variable	Distribution	Mean (μ)	Unit	C.o.V	Ref.
Dead load	D	Normal	$1.06 . D_k = 27.96$	kN/m	0.12	Santiago et al. [16]
Live load	L	Gumbel	$0.92 . L_k = 14.72$	kN/m	0.24	
Concrete	f_c	Normal	$1.22 . f_c = 36.60$	MPa	0.15	
Steel	f_y	Normal	$1.22 . f_y = 610.00$	MPa	0.04	
Model Error	E_r	Normal	1.02	---	0.06	Novak et al. [26]

Python language coding was used to perform the simulations. For the structural analyzer, the FINITO framework [27] was employed and for reliability analysis, an specific algorithm was developed.

The reliability analysis used the Crude Monte Carlo method described in section 3. The equation that defines only the ultimate limit state of the beams is given according to section 2, characterizing a bending failure without considering the effects of the beam-to-column connection. For the Monte Carlo analyses performed in this paper, a total of 2.10^5 samples of the five random variables (D, L, f_c, f_y, and E_r) were considered.

5 RESULTS AND DISCUSSION

The first part of this section presents the results concerning the reliability evaluation of the intact system, i.e., without foundation settlements. In the sequence are presented the results of the structure reliability for situations in which the differential settlements shown in Table 3 are inserted.

5.1 Reliability evaluation of the Intact Structure

The first part of the reliability results consists of evaluating the plane frame without the foundation settlements. Figure 4 shows the initial study of the number of samples required in the Monte Carlo method to have a convergence pattern. It can be seen, that after 150,000 samples the value of β does not change appreciably. Therefore, the sequence of reliability analyses was performed using n_s = 200,000 samples.

For the intact structure, reliability index (β_{MC}) for each beam element is presented in Figure 5.

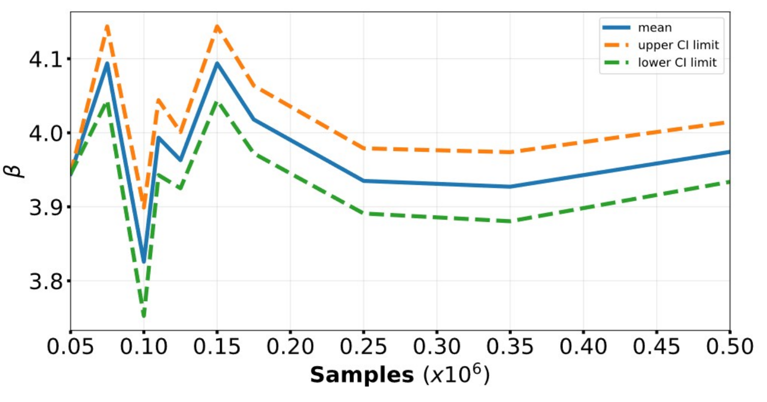

Figure 4. Convergence and Confidence Interval (95% confidence level) of the failure probability (β) for beams of the structural system.

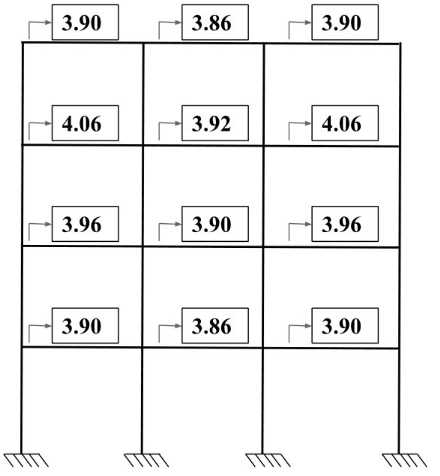

Figure 5. Reliability index β_{MC} for the beam elements.

It can be concluded from Figure 5 that the β_{MC} indexes are in agreement with the values in Table 1 that indicate a minimum required target index for the structure between 3.80 and 4.10 for a service life of 50 years. The higher reliability index value in beams V2 and V3 is expected since the control element for the typical floor design was beam V1.

It was also evaluated which of the five random variables in Table 4 have greater influence on Equation 1 which characterizes the ultimate limit state for bending. For this, a global sensitivity analysis is performed for each of the independent variables (Table 5) by a method based on variance decomposition, Sobol index [28] and by two regression-based methods, Standardized Regression Coefficients (SCR) and Partial Rank Correlation Coefficients (PRCC) [29].

In the Sobol technique, the first-order sensitivity S_i reports the influence of a variable x_i on the output and the total sensitivity index S_i^T refers to the influence of a variable x_i and the interactions of x_i with the other variables to the model output. The SRC technique quantifies the linear effect of each input variable on the response variable [30]. And the PRCC technique is the same as the Partial Correlation Coefficients (PCC), however the input and output values of the model are ranked. This technique allows qualitatively to verify only the order of importance of the variables, not how much the variable is more or less important than the others, the sensitivity of the response variable is determined as a function of the independent variable under study, disregarding the effect of the other variables [31], [32]. Table 5 shows the sensitivity values obtained from each independent variable for the methods used. In Table 5, negative values indicate strength variables, whereas positive values indicate load variables.

Table 5. Sensitivity analysis for the beams of the structural system.

Variable	S_i^T	SCR	PRCC
D	0.1514	0.4011	0.8033
L	0.2678	0.5414	0.8646
f_c	0.0403	-0.2167	-0.5815
f_y	0.1213	-0.3513	-0.7627
E_r	0.3492	-0.6093	-0.9005

S_i^T - Sobol Index; SCR - Standardized Regression Coefficients; and PRCC - Partial Rank Correlation Coefficients; D – Dead load L –Live Load; f_c – concrete strength; f_y – steel strength; and E_r – model error.

Through the global sensitivity analysis, it is possible to observe that despite different values for each of the different methods, the order of importance of the independent variables was the same. The independent variable of greater importance in the model (limit state) is the model error, E_r, represented by a total Sobol index of 0.35. The live load (L) was ranked second in the influence of the response variable, followed by the dead load (D), the steel strength (f_y), and lastly the concrete strength (f_c). This situation reveals the importance of the variability of the resistance model on the safety of the structure defined in the project. However, it is worth noting that the sensitivity analysis can be modified when the L_k/D_k ratio is altered. In the case of this work this ratio is 0.61.

5.2 Frame Reliability Evaluation after Settlements

Figures 6-8 present the reliability indexes of the differential settlement situations idealized in Table 3. It can be concluded that imposed settlements result in a reduction in the reliability index of the beams, reaching critical situations as in cases 3 and 6 with an imposed settlement of 50 mm and distortion 1/100. Clearly, the most affected beams are the reference beams (V1 and V4) that, theoretically, would be close to an optimal design ($R_d = S_d$).

To quantify the influence of settlement in the reduction of the reliability index of the beams, a reliability reduction index (D_μ) was associated for each of the cases of settlement analyzed. This index has already been used to determine the impact that explosive loads cause in existing structures, as can be verified in Momeni et al. [33]. Such an index is calculated by Equation 12 where β_{MC-INT} represents the reliability index of the intact structure and β_{MC-STL} the reliability index after the insertion of the settlement described in Table 3.

$$D_\mu = 1 - \frac{\beta_{MC-STL}}{\beta_{MC-INT}} \tag{12}$$

To quantify a single value for the D_μ index, an average value was established for the entire structure. This value represents an average of the D_μ values for each of the beams.

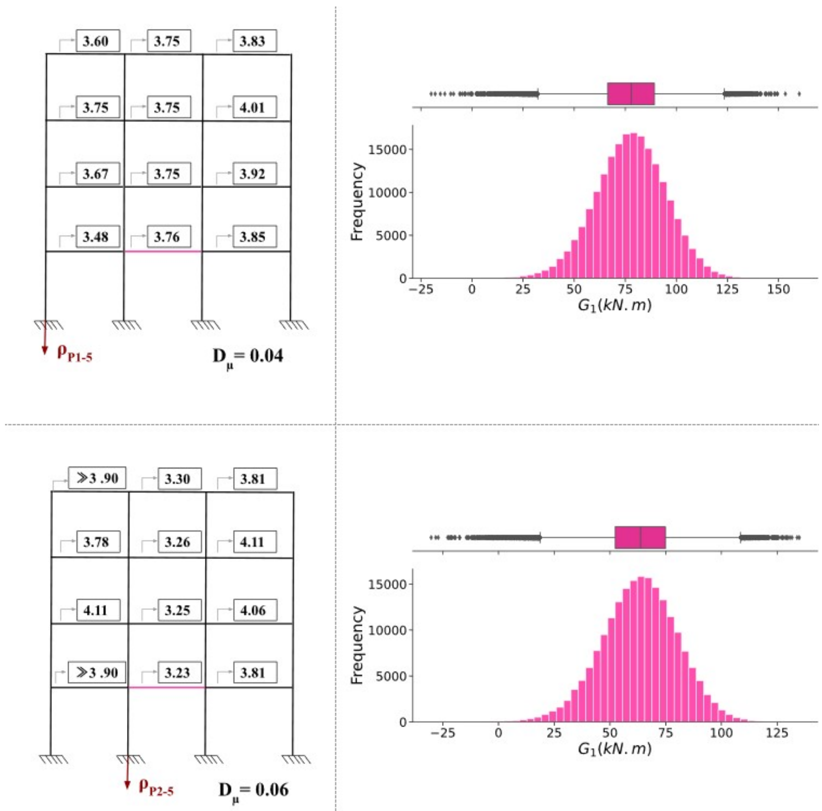

Figure 6. Reliability index β_{MC} for the beam elements considering the foundation settlement type ρ_{P1-5} and ρ_{P2-5}.

Figure 6 presents the reliability indices of the beams for the cases of settlement type ρ_{P1-5} and ρ_{P2-5}. Firstly, an index $D_\mu = 0.04$ and $D_\mu = 0.06$ is noted, which indicates an average reduction of about 5% in the reliability index of the original structure.

In terms of reducing the safety margin of the beams, defined by the reduction of the β_{MC} indices, the span most affected by the settlement at the internal column (ρ_{P2-5}) was span (2). Span (1) was more affected by the settlement at the external column (ρ_{P1-5}). In quantitative terms, the settlement at the internal column P2 promoted a more significant reduction in the reliability index of the beams. It is worth noting that this behavior also occurs for the 10 mm settlements (Figure 7).

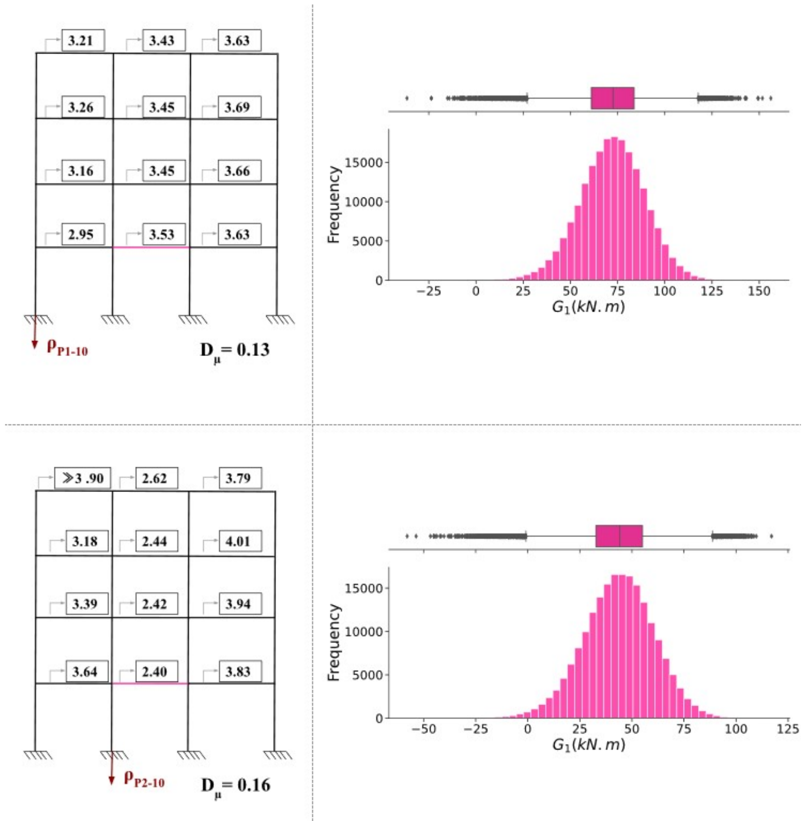

Figure 7. Reliability index β_{MC} for the beam elements considering the foundation settlement type ρ_{P1-10} and ρ_{P2-10}.

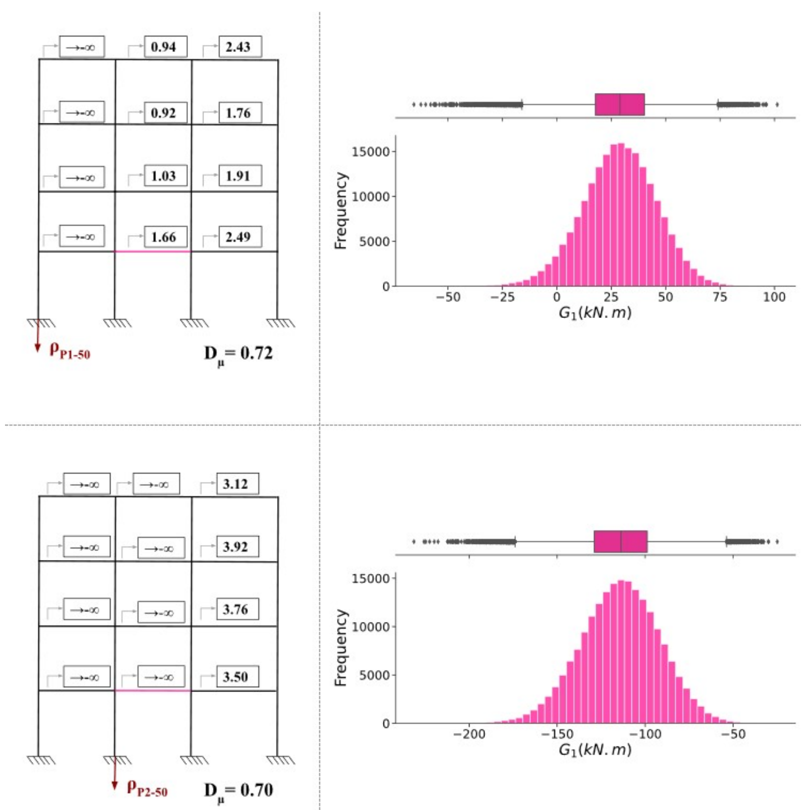

Figure 8. Reliability index β_{MC} for the beam elements considering the foundation settlement type ρ_{P1-50} and ρ_{P2-50}.

Figure 8 present the results of the reliability analysis for the last cases of settlement reported in Table 3. It can be perceived from these figures an appreciable reduction of the reliability indices of the beams for distortions of 1/500 and 1/100, especially the latter, which suggests that an ultimate limit state may have been reached, either by section failure or plastic hinge formation ($\beta_{MC} \rightarrow -\infty$ and $p_f \rightarrow 1$). This phenomenon occurs in spans (1) and (2) of the frame beams when the settlement occurs at column P2 and only in span (1) when the settlement occurs at the outer column P1, for settlement of 50 mm.

The Total Probability Theorem (Equation 13) was applied for each case to determine the total probability of failure of the structure for each of the effects of settlement on the reliability index of the structure, where event B represents a settlement event and A_i the mutually exclusive settlement events described in Table 3.

$$P[B] = P[B|A_1] \cdot P[A_1] + P[B|A_2] \cdot P[A_2] + \ldots + P[B|A_N] \cdot P[A_N] \tag{13}$$

For:

$$A_i \cap A_j = 0, i \neq j \tag{14}$$

$$A_1 + A_2 + \ldots + A_N = \Omega \tag{15}$$

The probability of occurrence of each of the effects were arbitrated in Table 3. The analysis of the reliability index after applying the theorem is shown in Figure 9.

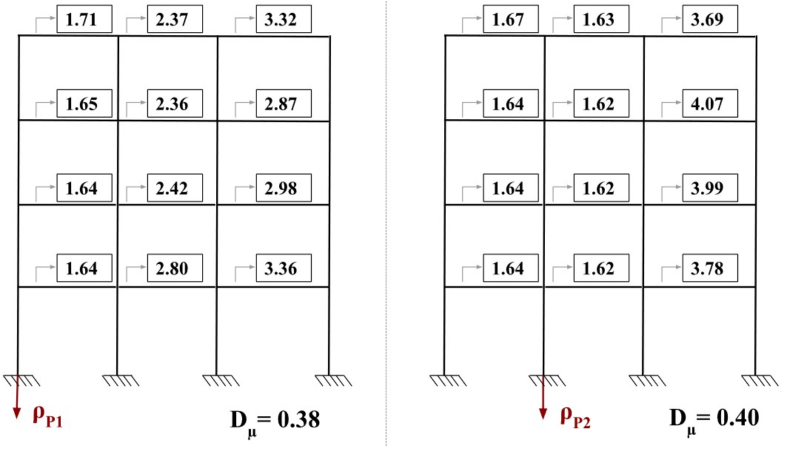

Figure 9. Reliability index β_{MC} for the internal (ρ_{P2}) and external (ρ_{P1}) settlements.

In terms of reduction of the index β_{MC} it is possible to confirm the information obtained in Figure 8 where the settlement condition in an external column (ρ_{P1}) led to a more attenuated failure condition compared to the settlement in the internal column (ρ_{P2}), with reductions of 38% and 40% respectively. Importantly, these values need to be confirmed by other studies addressing more realistic (measured) settlement statistics.

In order to reestablish reliability of the structure, some recovery measures are possible, as follows. If settlements have ceased, strengthening of the affected elements (beams, columns) or level off floor with hydraulic floor jacks is possible. When settlements are still ongoing, the foundation should be strengthened, by way of introducing new deep piles, use of belts to stabilize the foundation, and so on.

6 CONCLUSIONS

In this paper, the reliability of a reinforced concrete frame subjected to settlement was evaluated. The study revealed the importance of evaluating existing structures against events not foreseen in the project since many of these events can reduce the structure's safety. Moreover, the methodology presented here can assist engineers in determining the safety indexes of structures when exposed to unexpected settlements and thus help in the decision about the need for eventual reinforcements to restore the structure's original safety.

In the frame analyzed in this paper, it was found that, on average, the settlement in a column located at the end of the building reduced the reliability of the beams to a greater extent when compared to the effects produced by the settlement in an internal column.

In quantitative terms, the settlement of 10 mm (corresponding distortion of 1/500) reduced reliability of the structure's beam elements by about 14%. Although this reduction may look small, it lead to reliability index values β_{MC} as low as 2.40, which is lower than minimum recommended by the Model Code 2010 [15]. Thus, if the frame does not present significant load redistribution, the analysis indicates that the structure will present a safety level lower than that predicted in the design.

In this study, arbitrary values were considered for the probabilities of different magnitudes of foundation settlement, following the concept of performance-based design: larger settlement magnitudes are associated to smaller probabilities of occurrence. Results in Figure 9 should be reviewed considering more realistic (measured) foundation settlements for specific soil types.

In terms of the sensitivity of the variables, it can be concluded that the Error model (E_r) was the most important factor in the analyzed frame. The second most influential variable in this analysis was the live load (L).

In this work only one type of condition not foreseen in the project was addressed, which was the support settlements. However, other situations can be addressed in the future with this methodology, such as, for example, the assessment of the reliability of structures submitted to reinforcement corrosion. With this, the methodology can assist in decision-making about the need for corrective actions in structures deteriorated by the corrosion process.

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