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Reliability of prestressed concrete girders design: Comparison between NBR and Eurocode codes

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***Abstract*— This work evaluates the reliability index of prestressed girders beams of highway bridges designed according to Brazilian and European codes concerning the bending ultimate limit state. A model was used to verify the resistant capacity of the beams in the bending ultimate limit state and to evaluate the live load models proposed by the regulations studied. The bridge studied spans 27.4 meters, and four T section prestressed concrete girders support it. To obtain the reliability index β, the First Order Reliability Method (FORM) was used, which proved to be very fast and efficient. The values collected by the analytical method are validated using the Monte Carlo simulation method. For all cases, the reliability index value was higher than 5.5, showing that the two normative recommendations for the design of girders beams guarantee a very conservative level of safety.*

I. INTRODUCTION

It is called a bridge, the structure to overcome obstacles, such as rivers, deep valleys, other roads, and other cases. Bridge failures can cause great losses and inconvenience to people around them and the economy. Therefore, it is of great importance to study these structures' safety to avoid such adversities.

The most used method for evaluating the safety of bridges is structural reliability. Its main objective is to determine the probability of structural failures, which is related to the reliability index.

The structural reliability method requires the statistical definition of the parameters included in the model, which depends on the quality of the statistical data referring to the problem and the precision of the mathematical model used to verify the limit state equations.

Some codes, such as the Eurocodes from Europe and the "Load and Resistance Factor Design" from the United States, define a target reliability index according to the importance of the structure or structural element.

The main Brazilian codes for bridge design are NBR 7187 [1] and NBR 7188 [2], entitled "Design of concrete bridges, viaducts and footbridges", and "Road and pedestrian live load on bridges, viaducts, footbridges and other structures", respectively. In Europe, the Eurocode 1 [3] and Eurocode 2 [4] codes, entitled "Actions on structures" and "Design of concrete structures", respectively, are used for the design of such structures.

Brazilian and European design codes take into account numerous uncertainties associated with the materials used in the construction of the structure and the actions that will be applied through the limit state method. This method involves establishing boundaries between desirable and undesirable behavior of the structure.

In this format, partial safety factors are proposed to reduce the strength of the structural elements and increase the actions, creating a safety margin.

In Brazilian design codes, the live load model consists of a 3-axle vehicle plus a distributed load, applied in the region outside the vehicle boundaries, and multiplied by a dynamic amplification factor, called impact coefficient, which is a function of span length, number of spans and material used in the structure.

The European design codes define four vertical load models that must be considered on road bridges. It's also defined horizontal forces associated with two of these models due to braking, acceleration, and centrifugal forces. The loads are then combined in groups of traffic loads, and the worst case is included in the load combinations.

This work aims to compare, based on structural reliability, the principal Brazilian and European codes used for the design of prestressed concrete bridges, with the main differences being the partial safety factors and the live load models applied. Thus, it will be possible to observe which code provides greater security in relation to the other through its regulations.

II. METHODOLOGY

2.1 Structural analysis and design of the bridge

Two prestressed concrete bridge girders were designed, one based on Brazilian codes NBR 7187 [1] and NBR 7188 [2] and another based on European regulations Eurocode 1 [3] and Eurocode 2 [4]. For both structures, the same cross-section and the same properties of the bridge materials were adopted.

2.1.1 Geometric characteristics and properties of bridge materials

The bridge studied, shown in Figure 1, is a single-span bridge with prestressed precast beams and an "in loco" casted slab and has a span of 27.4 m. Four prestressed precast beams support the bridge, with an "in loco" casted slab 21 cm thick and the asphalt layer 7 cm thick. The precast prestressed beam has a height of 2 m. The dimensions of the prestressed precast beam and the guardrail are shown in Figure 2. For the analysis, beam number 1 was considered.

It was used seven high-strength wire strands for both design examples. Wire strands with a 12.7-millimeter diameter, designated as "Y1860S7G" were selected. Table 1 shows the values of the main properties of the bridge materials.

2.1.2 Structural internal forces in beam number 1

The analysis of transverse load distribution is performed using the Fauchart Method, used for bridges without intermediate crossbeams. This method provides simple modeling and results very close to those obtained by a process with greater precision, such as the Finite Element Method, according to Moura et al. [5].

The Fauchart Method allows converting a two-dimensional loading model into a one-dimensional one by considering the stiffness of the girders based on the use of springs (FONTANA [6]).

Heinen [7] states that the Fauchart Method is used in decks with several beams without intermediate crossbeams. The longitudinal beams must be simply supported, have constant inertia, and behave according to the rules of Strength of Materials (linear elastic behavior). In this method, the longitudinal behavior of the slab is neglected.

The Fauchart Method suggests calculating a plane structure corresponding to one meter in width of the slab's cross-section. The beams are replaced by springs that have resistance to vertical displacement and in-plane rotation. The spring constants are obtained by applying the Euler-Bernoulli beam theory and the elastic torsion theory. Their resolution is given using the Fourier series, resulting in Equation 1 and Equation 2.

$$k_v = \left(\frac{\pi}{L}\right)^4 \cdot E_{cs} \cdot I \tag{1}$$

$$k_t = \left(\frac{\pi}{L}\right)^4 \cdot G \cdot J \tag{2}$$

Where L is the span length in meters, I is the moment of inertia of the beam section in m^4, J is the torsion constant of the beam in m^4, E_{cs} is the secant modulus of elasticity of the beam's material in kN/m^2, G is the transverse modulus of elasticity of the beam's material in kN/m^2, k_v is the coefficient of the vertical spring in kN/m and k_t is the coefficient of the torsion spring in $kN \cdot m/rad$ (STUCCHI [8]).

The influence lines and the reactions on the supports are obtained by solving the structure on elastic supports for different positions of a unit load, as shown in Figure 3 (HEINEN [7]).

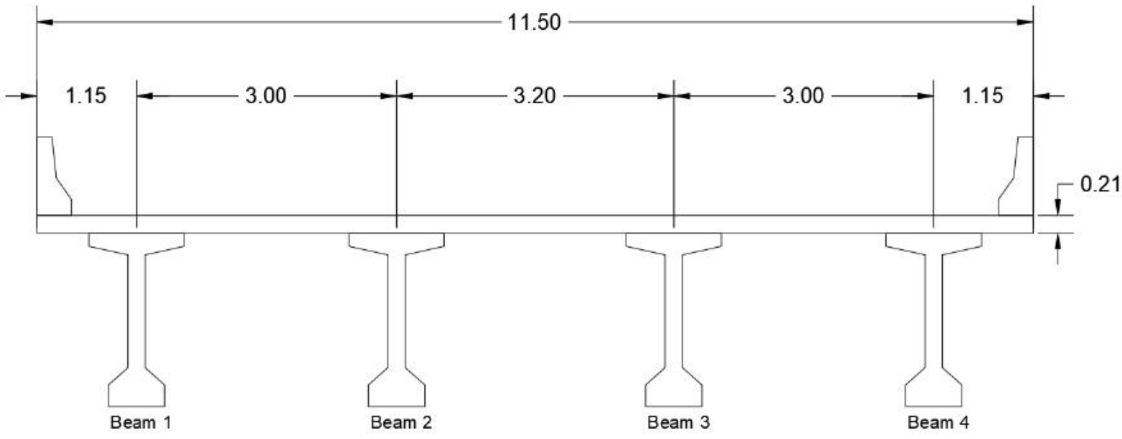

Fig.1 - Bridge cross section (dimensions in meters)

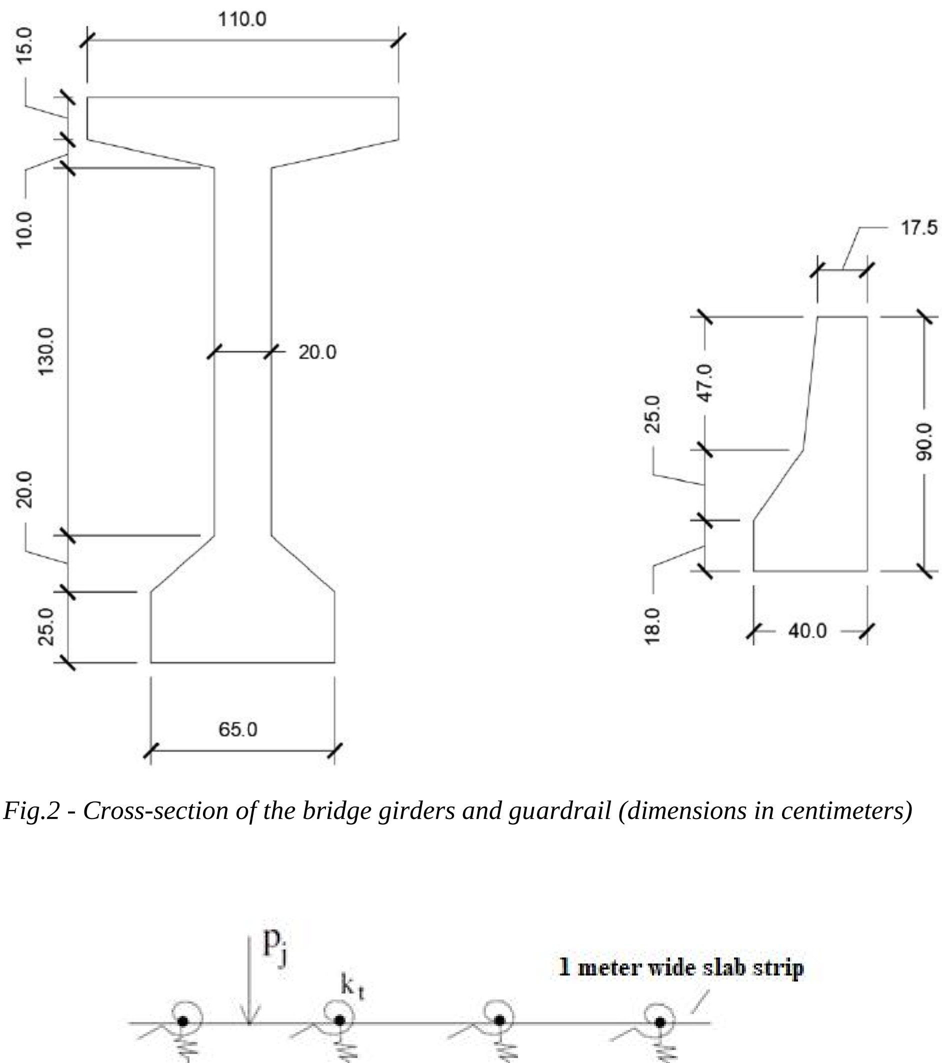

Fig.2 - Cross-section of the bridge girders and guardrail (dimensions in centimeters)

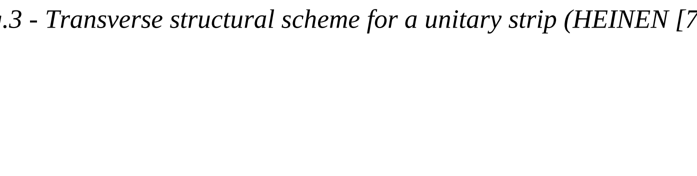

Fig.3 - Transverse structural scheme for a unitary strip (HEINEN [7])

Table 1 - Properties of bridge materials

Data	Symbol	Value	Unity
Beams concrete compressive strength	f_{ck}	50	MPa
Slab concrete compressive strength	f_{ck}	30	MPa
Characteristic tensile strength of prestressing steel	f_{ptk}	1860	MPa
Characteristic resistance to yielding of prestressing steel	f_{pyk}	1610	MPa
Prestressing steel modulus of elasticity	E_p	195	GPa
Steel crossectional area per strand	A	112	mm²
Characteristic value of maximum prestressing force	P_u	208	kN
Characteristic value of initial prestressing force	P_i	153	kN

Table 2 shows the values of the constants of the vertical springs (k_v) and the constants of the torsion springs (k_t), calculated with the formulas presented above, based on the T section previously shown. The parameters L = 27.4 m, f_{ck} = 50 MPa, E_{cs} = 29402.92 GPa, G = 12251.22 GPa, and the Poisson's coefficient equal to 0.2 are common to all the girders.

Figure 4 shows the beams representing the bridge to be analyzed, with the spring connections in place of the girders and their respective constants

2.1.3 Design of beam number 1 prestressing force

Table 3 presents the data used for the prestressing force design and the resulting number of strands for each case.

Table 2 - Values used in the Fauchart Method

Data	Beam 1 and 4	Beam 2 and 3
$I\ (m^4)$	0.7446	0.7772
$J\ (m^4)$	0.0135	0.0149
$k_v\ (10^3\ kN/m)$	3.784	3.949
$k_t\ (10^3\ kN.m/rad)$	2.176	2.400

Fig.4 - Beam representing the four girders bridge

Table 3 - Design of prestressing according to Brazilian codes

Description	Symbol	Brazilian codes	European codes
Number of prestressing strands	-	44 strands	52 strands
Distance from prestressing steel to the top of the beam	d_p	206 cm	201 cm
Total value of initial prestress force	P_i	6732 kN	7956 kN

2.2 RELIABILITY

The design based on technical codes applies the limit states format, which approximates the uncertainties in the design variables by using safety factors. The properties of materials, geometry, and loading are, in reality, random variables that can be specified through probability distributions. A structure is considered reliable if it performs the proposed function during the prescribed service life. The reliability analysis provides the probability of failure of the structure concerning a specified limit state. The probability of failure (p_f) and the reliability index (β) are related according to Equation 3.

$$\beta = \Phi^{-1}(1 - p_f) \quad (3)$$

Being Φ^{-1} the inverse standard normal distribution function. In this study, the reliability index (β) is calculated using the first-order reliability method (FORM) and validated using the Monte Carlo simulation method.

According to Beck [9], the FORM method calculates the reliability index β through the first-order approximation of the performance function. It converts all non-normal distributions into equivalent normal distributions at the point of failure *x** illustrated by Figure 5.

The FORM procedure consists of defining the limit state equation *G(x)*; assuming an initial point of failure *x**; then calculating, as a function of *x**, the corresponding mean ($\mu_{X_i}^N$) and standard deviation ($\sigma_{X_i}^N$) of each variable not normally distributed, calculating the partial derivatives of *G(x)* at point *x**; calculating the director cosines α_i; and getting the new failure point, according to Equation 4.

$$x_i^* = \mu_{Xi}^N - \alpha_i . \beta . \sigma_{Xi}^N \quad (4)$$

Finally, calculate the updated β value; and repeat the previous steps until convergence between β and *x** is reached.

The FORM requires the analysis to be carried out in the standard space. The statistical parameters of random variables with the most diverse probability distributions must be transformed into normal equivalents (BECK [9]).

The Monte Carlo method can be summarized in obtaining an estimate of the probability distribution of the response of a system by randomly generating the input parameters according to their probability distributions and performing N simulations of the system performance and then carrying out statistical analysis (MOURA [11]).

The reliability study for engineering structural systems basically consists of three possible responses of the performance function *G(x)*: in safety (*G(x) > 0*), limit state (*G(x) = 0*) and failure (*G(x) < 0*).

The probability of failure can be determined by Equations 5 and 6.

$$p_f = \frac{1}{N} . \sum_{i=1}^{N} I_g(x_i) \quad (5)$$

$$I_g(x) = \begin{cases} 0, if\ G(x) > 0 \\ 1, if\ G(x) < 0 \end{cases} \quad (6)$$

For the Monte Carlo analysis, the importance sampling method was used to accelerate the simulation convergence, adopting the probable failure points of each random variable provided by the FORM method (BECK [9]).

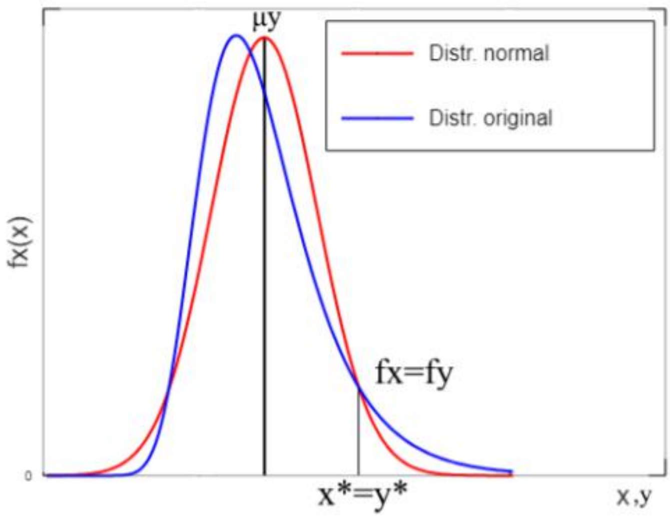

Fig.5 - FORM method for non-normally distributed variables (SILVA JÚNIOR [10])

2.2.1 Limit state equation

The limit state function used is defined by the difference between the resistance capacity and the sum of the loading actions and is mathematically expressed by Equation 7.

$$g(x) = R - S = 0 \quad (7)$$

For the use of the reliability method, a deterministic model that adequately represents the structure's response to be studied is essential. In this way, according to Moura [11], a mechanical model was developed, which calculates the resistant bending moment of cross-sections of prestressed concrete beams. The model developed can calculate the ultimate resistant bending moment of T sections.

In this work, the load-bearing capacity of prestressed concrete beams is defined by its resistant bending moment, M_r, multiplied by the model error estimate, θ_r, and the loading actions are represented by the sum of the bending moments caused by the permanent load, M_g, and the variable load, M_q, multiplied by a factor due to the uncertainties of the actions, θ_s, so that Equation 8 results.

$$g(x) = \theta_r \cdot M_r - \theta_s \cdot (M_g + M_q) = 0 \quad (8)$$

The calculation of the bending moments caused by the loading actions was carried out as follows. The bending moment due to permanent load is calculated according to Equation 9.

$$M_g = \frac{g \cdot l^2}{8} \quad (9)$$

Being *l* the design span, and *g* is the permanent load.

The moment due to variable load is calculated using the Fauchart Method, using the live load models recommended by the corresponding design code.

III. RESULTS AND DISCUSSIONS

3.1 Probability distributions and parameters of random variables

Using the Fauchart Method, implemented using the FTOOL software [12], the internal forces caused by the structure's self-weight and the live load models on beam number 1 were determined. The live load models TB-450 of the Brazilian code and the LM1 of the European standard were considered for the reliability analysis.

For the Brazilian live load model, the load train TB-450 was considered, as illustrated in Figure 6. It is defined by a three-axle vehicle, with a total load of 450 kN, evenly distributed on each wheel and surrounded by a constant uniform distributed load of 5 kN/m². It was also considered that the vehicle transits leaning against the guardrail, which is the most unfavorable position for the loading, to generate the greatest internal forces on the bridge.

The values of the live load applied to the level of the pavement are then multiplied by the Vertical Impact Coefficient (CIV), the Number of Lanes Coefficient (CNF), and the Additional Impact Coefficient (CIA), defined by Equations 10 to 12. An increase of 10% in the loading was also considered because the designed structure is placed at a distance of less than 100 km from port terminals.

$$CIV = \begin{cases} 1.35 & L_{iv} < 10m \\ 1.0 + \left(\dfrac{21.2}{L + 50}\right) & 10m \leq L_{iv} \leq 200m \end{cases} \quad (10)$$

$$CNF = 1.0 - 0.05 * (n - 2) > 0.9 \quad (11)$$

$$CIA = \begin{cases} 1.25 & for\ reinforced\ concrete\ structures \\ 1.15 & for\ steel\ structures \end{cases} \quad (12)$$

The European model consists of a tandem system (TS) and uniformly distributed load (UDL) in each lane with a maximum of three notional lanes. The two axles are 1.2 meters apart for each tandem system, with the wheels spaced at 2.0 meters, as shown in Figure 7. The axles should be centered in the lanes, and the lanes should be positioned to produce the most severe effect. The position of the UDL and TS longitudinally along the lanes should also be the most unfavorable. To this effect, it is not required to apply the UDL to the full width or span of a lane (DIETRICH [13]).

Table 4 presents the values for the axle loads and uniformly distributed loads (UDL) in each lane.

The self-weight of the structure generated a uniformly distributed load (*g*) on beam number 1 of 42.68 kN/m. The TB-450 mobile load model resulted in a total moment (M_q) of 3728.3 kN.m on the same equivalent part. The LM1 mobile load model resulted in a total moment (M_q) of 4194.1 kN.m.

For the bending moment due to the Brazilian live load model (M_q) a bias factor (λ) was considered, the ratio between the characteristic value and the mean, equal to 1.615, coefficient of variation equal to 0.18, these values were based on Lyra [14]. The live load was considered as obeying a probability distribution of Extreme Values of Type I (Gumbel).

For the bending moment due to the European live load model (M_q) a bias factor (λ) was considered equal to 1.885, a coefficient of variation equal to 0.18, and the distribution of Extreme Values of Type I (Gumbel) was adopted. These values were based on Caprani [15].

The mean, standard deviation, or coefficient of variation and the probability distribution law of the variables considered are contained in Table 5.

3.2 Reliability index

To determine the β reliability index, four different situations were adopted. The following cases were considered:

1. Bridge designed according to the Brazilian code and applied the TB-450 live load model for reliability analysis;
2. Bridge designed according to the Brazilian code and applied the LM1 live load model for reliability analysis;
3. Bridge designed according to the European code and applied the TB-450 live load model for reliability analysis;
4. Bridge designed according to the European code and applied the LM1 live load model for reliability analysis.

The reliability methods used in this study, FORM and Monte Carlo simulation, were both implemented using the PYTHON programming language.

Table 6 presents the results of the reliability index using the First Order Reliability Method (FORM) and the Monte Carlo simulation method.

The results achieved show that the values of the reliability indices calculated using the FORM method and the Monte Carlo simulation method are close, which confirms the validation of the models.

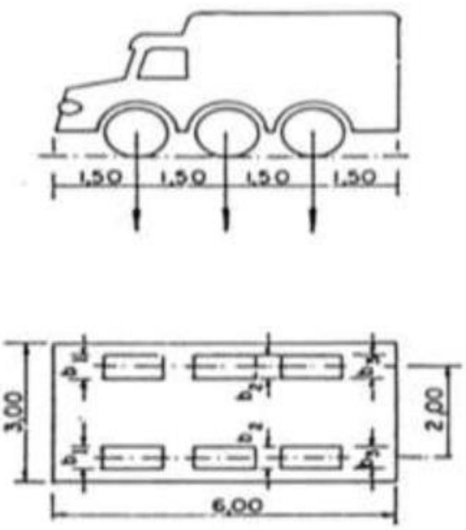

Fig.6 - Truck used in the TB-450, according to NBR 7188 [2]. Quotas in m.

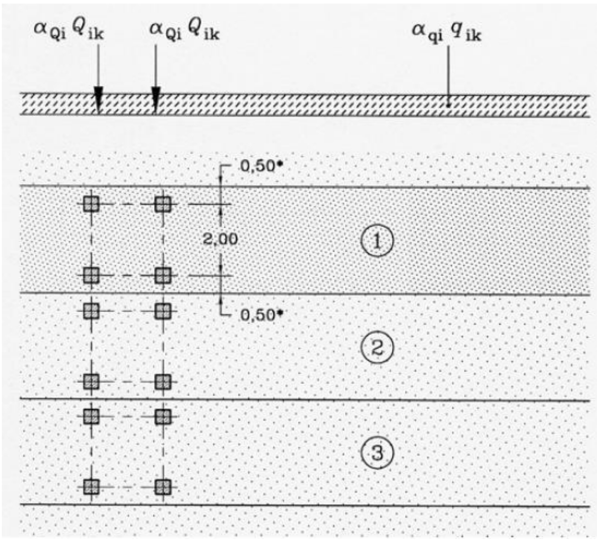

Fig.7 - Application of LM1(Eurocode 1 [3])

Table 4 - LM 1 - Characteristic values and adjustment factors (Eurocode 1 [3])

Description	Tandem System			UDL		
	Q_{ik} (kN)	α_{Qi}	$Q_{ik} * \alpha_{Qi}$ (kN)	q_{ik} (kN)	α_{Qi}	$q_{ik} * \alpha_{Qi}$ (kN)
Lane 1	300	1.00	300	9	0.61	5.5
Lane 2	200	1.00	200	2.5	2.2	5.5
Lane 3	100	1.00	100	2.5	2.2	5.5
Other Lanes	0	-	-	2.5	2.2	5.5
Remaining Area	0	-	-	2.5	2.2	5.5

Table 5 – Distributions for the random variables used in the reliability analysis

Variable	Mean μ	Standard deviation σ	Coefficient of Variation $V = {}^{\sigma}/_{\mu}$	Type of Distribution	Reference
f_c	$1.22 \cdot f_{ck}$	-	0.15	Normal	Santiago [16]
f_{pt}	$1.07 \cdot f_{ptk}$	-	0.015	Normal	Santiago [16]
A_p	$1.03 \cdot A_{p,nom}$	-	0.01	Log-Normal	Santiago [16]
d_p	$d_{p,nom}$	1.00	-	Normal	JCSS [17]
θ_r	1.044	0.092	-	Normal	San Martins [18]
θ_s	1.00	-	0.10	Log-Normal	JCSS [17]
g	$1.06 \cdot g_{nom}$	-	0.12	Normal	Santiago [16]
M_q	$M_{q,nom}/1.615$ or $M_{q,nom}/1.885$	-	0.18	Gumbel	Lyra [14] and Caprani [15]

Table 6 - Reliability index of the bridges studied

Method	Brazilian code designed bridge		European code designed bridge	
	TB-450	LM1	TB-450	LM1
FORM	5.60	5.68	6.20	6.28
Monte Carlo	5.63	5.71	6.25	6.33

Furthermore, it is possible to notice that the bridge designed according to the European codes resulted in safety levels higher than those obtained by the structure designed according to the Brazilian regulations. Even with such differences, very conservative values of the structure's safety were still obtained.

3.3 Analysis of sensitivity

A sensitivity analysis was performed by calculating the Importance Index, which is a function of the sensitivity factor and is defined by Equation 13 (NOVA [19]):

$$I_i = \alpha_i^2 \tag{13}$$

Sensitivity analysis makes it possible to determine the uncertainties that influence the failure event under investigation and then locate those variables that present greater effect in the response.

The sensitivity factor was obtained by implementing the FORM method, and it is presented in Table 7.

According to Table 7, the variables that most influence the probability of failure of the structure are the resistance error model, the bending moment caused by the live load,

Table 7 - Importance index for each random variable (%)

Variable	Brazilian code designed bridge		European code designed bridge	
	TB-450	LM1	TB-450	LM1
M_q	29.34	27.71	29.71	27.79
θ_r	41.99	43.32	44.63	46.36
θ_s	21.75	21.79	19.84	19.78

g	6.58	6.84	5.50	5.75
f_c	0.04	0.04	0.05	0.05
f_{pt}	0.04	0.04	0.04	0.04
d_p	0.05	0.05	0.05	0.05
A_p	0.21	0.21	0.19	0.19

and the loading error model. Therefore, the parameters of these variables should be determined very carefully.

IV. CONCLUSIONS

Using the First Order Reliability Method (FORM), the reliability index was determined, and the degree of safety of prestressed concrete bridges designed according to Brazilian and European codes was verified. The values obtained by the FORM method were validated through the Monte Carlo simulation method since they presented similar results.

It was possible to verify that structures designed according to European codes provide higher safety degrees than those designed according to Brazilian regulations. In addition, the LM1 mobile load model generates bending moments in the structure more elevated than those caused by the TB-450.

Through sensitivity analysis, it was shown that the random variables of the moment due to the moving load (M_q), the resistance model error estimate (θ_r) and the loading model error estimate (θ_s) have more significant contributions to the failure probability. On the other hand, the permanent load (g) has a smaller contribution to the probability of failure, and the other variables have minimal contributions.

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