

Structural Reliability Analysis Applied to the Flexural and Shear Design of a Reinforced Concrete Beam

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ABSTRACT: The design of reinforced concrete structures in Brazil is governed by NBR 6118:2026. However, deterministic design provisions do not explicitly account for uncertainties in structural behavior, which may result in uneconomical designs or inadequate safety levels. Structural reliability analysis provides a probabilistic basis for assessing these uncertainties and quantifying structural safety. This study evaluates the reliability of a reinforced concrete beam with respect to flexural and shear limit states. Failure probabilities were estimated using Monte Carlo simulation, the First-Order Second-Moment method (FOSM), and the First-Order Reliability Method (FORM). Two analyses were conducted. First, the beam was assessed under its original design condition, with $f_{ck} = 30$ MPa. Second, the same beam was analyzed considering concrete strength classes C20, C30, C40, and C50. The three methods provided comparable reliability estimates. The results showed that the probability of failure generally decreased with increasing concrete compressive strength and that the relative importance of the random variables varied according to the failure mode.

KEYWORDS: Reinforced concrete, Structural reliability, Monte Carlo simulation, FOSM, FORM

I. INTRODUCTION

Engineering design relies on analytical, numerical, and experimental models to predict structural response. Regardless of their level of refinement, however, these models involve assumptions and uncertainties and cannot reproduce all aspects of actual structural behavior. Uncertainty is also inherent in material properties, structural dimensions, and applied loads. Consequently, structural safety cannot be described only in deterministic terms, since a nonzero probability of failure remains even for structures designed in accordance with established standards.

Structural reliability analysis provides a means of accounting for these uncertainties and expressing structural safety in probabilistic terms [1]. In Brazil, reinforced concrete structures are designed according to national code requirements that define the relevant ultimate and serviceability limit states [2]. Reliability methods have been applied to different structural engineering problems, including the assessment of steel members under fire conditions [3], while established formulations of structural reliability provide the theoretical basis for evaluating failure probabilities and reliability indices [4]. Reliability concepts have also been used in risk analyses involving other civil

engineering systems, such as underground structures in fractured rock masses [5].

The interpretation of a calculated reliability level requires a reference against which the result can be assessed. The Joint Committee on Structural Safety (JCSS) provides target reliability indices for different consequences of failure and relative costs of safety measures, together with the corresponding probabilities of failure [6]. These target values are used in this study as reference criteria for interpreting the reliability results.

Against this background, this study evaluates the reliability of a reinforced concrete beam with respect to flexural and shear limit states. The analysis compares different reliability methods and examines the effect of concrete compressive strength on the probability of failure. Section II presents the basic concepts and reference reliability criteria, Section III describes the beam, random variables, analysis procedures, and limit-state functions, and Section IV presents and discusses the results. The main conclusions are summarized in Section V.

II. STRUCTURAL RELIABILITY

A. Initial Considerations

Structural reliability expresses the probability that a structure will satisfy a prescribed limit state over a specified reference period and under given service conditions. For an ultimate limit state, reliability is associated with the probability that structural resistance remains greater than the corresponding load effects during the considered period [1].

Limit states are generally classified into two categories: Ultimate Limit States (ULS), associated with structural safety, and Serviceability Limit States (SLS), associated with conditions of normal use and performance [2].

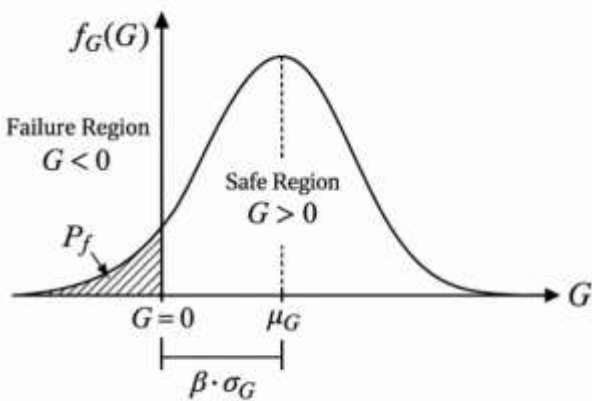
A simple formulation of the structural reliability problem considers two statistically independent and normally distributed random variables, $\mathbf{R}$ and $\mathbf{S}$, representing resistance and load effect, respectively. The corresponding limit-state function is defined by Equation 1.

$$G(R, S) = R - S \quad (1)$$

Failure occurs when the load effect exceeds the resistance, i.e., when $G < 0$ (Figure 1). Thus, the probability of failure is given by Equation 2.

$$P_f = P[G < 0] \quad (2)$$

Figure 1. Probability density function



of the limit-state function G and failure region, adapted from [3].

Since $\mathbf{R}$ and $\mathbf{S}$ are assumed to be statistically independent normal random variables, the mean and standard deviation of G can be obtained from Equations (3) and (4) [4].

$$\mu_G = \mu_R - \mu_S \quad (3)$$

$$\sigma_G = \sqrt{\sigma_R^2 + \sigma_S^2} \quad (4)$$

The probability of failure can be calculated using the following equation:

$$P_f = \Phi\left(-\frac{\mu_G}{\sigma_G}\right) \quad (5)$$

where Φ is the cumulative distribution function of the standard normal distribution, $N(0,1)$.

For this simplified case, the reliability index, β , is defined by the ratio μ_G/σ_G , as given in Equation (6).

$$\beta = \frac{\mu_G}{\sigma_G} = \frac{\mu_R - \mu_S}{\sqrt{\sigma_R^2 + \sigma_S^2}} \quad (6)$$

The reliability index provides a measure of structural safety; larger values of β correspond to lower probabilities of failure.

In more general problems, resistance and load effects depend on several basic random variables, which may follow non-Gaussian distributions. In such cases, the simplified formulation given above is no longer generally applicable, and more comprehensive reliability methods may be required, such as the First-Order Second-Moment (FOSM) method, the First-Order Reliability Method (FORM), and Monte Carlo simulation.

B. Target Reliability Indices

A reference criterion is required to assess whether the calculated reliability of a structural element is adequate. Such criteria may be established from design codes, standards, or reliability guidelines [5].

The Joint Committee on Structural Safety (JCSS) provides target reliability indices and corresponding probabilities of failure according to the consequences of failure and the relative cost of safety measures [6]. Table I presents these values for ULS and SLS over a one-year reference period.

Minor consequences of failure are those for which the risk to life and the economic consequences are small or negligible, as in agricultural structures [6]. Moderate consequences of failure involve considerable risk to life or economic impacts, as in office buildings and apartment buildings. Large consequences of failure are associated with significant risks to life and major economic consequences, as in bridges, theaters, hospitals, and high-rise buildings.

TABLE I: RELIABILITY INDICES FOR ULS AND SLS OVER A ONE-YEAR REFERENCE PERIOD, ADAPTED FROM JCSS [6].

Relative cost of safety measure	ULS			SLS
	Consequences of failure			
	Minor	Moderate	Large	

Large (A)	$\beta = 3.1$ ($P_f \approx 10^{-3}$)	$\beta = 3.3$ ($P_f \approx 5 \times 10^{-4}$)	$\beta = 3.7$ ($P_f \approx 10^{-4}$)	$\beta = 1.3$ ($P_f \approx 10^{-1}$)
Normal (B)	$\beta = 3.7$ ($P_f \approx 10^{-4}$)	$\beta = 4.2$ ($P_f \approx 10^{-5}$)	$\beta = 4.4$ ($P_f \approx 5 \times 10^{-6}$)	$\beta = 1.7$ ($P_f \approx 5 \times 10^{-2}$)
Small (C)	$\beta = 4.2$ ($P_f \approx 10^{-5}$)	$\beta = 4.4$ ($P_f \approx 5 \times 10^{-6}$)	$\beta = 4.7$ ($P_f \approx 10^{-6}$)	$\beta = 2.3$ ($P_f \approx 10^{-2}$)

III. METHODOLOGICAL PROCEDURES

A. Initial Considerations

A reliability analysis was performed for a reinforced concrete beam in a residential building, designed using Eberick 2026 software, with a rectangular cross-section measuring 18 cm $\times$ 55 cm and a span of 5.5 m. The concrete used has a characteristic compressive strength, f_{ck} , of 30 MPa; the longitudinal reinforcement and stirrups consist of CA-50 and CA-60 steel, respectively. The beam's geometric properties and reinforcement are shown in Figure 2.

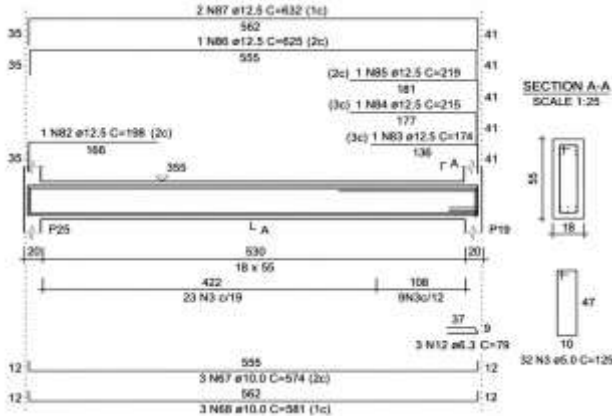


Figure 2. Geometry and reinforcement details of the analyzed beam.

Table II presents the statistical parameters of the loads, material properties, and geometric variables considered in the reliability analysis.

Here, f_{ck} denotes the characteristic compressive strength of concrete, f_{yk} the characteristic yield strength of the longitudinal reinforcement, and f_{yw} the characteristic yield strength of the transverse reinforcement.

The statistical parameters for the dead and live loads were obtained from Ellingwood and Galambos [7]. The dead load was modeled using a normal distribution with a mean of $1.05g_{nom}$ and a coefficient of variation of 0.1, whereas the live

load was modeled using a Gumbel distribution with a mean of $1.0q_{nom}$ and a coefficient of variation of 0.25.

For f_{ck} , the data were obtained from Santiago and Beck [8]. The data relating to f_{yk} were taken from Szerszen and Nowak [9] and follow a normal distribution with a mean of $1.08f_{yknom}$ and a coefficient of variation of 0.08.

Statistical information regarding the beam dimensions was obtained based on parameters provided by JCSS [6]. These parameters quantify the uncertainty in the dimensions of concrete members and follow a normal distribution.

In this study, two distinct procedures were used to analyze the reliability of the beam. In the first procedure, f_{ck} was set to 30 MPa, as specified in the structural design, and the reliability analysis was performed using the Monte Carlo, FOSM, and FORM methods. These analyses made it possible to obtain the probability of failure and its corresponding reliability index, as well as the sensitivity indices for each random variable present in the problem. Sensitivity indices were used to assess the relative influence of each random variable on the estimated probability of failure.

In the second procedure, the same beam was analyzed considering concrete strength classes C20, C30, C40, and C50, corresponding to characteristic compressive strengths of 20, 30, 40, and 50 MPa, respectively. In this analysis, the Monte Carlo method was used, and the number of simulations was varied. This method made it possible to determine the probability of failure.

The Monte Carlo and FOSM methods were implemented in MATLAB using custom-developed algorithms, while the FORM method was implemented using the Risk Tools (RT) software.

B. Limit-State Functions

Three limit-state functions were used to analyze the beam's reliability: one for flexural failure and two for shear failure, corresponding to failure of the compressed concrete diagonals and failure due to diagonal tension. These limit-state functions are given in Equations (7) – (9).

TABLE II: STATISTICS FOR THE RANDOM VARIABLES.

Random Variables		Unit	Nominal Value	Mean	Standard deviation	Distribution	Reference
Loads	Dead Load (g)	kN/cm	0.49171	0.5163	0.05163	Normal	[7]
	Live Load (q)	kN/cm	0.10755	0.10755	0.02689	Gumbel	[7]
Material Strengths	Concrete C20 (f_{ck})	kN/cm ²	2	2.60476	0.18470	Normal	[8]
	Concrete C30 (f_{ck})	kN/cm ²	3	3.37641	0.35540	Normal	[8]
	Concrete C40 (f_{ck})	kN/cm ²	4	4.18905	0.35700	Normal	[8]

	Concrete C50 (f_{ck})	kN/cm ²	5	4.73015	0.29100	Normal	[8]
	Steel CA-50 (f_{yk})	kN/cm ²	50	54	4.32000	Normal	[9]
	Steel CA-60 (f_{ywk})	kN/cm ²	60	64.8	5.18400	Normal	[9]
Geometric Variables	Width - b_w	cm	18	17.946	0.50800	Normal	[6]
	Height - h	cm	55	54.835	0.73000	Normal	[6]

- Limit-state function for flexural failure:

$$G_1(\mathbf{X}) = (10846 b_w f_{ck} + 4.71 f_{yk})(h - 12.88) - 37812.5(g + q) \quad (7)$$

- Limit-state function for diagonal compression failure:

$$G_2(\mathbf{X}) = 0.27 \left(1 - \frac{f_{ck}}{250} \right) f_{ck} b_w (h - 6.5) - 275(g + q) \quad (8)$$

- Limit-state function for diagonal tension failure:

$$G_3(\mathbf{X}) = (h - 6.5)(0.126 f_{ck}^{2/3} b_w + 0.01895 f_{ywk}) - 275(g + q) \quad (9)$$

IV. RESULTS AND DISCUSSION

A. Reliability Analysis for C30 Concrete

1) Flexural failure:

For flexural failure, the Monte Carlo, FOSM, and FORM methods yielded the failure probabilities and corresponding reliability indices shown in Table III.

TABLE III: RELIABILITY ANALYSIS FOR FLEXURAL FAILURE.

Method	β	P_f	Iterations	time (s)
Monte Carlo	3.958	3.77×10^{-5}	1.50×10^7	0.5703
FOSM	3.843	6.08×10^{-5}	5	0.0038
FORM	3.838	6.21×10^{-5}	11	0.1000

Given that the beam under investigation is associated with moderate consequences of failure, as it is part of a residential building, the JCSS [6] recommends target reliability indices ranging from 3.3 to 4.4, depending on the relative cost of safety measures. The reliability indices obtained using the three methods, ranging from 3.838 to 3.958, fall within this overall range of target values.

For the Monte Carlo method, the required number of simulations was determined by limiting the coefficient of variation of the estimated failure probability to 5%. The adopted number of simulations resulted in a CV of 0.04 (4%). Regarding computational performance, the Monte Carlo method required the longest processing time (0.5703 s), with

1.50×10^7 simulations, followed by FORM (0.10 s), whereas FOSM showed the shortest processing time (0.0038 s). These results refer to the implementations adopted in this study.

Figure 3 presents the sensitivity indices for the random variables considered in the analysis. The concrete characteristic compressive strength (f_{ck}) exhibits the highest sensitivity index (0.51), followed by the dead load (g), with an index of 0.24.

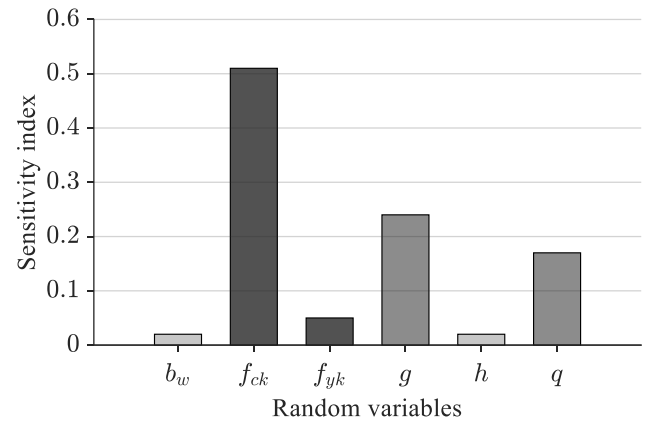


Figure 3. Sensitivity indices for flexural failure.

2) Diagonal compression failure:

For diagonal compression failure, the Monte Carlo, FOSM, and FORM methods yielded the failure probabilities and corresponding reliability indices shown in Table IV.

TABLE IV: RELIABILITY ANALYSIS FOR DIAGONAL COMPRESSION FAILURE.

Method	β	P_f	Iterations	time (s)
Monte Carlo	> 8.00	0	1.0×10^8	4.7955
FOSM	7.201	3.00×10^{-13}	6	0.0043
FORM	7.214	2.72×10^{-13}	6	0.1040

The three methods consistently indicated an extremely low probability of failure for diagonal compression. No failure events were observed in the Monte Carlo simulation with 10^8 samples, resulting in an empirical failure probability of zero. The reliability indices obtained with FOSM and FORM were substantially higher than all JCSS target values reported for moderate consequences of failure, indicating a very high reliability level with respect to diagonal compression failure. FOSM and FORM required the same

number of iterations; however, for the implementations adopted in this study, FORM required a longer processing time than FOSM.

Figure 4 presents the sensitivity indices for the random variables considered in the analysis. The concrete characteristic compressive strength, f_{ck} , exhibited the highest sensitivity index (0.91), indicating that it was the most influential variable for diagonal compression failure.

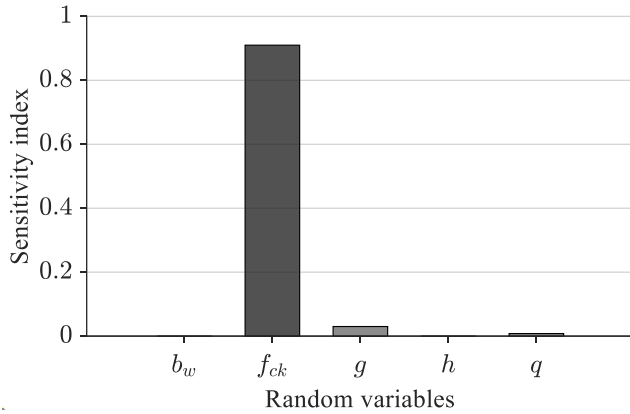


Figure 4. Sensitivity indices for diagonal compression failure.

3) Diagonal tension failure:

The Monte Carlo, FOSM, and FORM methods yielded the failure probabilities and corresponding reliability indices shown in Table V. All three methods indicated a very low probability of failure, with FORM providing the highest estimate.

TABLE V: RELIABILITY ANALYSIS FOR DIAGONAL TENSION FAILURE.

Method	β	P_f	Iterations	time (s)
Monte Carlo	5.293	6.00×10^{-8}	1.00×10^8	131.4539
FOSM	5.191	1.04×10^{-7}	5	0.0021
FORM	5.025	2.51×10^{-7}	15	0.1360

The reliability indices obtained with all three methods were higher than all JCSS [6] target values reported for moderate consequences of failure, indicating a very high reliability level with respect to diagonal tension failure.

For the Monte Carlo method, 1.00×10^8 simulations were performed because of the very low failure probability, resulting in a computational time of 131.4539 s. Among the implementations considered in this study, Monte Carlo required the longest computational time. FORM required a longer computational time than FOSM, with 0.1360 s and 0.0021 s, respectively.

Figure 5 presents the sensitivity indices for the random variables considered in the analysis.

The live load, q , exhibited the highest sensitivity index of 0.37, followed closely by the concrete characteristic compressive strength, f_{ck} , with an index of 0.36. The relatively high influence of q may be associated with its coefficient of variation of 0.25.

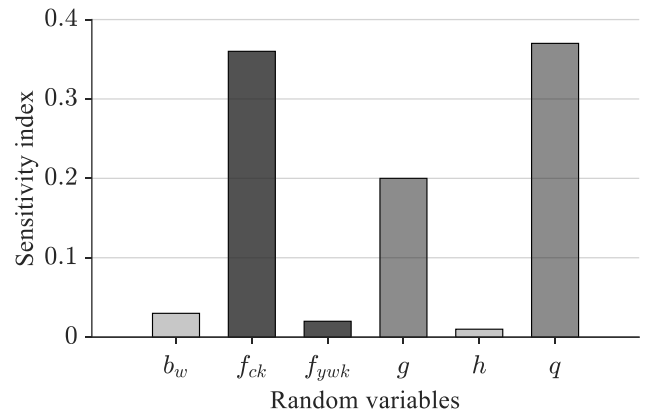


Figure 5. Sensitivity indices for diagonal tension failure.

B. Reliability Analysis for Concrete Strength Classes C20, C30, C40, and C50

The following section presents the reliability analyses for the three limit states considering concrete strength classes C20, C30, C40, and C50.

1) Flexural failure:

The Monte Carlo method was used to estimate the probability of flexural failure for concrete strength classes C20, C30, C40, and C50, considering different numbers of simulations, as shown in Table VI.

TABLE VI: MONTE CARLO ESTIMATES OF THE PROBABILITY OF FLEXURAL FAILURE FOR DIFFERENT CONCRETE STRENGTH CLASSES AND NUMBERS OF SIMULATIONS.

No. of simulations	Probability of failure P_f			
	C20	C30	C40	C50
1.0×10^4	1.6×10^{-3}	1.00×10^{-4}	0	0
1.0×10^5	1.3×10^{-3}	4.00×10^{-5}	0	0
1.0×10^6	1.1×10^{-3}	3.30×10^{-5}	0	0
1.0×10^7	1.1×10^{-3}	3.31×10^{-5}	0	0
3.0×10^7	1.1×10^{-3}	3.38×10^{-5}	0	0
5.0×10^7	1.1×10^{-3}	3.46×10^{-5}	4.0×10^{-8}	0

The results show that increasing the concrete characteristic compressive strength leads to a substantial reduction in the estimated probability of flexural failure. For C20 concrete, the estimated probability of failure stabilized at approximately 1.1×10^{-3} , which is higher than all JCSS target failure probabilities reported for moderate consequences of failure. For C30 concrete, the estimated probability of failure was approximately 3.4×10^{-5} , falling within the overall range

of target failure probabilities reported by the JCSS for moderate consequences of failure.

For C40 concrete, no failure events were observed up to 3.0×10^7 simulations, whereas failure events were observed when 5.0×10^7 simulations were performed, resulting in an estimated failure probability of 4.0×10^{-8} . For C50 concrete, no failure events were observed throughout the analyzed simulation range. These results indicate a very low probability of flexural failure for the higher concrete strength classes. However, the absence of observed failure events should not be interpreted as a true failure probability equal to zero.

Figure 6 illustrates the variation in the estimated probability of flexural failure with the number of Monte Carlo simulations for each concrete strength class. For C20 and C30, the estimates initially decrease and then become approximately stable as the number of simulations increases. For C40 and C50, cases in which no failure events were observed are displayed at the lower plotting limit solely for visualization purposes. Therefore, these plotted values do not represent estimated failure probabilities of 1.0×10^{-8} ; they are used only to allow the corresponding cases to be displayed on the logarithmic scale.

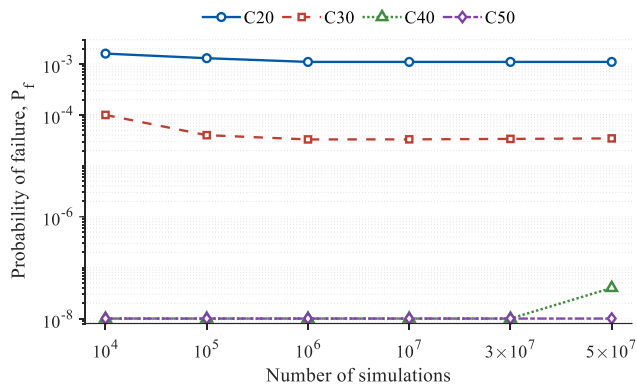


Figure 6. Monte Carlo estimates of the probability of flexural failure versus number of simulations for different concrete strength classes.

2) Diagonal compression failure:

The Monte Carlo method was used to estimate the probability of diagonal compression failure for concrete strength classes C20, C30, C40, and C50, considering different numbers of simulations, as shown in Table VII.

TABLE VII: MONTE CARLO ESTIMATES OF THE PROBABILITY OF DIAGONAL COMPRESSION FAILURE FOR DIFFERENT CONCRETE STRENGTH CLASSES AND NUMBERS OF SIMULATIONS.

No. of simulations	Probability of failure P_f			
	C20	C30	C40	C50
1.0×10^4	0	0	0	0
1.0×10^5	0	0	0	0

1.0×10^6	0	0	0	0
1.0×10^7	0	0	0	0
3.0×10^7	0	0	0	0
5.0×10^7	0	0	0	0

No failure events were observed for any of the concrete strength classes over the analyzed simulation range, up to 5.0×10^7 simulations. Consequently, the empirical failure probability remained equal to zero for all cases considered. These results indicate a very high reliability level with respect to diagonal compression failure. However, the absence of observed failure events should not be interpreted as a true failure probability equal to zero.

Because no failure events were observed for any concrete strength class, a graph of the estimated probability of failure versus number of simulations was not included, as all curves would coincide at zero over the analyzed simulation range.

3) Diagonal tension failure:

The Monte Carlo method was used to estimate the probability of diagonal tension failure for concrete strength classes C20, C30, C40, and C50, considering different numbers of simulations, as shown in Table VIII.

TABLE VIII: MONTE CARLO ESTIMATES OF THE PROBABILITY OF DIAGONAL TENSION FAILURE FOR DIFFERENT CONCRETE STRENGTH CLASSES AND NUMBERS OF SIMULATIONS.

No. of simulations	Probability of failure P_f			
	C20	C30	C40	C50
1.0×10^4	0	0	0	0
1.0×10^5	0	0	0	0
1.0×10^6	1.00×10^{-6}	0	0	0
1.0×10^7	5.00×10^{-7}	4.00×10^{-7}	0	0
3.0×10^7	2.67×10^{-7}	1.00×10^{-7}	0	0
5.0×10^7	5.40×10^{-7}	4.00×10^{-8}	0	0

The results indicate that increasing the concrete characteristic compressive strength generally leads to a reduction in the estimated probability of diagonal tension failure. For C20 and C30 concrete, the estimated probabilities of failure remained below all JCSS [6] target failure probabilities reported for moderate consequences of failure over the analyzed simulation range. For C40 and C50 concrete, no failure events were observed in any of the Monte Carlo simulations, and the corresponding empirical probability of failure therefore remained equal to zero. However, the absence of observed failure events should not be interpreted as a true probability of failure equal to zero.

Figure 7 illustrates the variation in the estimated probability of diagonal tension failure with the number of Monte Carlo simulations.

For C20 and C30, the estimates fluctuate as the number of simulations increases, reflecting the very small number of observed failure events and the resulting sensitivity of the estimates to sample size. For C40 and C50, no failure events were observed throughout the analyzed simulation range. In Figure 7, the probability of failure is presented on a logarithmic scale. For cases in which no failure events were observed, the zero empirical estimates are positioned at the lower plotting limit solely for visualization purposes and should not be interpreted as probabilities equal to that value.

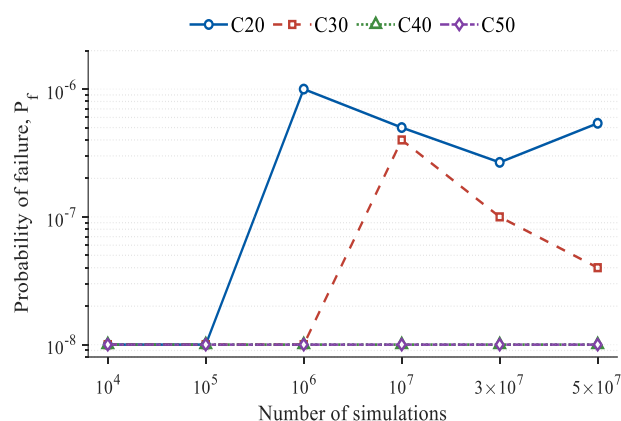


Figure 7. Monte Carlo estimates of the probability of diagonal tension failure versus number of simulations for different concrete strength classes.

V. CONCLUSIONS

This study evaluated the structural reliability of a reinforced concrete beam with respect to flexural and shear limit states. Two complementary analyses were performed. First, the beam designed with $f_{ck} = 30$ MPa was assessed using Monte Carlo simulation, the First-Order Second-Moment method (FOSM), and the First-Order Reliability Method (FORM). Second, Monte Carlo simulations were conducted for concrete strength classes C20, C30, C40, and C50, considering different numbers of simulations. For the beam as originally designed, $f_{ck} = 30$ MPa, the three methods yielded comparable reliability estimates, with particularly close agreement between FOSM and FORM. For the implementations adopted in this study, Monte Carlo required the greatest computational effort.

For flexural failure, the reliability indices obtained for $f_{ck} = 30$ MPa ranged from 3.8377 to 3.9580, falling within the overall range of JCSS target values reported for moderate consequences of failure. The shear limit states exhibited substantially higher reliability levels: no diagonal compression failure events were observed in the Monte Carlo simulations, while diagonal tension failure probabilities remained very low. The parametric analysis also showed that increasing the concrete characteristic compressive strength generally reduced the estimated probability of failure. For C20 concrete, the estimated probability of flexural failure stabilized at approximately 1.1×10^{-3} , exceeding all JCSS

target failure probabilities reported for moderate consequences of failure, whereas for C30 it was approximately 3.4×10^{-5} , within the overall range of the corresponding target values. For C40 and C50, failure events were either extremely rare or not observed within the analyzed simulation ranges, depending on the limit state considered. Overall, the results show that the influence of concrete strength on structural reliability depends on the failure mode considered, with f_{ck} being particularly influential in the flexural and diagonal compression analyses. The absence of observed failure events in Monte Carlo simulations should, however, not be interpreted as a true probability of failure equal to zero.

Future studies may extend the present analysis to other reinforced concrete members, structural configurations, and loading conditions in order to assess the generality of the observed reliability trends. Further investigations could also incorporate additional sources of uncertainty, including model uncertainty and variability in other material and geometric parameters. For failure modes associated with very low probabilities, the use of advanced reliability or variance-reduction techniques may also be explored to improve the estimation of rare-event probabilities while reducing the computational effort required by direct Monte Carlo simulation.

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