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PROBABILISTIC ASSESSMENT OF AN EXISTING REINFORCED CONCRETE BRIDGE

Summary: This study presents a probabilistic assessment of an existing four-span reinforced concrete slab bridge, considering the stochastic variations of concrete strength and the yield strength of reinforcement steel. Although Eurocode permits probabilistic assessment of existing bridges, it provides limited practical guidance. Bending and shear limit-state functions are formulated using experimental material data to define probabilistic distributions. Other parameters remain deterministic or taken from literature. Results are also compared with traditional calculation using partial coefficients. Reliability is evaluated by estimating failure probability and safety margins for governing Ultimate Limit States (ULS). The performed calculation enables a more realistic assessment than conventional deterministic verification and may support decisions on continued use, maintenance or strengthening, with significant economic benefits for bridge owners.

Keywords: probabilistic assessment, structural reliability, limit state function, existing bridge

PROBABILISTIČKA PROCENA POSTOJEĆEG ARMIRANOBETONSKOG MOSTA

Rezime: Ova studija prikazuje probabilističku procenu postojećeg armirano betonskog mosta preko četiri polja, uzimajući u obzir stohastičke varijacije čvrstoće betona i granice tečenja armature. Iako Evrokod omogućava probabilističku procenu postojećih mostova, ne daje dovoljno praktičnih smernica za njenu primenu. Funkcije graničnog stanja za savijanje i smicanje formulisane su na osnovu izmerenih svojstava materijala. Ostali parametri tretirani su kao deterministički. Rezultati proračuna upoređeni su sa postojećim proračunom sanacije mosta koristeći parcijalne koeficijente. Pouzdanost je ocenjena određivanjem verovatnoće otkaza i sigurnosne rezerve za merodavna granična stanja nosivosti. Sprovedeni omogućava realniju procenu od konvencionalne determinističke provere i može doprineti donošenju odluka o daljoj upotrebi, održavanju ili ojačanju mosta, uz značajne ekonomske koristi za vlasnike mostova.

Ključne reči: probabilistička procena, pouzdanost konstrukcija, granična funkcija, postojeći most

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1. INTRODUCTION

Assessment of existing reinforced concrete bridges is important, especially for structures designed to older standards and exposed to growing traffic demands. Many bridges are approaching or exceeding their design life, requiring reliable evaluation of their condition and the limit for future use. Deterministic procedures based on characteristic values and partial safety factors [5], [6] may be conservative for existing structures. Also, probabilistic approach could bring significant savings to the Owner. Inspection results, material tests and knowledge of the actual condition can be incorporated in probabilistic methods [1]–[3]. This paper presents a probabilistic assessment of an existing reinforced concrete two-web slab bridge using measured concrete properties from field. Bending and shear limit states are analysed by FORM and Monte Carlo simulation. Bayesian updating is used to refine prior material model for concrete material data, while two probabilistic models are compared to evaluate two procedures related uncertainties.

2. OVERVIEW OF THE CONSIDERED BRIDGE AND PERFORMED IN-SITU TESTS

The analysed structure is an integral frame reinforced concrete bridge over four spans of 22 m, 28 m, 35 m and 25 m. The deck is 11.10 m wide and has two longitudinal webs. The flexural assessment refers to the positive-moment section in the 35 m span. The shear assessment refers to the support section between the 28 m and 35 m spans. Figures 1 and 2 and Table 1 record the supplied geometry without scaling the action effects.

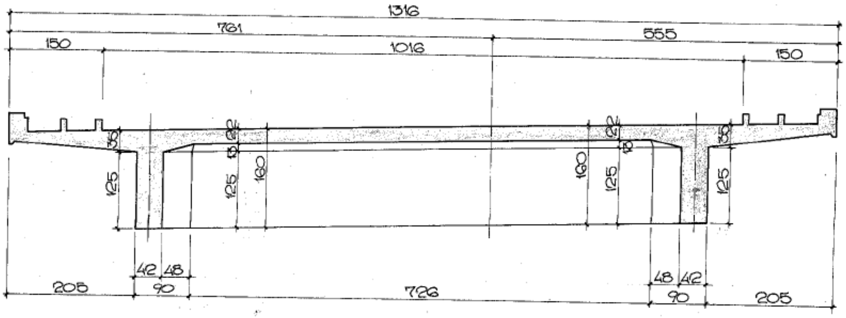

Figure 1: Typical cross-section of the analysed integral bridge

Figure 2: Present state of the girders

Table 1: Adopted geometric properties

		Midspan	At pier
b_{eff}	effective flange width	4850 mm	3665 mm
b_w	web width	420 mm	1000 mm
h_f	slab thickness	220 mm	220 mm
h	total section depth	1600 mm	2600 mm
d_s	effective depth of ordinary reinforcement	1397 mm	2495 mm
A_s	ordinary tensile reinforcement area	32576 mm²	18312 mm²
A_{sw}	shear reinforcement area	1232 mm²/m	1232 mm²/m
s	stirrup spacing	250 mm	250 mm

Concrete properties were determined through in-situ core sampling. Only six concrete cores were extracted from the bridge girders and tested in compression. Concrete compressive strength is assumed to follow a normal distribution. Measured compressive strengths are: 55.8, 59.1, 61.3, 54.0, 48.1, 54.3 MPa. Mean value and standard deviation are 55.43, 4.58.

The measured strengths are significantly higher than according to the initially taken design concrete class. This indicates that the actual concrete quality may be better than originally expected. However, due to the relatively small number of samples, the uncertainty associated with the statistical estimation remains significant. These are prior distributions marked as f_c^{prior}. Using Bayesian update, in further sections, these values will be used to introduces posterior information which will be incorporated in limit state function. The ordinary reinforcement is GA 240/360 with characteristic yield strength f_{yk} = 240 MPa. Following the JCSS-type model, yield strength is lognormal with mean 300 MPa and standard deviation 30 MPa [1].

3. PROBABILISTIC CALCULATION

3.1. BAYESIAN UPDATING OF MATERIAL PROPERTIES

Bayesian updating is used to combine prior engineering knowledge with newly obtained experimental data. The general Bayesian relationship is defined according to Bayesian probability theory and probabilistic reliability methods [1], [2], [3]:

$$p(\theta|D) = \frac{p(D|\theta)p(\theta)}{p(D)} \tag{1}$$

where:
$p(\theta)$ is the prior distribution,
$p(D|\theta)$ is the likelihood function,
$p(\theta|D)$ is the posterior distribution,
D represents the experimental data.

In practical applications, assumed material properties are updated using test results, while model uncertainties are adopted from JCSS [1]. Bayesian updating is used for concrete compressive strength and reinforcement yield strength, particularly when sample sizes are limited.

The prior concrete strength distribution is assumed as:

$$f_c^{prior} \sim N(X, 8^2) \tag{2}$$

The mean value will be assumed as uncertain, while the standard deviation (deterministic) σ_{f_c} is equal to 8 MPa. The uncertain mean value $\mu_{f_c}$was assumed normally distributed with known mean value $\mu' = 30\ MPa$ and standard deviation $\sigma' = 8\ MPa$. This is based on used class of concrete in existing design from 1977. The measured concrete strength samples are used for evaluating the mean value. Based on the test results the prior probabilistic model for the mean value of the yield stress can be updated using natural conjugate distributions [8]. In the case considered with a normally distributed variable with uncertain mean and known standard deviation the posterior may be found to be reduced according to proposed equations in [8]. Based on new observations the posterior parameters are μ'' =55.43 *and* σ''' =4.93. Also, the likelihood of the observation can be established. Figure 3 shows the prior, likelihood and posterior distributions together with the measured concrete core strengths. The prior model, defined by a mean of 30 MPa and a standard deviation of 8 MPa, reflects a much smaller value with considerable uncertainty compared to measured. Bayesian updating shifts the posterior mean to approximately 55 MPa and increases the standard deviation from 4.58 to about 4.93 MPa.[4] This standard deviation is actually the predictive probability. The predictive distribution is actually to predict strength at some other location and when small spatial variability is assumed one can take posterior distribution as the distribution of concrete strength (not just mean value). Actually, when the difference is so large between prior and posterior then one can actually ignore prior.

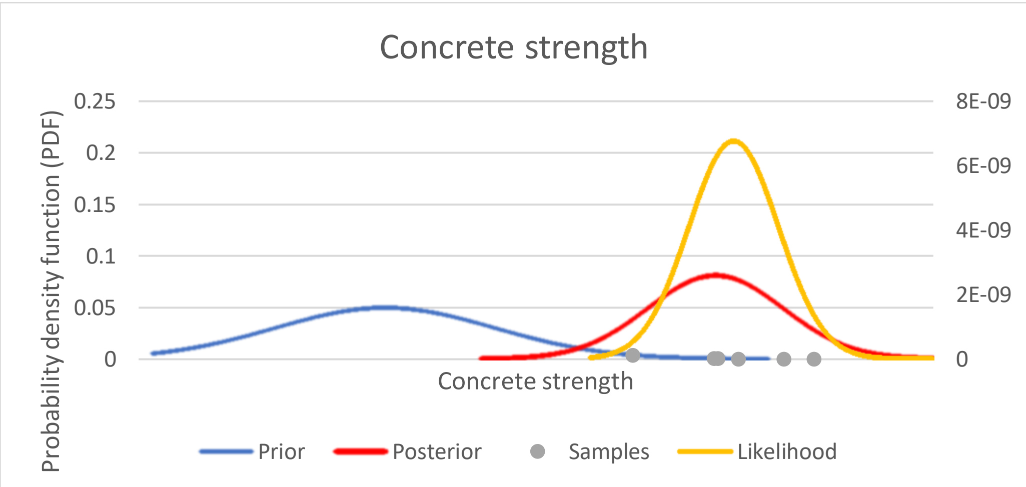

Figure 3: Prior, posterior and likelihood results for concrete strength

3.2. RANDOM VARIABLES

This section defines the material and geometric properties modelled as random variables in the reliability assessment. A specific probabilistic case is considered, reflecting different levels of uncertainty representation through the selection of random variables. These variables form the basis of the limit state functions and are subsequently employed in the FORM and Monte Carlo reliability analyses. Concrete compressive strength is represented by the normal distribution fc ~ N(55.43, 4.93^2) MPa. The corresponding lower 5% fractile is approximately 47.32 MPa; therefore, the Eurocode 2 rectangular stress-block parameters lambda = 0.80 and eta = 1.00 are adopted [6]. In the present study, a coefficient of variation of 0.08 was adopted for the bending resistance model uncertainty [1]. For shear, The fib Model Code for Concrete Structures 2020 (hereafter referred to as MC2020) represents the most refined model for calculating the shear strength of reinforced concrete elements. Following the enhanced procedure for corroded reinforced elements, larger coefficient of variation of 0.19 was adopted. Following random

[4] The posterior distribution of the mean value is equal to posterior distribution of concrete strength assuming that spatial variability is negligible. For further discussion on spatial variability see [12].

variables are considered: X = $\{f_c, f_y, Q, \theta_R, \theta_E\}$. Table 2 presents summarized variables and their distributions. At midspan, the mean permanent bending effect is M_G = 3416 kNm and the reference traffic effect is $M_{Q,ref}$= 2237 kNm. At the support, the corresponding values are V_G = 1633 kN and $V_{Q,ref}$= 670 kN.

Table 2: Random variables

Variable	Distribution	Mean	COV/SD	Source
f_c	Normal	54.11 MPa	4.93 Mpa	Bayesian update
f_y	Lognormal	300 MPa	30 Mpa	GA 240/360; JCSS
Q	Lognormal	1	COV 0.20	Literature
$\Theta_{R,M}$	Lognormal	1	COV=0.08	Flexural model uncertainty
$\Theta_{R,V}$	Lognormal	0.9506	COV=0.19	Shear model uncertainty
Θ_E	Lognormal	1	COV=0.10	Load-effect uncertainty

Permanent effects are considered as deterministic. The dimensionless traffic factor Q is lognormal with mean 1.00 and coefficient of variation 0.20 and multiplies only the reference traffic effects. The adopted distribution of Q is assumed to describe the maximum traffic factor over the selected 50-year assessment period. Normally, load effect distribution is described using Gumbel distribution, but Lognormal is also a possibility. Site-specific traffic data would be required to recalibrate Q for another period. All basic variables are assumed mutually independent.

3.3. LIMIT STATE FUNCTIONS

The structural reliability assessment is performed through limit state functions representing the difference between structural resistance and load effects. Failure occurs when: g (X) ≤ 0, while g (X) > 0 corresponds to a safe structural state. Two limits states are considered: bending resistance and shear resistance and two probabilistic models are analyzed. Random variables and deterministic values from previous section are used in following analysis.

$$g_M = \theta_{R,M} M_R(f_c, f_y) - \theta_E M_E(Q) \tag{3}$$

where:
M_R is the bending resistance,
M_E is the bending moment caused by loading, $M_E = M_G + QM_{Q,ref}$
For analysing the shear phenomena, the shear limit state function is:

$$g_V = \theta_{R,V} V_R(f_c, f_y) - \theta_E(V_G + QV_{Q,ref}) \tag{4}$$

3.4. RESISTANCE MODELS

Before performing the probabilistic reliability assessment, resistance models are established for both bending and shear limit states. To determine the resistance model, first the position of neutral axis has to be found. This is done by assuming that the neutral axis is in the slab. Since the calculation has shown the assumption as valid, the equation for resisting moment of section can be defined around compression block centroid :

$$M_R = A_s \cdot f_y \cdot \left(d_s - \frac{a}{2}\right) \tag{5}$$

The compression block depth is obtained from force equilibrium $A_s f_y = \eta f_c b_{eff} a$. The value for $a = 39.26\ mm$. Since $a < h_f = 220\ mm$ the neutral axis remains within the slab. When substituting all the values,deterministically the ultimate bending resistance is approximately

$M_R = 13461\ kNm$. This value represents the ultimate flexural resistance of the reinforced concrete section and forms the resistance side of the bending limit state function.

For defining the shear resistance model, the support-section geometry and stirrups amount are used, provided in Table 1. One group of 2Ø36 bars is taken to cross the critical shear crack at θ = 45 degrees. The resistance model for shear is evaluated in following way:

$$V_R = \min(V_{R,c} + V_{R,s} + V_{R,b}, V_{R,max}) \quad (6)$$

The shear reinforcement contribution is $V_{R,s}$; the concrete contribution is $V_{R,c}$; the contribution of diagonal bars $V_{R,b}$. According to MC2020 and recently published work [12], following expressions are used to cacluate suitable contributions:

$$V_{R,c} = k_v \cdot b_w \cdot z_v \cdot \sqrt{f_c} \quad (7)$$

$$V_{R,S} = \frac{A_{sw,v}}{s_v} \cdot z_v \cdot f_{yw,v} \cdot cot\theta \quad (8)$$

$$V_{R,b} = \frac{A_{sw,d}}{s_d} \cdot z_v \cdot f_{yw,v} \cdot (cot\theta + cot\alpha)sin\alpha \quad (9)$$

Direct addition of the three components is an assessment model, not the standard EC2 design equation for a member requiring shear reinforcement; The before mentioned value $\Theta_{R,V}$ in table 2, with COV 0.19 represents the associated model uncertainty. This value is calculated using JCSS calcualtion concept for model uncertainity and refined calculation procedure from [12] for uncorroded bars. Since the girder's effective depth considerably exceeds the maximum value for height included in experimental validation database from [12], the application of beforementioned procedure represents a scale extrapolation.

Shear models generally involve greater uncertainty than bending models because several mechanisms contribute to resistance [6], [7].

3.5. LOAD EFFECT MODEL

The load effect model consists of permanent actions and traffic actions. Permanent actions include the self-weight of the concrete girder, pavement layers and additional permanent loads. Traffic actions are represented by a stochastic load factor applied to a reference traffic load effect. In the probabilistic approach total bending moment is decomposed into permanent and traffic load contributions.

$$M_E = M_G + Q \cdot M_{Q,ref} \quad (10)$$

$$V_E = V_G + QV_{Q,ref} \quad (11)$$

where:

M_G, V_G is the permanent load bending moment and shear force,

$M_{Q,ref}, V_{Q,ref}$ is the reference traffic load moment and shear force,

Q is the traffic load random variable.

The adopted permanent line load is: $g = 23.0\ kN/m$. The maximum bending moment in the midspan of 35 m due to permanent load is: $M_G = 3416\ kNm$. The shear force at supports $V_G = 1633\ kN$.

For calculation of the traffic load, the vehicle V440 is taken from rehabilitation design project which is in acoordance with Serbian standards. The reference bending moment due to traffic load is: $M_{Q,ref} = 2237.0\ kNm$. The reference shear force due to traffic load is: $V_{Q,ref} = 670\ kN$. Traffic load uncertainty is introduced by a random traffic load factor modelled as: $Q \sim Lognormal(1.0, COV = 0.20)$. This formulation represents a simplified probabilistic traffic load model commonly used in reliability assessment when site-specific traffic data are not available [1], [3], [4].

3.6. RELIABILITY ANALYSIS

Traditional deterministic assessment methods use partial safety factors, as adopted in the Eurocodes [5], [6], to account indirectly for uncertainties in material properties, loads and modelling assumptions. This semi-probabilistic approach is practical and standardized but does not explicitly quantify structural failure probability. For existing bridges, standard partial factors may also be overly conservative because measured material properties and actual structural data are not directly considered. Probabilistic reliability analysis addresses these limitations by modelling uncertainties through random variables and probability distributions, allowing direct estimation of the probability of failure and reliability index. The final result of the analysis is the value of reliability index. This value is further compared to proposed standard values and conclude whether the structure fulfils the requirements.

The First Order Reliability Method (FORM) is an approximate probabilistic method used to estimate the failure probability and reliability index. Random variables are transformed into standard normal space, where the limit state is expressed as $g(\boldsymbol{U}) = 0$. For $g(\boldsymbol{X}) = R(\boldsymbol{X}) - E(\boldsymbol{X})$, the safe domain is defined by $g(\boldsymbol{X}) > 0$, while failure occurs when $g(\boldsymbol{X}) \leq 0$. The reliability index $\beta_{FORM} = min\sqrt{u_1^2 + u_2^2 + \cdots + u_n^2}$ is the shortest distance from the origin to the limit state surface, and corresponds to the most probable failure point, or design point. Its coordinates indicate the critical values and relative influence of the random variables. The failure probability is approximated as:

$$P_f = \Phi(-\beta_{FORM}) \tag{12}$$

Monte Carlo simulation is a probabilistic method used in structural reliability analysis to estimate failure probability through repeated random sampling of input variables. For each simulation, a set of random variables is generated and evaluated using the limit state function. Failure occurs when resistance is lower than the load effect, i.e. when $g(\boldsymbol{X}^{(i)}) \leq 0$. The probability of failure is estimated as $P_f \approx \frac{N_f}{N}$ where N_f is the number of failed simulations and N is the total number of simulations. The corresponding reliability index is

$$\beta_{MC} = -\Phi^{-1}(P_f) \tag{13}$$

4. CONVENTIONAL PARTIAL FACTOR METHOD CALCULATION

For comparison, the conventional partial safety factor method was implemented in order to evaluate the loads and effects on existing bridge structures and to decide if the strengthening is required. The new loads were calculated and compared to loads from original design from 1977. The partial factors used for comparison are taken from previous standard design used in Serbia, BAB 87. The assessment of existing bridge resulted in unsufficient factor values to those used for new structures, 1.51 to 1.6 for permanent load, and 1.75 to 1.80 for traffic loads. The shear assessment indicates that the existing transverse reinforcement is insufficient. The total design shear force is 3818.82 kN, of which 1970.48 kN must be resisted by the stirrups, whereas the existing 2U Ø14/250 mm stirrups provide only 1371.01 kN, resulting in a resistance deficit of approximately 599.5 kN. Therefore, additional shear strengthening is required.

5. DISCUSION AND COMPARISON

Considering the probability of failure in bending, Table 3 compares the reliability results obtained using FORM and Monte Carlo simulation. The design-point coordinates indicate the relative influence of material properties, loads and model uncertainties, with larger absolute values representing a stronger effect on structural reliability.

Table 3: Results for bending limit state function

Limit state	Method	Random variable	P_f	β
Flexure	FORM	f_c, f_y, Q, $\theta_{R,M}$, θ_E	1.138×10^{-6}	4.73
Flexure	Monte Carlo	f_c, f_y, Q, $\theta_{R,M}$, $\theta_{E,}$, N=2,000,000	1.263×10^{-6}	4.71
Shear	FORM	f_c, f_y, Q, $\theta_{R,M}$, θ_E	1.394×10^{-3}	2.99
Shear	Monte Carlo	f_c, f_y, Q, $\theta_{R,M}$, θ_E N=10^7	1.289×10^{-3}	3.01

In practical terms, the sensitivity factor evaluates how much each variable effects the reliability of structure and the design point is the most probable combination of random variables that causes the failure. These values are presented in Table 4.

Table 4: FORM design point for the flexural limit state

Variable	Design value x_d	Standard coordinate u_d	$\alpha = u_d/\beta$	α^2
f_c	54 MPa	−0.02810	−0.00594	1.87%
f_y	234.0 MPa	−2.44071	−0.51632	0.04%
Q	1.61609	2.5228	0.53368	8.09%
$\theta_{R,M}$	0.85108	−1.97888	−0.41862	70.29%
θ_E	1.27322	2.47139	0.52281	19.72%

The values are interpreted as follows: x_d is the design point in the physical space and represents the most probable combination of variable values at failure; u_d is the same point in the standard normal space. For example, u_d =2.528 means that the variable is about 2.899 standard deviations above its mean.; alpha is the sensitivity factor, indicating each variable's influence on reliability. Larger absolute values correspond to a greater contribution to failure probability. The sum of the squared sensitivity factors is equal to one. The largest flexural FORM sensitivities are associated with the traffic factor Q, the load-effect model factor θ_E and reinforcement yield strength, while concrete strength has only a minor effect. Concrete strength is negligible because the compression block remains thin and entirely within the slab. FORM and Monte Carlo results are similar, with small differences caused by the nonlinear limit state function and FORM linearization.

The design point values and their characteristics for shear limit state function are presented in the Table 5. Shear reliability is governed by the shear-resistance model factor $\theta_{R,V}$, followed by the load-effect model factor θ_E and the traffic factor Q. This confirms the big influence of model uncertainty in definition of shear resistance function. Taking into account corrosion of the reinforcement would additionally affect the results.

Table 5: FORM design point for the shear limit state

Variable	Design value x_d	Standard coordinate u_d	Alpha = u_d/β	α^2
f_c	54.11 MPa	−0.40858	−0.13664	0.00%
f_y	300.22 MPa	0.05751	0.01923	26.66%
Q	1.16044	0.85036	0.28438	28.48%
$\theta_{R,V}$	0.58243	−2.50694	−0.83838	17.52%
θ_E	1.13597	1.32792	0.44409	27.33%

At the design point, the load-related variables are higher than their mean values. Consequently, the most probable failure scenario combines a relatively low effective shear resistance with increased traffic and load-effect values. The alpha factor for model uncertainty indicates that the actual shear resistance is lower than the resistance predicted by the analytical model. The results are very sensitive to adopted COV for model uncertianity $\theta_{R,V.}$ The calculated reliability indices should also be considered in relation to the target reliability levels recommended in relevant standards and guidelines. According to fib Bulletin 80, the target reliability level for an existing structure should be selected by considering the consequences of failure, the remaining working life, the available information and the relative cost of safety measures. Consequently, lower target values than those adopted for the design of new structures may be justified; however, no single target value is universally applicable to all existing bridges. For comparison, EN 1990 recommends a target reliability index of approximately $\beta t = 3.8$ for ultimate limit states, a 50-year reference period and reliability class RC2, which is commonly applicable to structures with medium consequences of failure, and this is referred only to buildings. Main bridges may require classification in a higher consequence or reliability class, depending on their importance and the consequences of collapse. On the other hand, the Swiss standard SIA 269 provides precise guidelines for evaluation of existing bridges. In the present study, FORM gives $\beta = 4.73$ and $P_f = 1.138 \times 10^{-7}$, close results to MC simulation. For shear, the main assessment model, including the concrete contribution, gives $\beta = 2.99$ and $P_f = 1.394 \times 10^{-3}$ by FORM and beta approximately 3.01 and P_f 1.289×10^{-3} by MC. These values for shear limit state are below the target levels, which indicates that either the resistance model need further refinement or the existing bridge girder needs strengthening. Furthermore, in this assessment of an existing bridge, measured data were available only for the concrete compressive strength. More accurate results could be obtained if the current mechanical properties of the reinforcing steel were also determined experimentally. The reliability index obtained for flexure indicates that the structure remains within the safe domain and that no additional flexural strengthening is required. This conclusion differs from that of conventional calculation, which was based on a semi-probabilistic analysis using partial safety factors and indicated that strengthening with externally bonded carbon-fibre-reinforced polymer strips was necessary. The comparison demonstrates that, for the considered flexural limit state, a probabilistic assessment of the existing structure leads to considerably less conservative results. This difference arises primarily from the possibility of updating the probabilistic model using measured data and explicitly accounting for the uncertainties associated with resistance and load effects. Consequently, probabilistic assessment can also have a significant influence on the economic efficiency of structural rehabilitation. A study by Tošić and Hajdin [10] reported an average flexural reliability index of 4.92 for structures designed according to the former Serbian regulations. This value is very close to the flexural reliability index obtained in the present assessment. On the other hand, when observing shear reliability index, a higher difference is present: 2.99 to 4.42 average. Although the Eurocodes provide a general framework for probabilistic analysis, the guidance remains predominantly conceptual and offers limited detail for its practical implementation in the assessment of existing structures.

6. CONCLUSIONS

The following conclusions can be drawn:

- The flexural assessment produced consistent results using FORM $\beta = 4.73$, $P_f = 1.138 \times 10^{-6}$ and Monte Carlo simulation $\beta = 4.71$, $P_f = 1.263 \times 10^{-6}$. The high flexural reliability index indicates that the bridge is within the safe domain and that additional flexural strengthening is not required.
- Compared to semi-probabilistic, the more favourable probabilistic result obtained for flexure can primarily be attributed to the larger resistance reserve, the use of updated mean material properties and the relatively low uncertainty of the flexural resistance model. In contrast, shear is characterized by a brittle and considerably more uncertain resistance mechanism, represented by a resistance-model coefficient of variation of 19%. Consequently, the probabilistic and conventional assessments lead to different levels of conservatism for flexure but converge towards the same conclusion of insufficient safety for shear.
- Although the Eurocodes establish a general probabilistic framework, more detailed guidance is needed for its practical application to existing bridges.

- This calculation is only valid if there is no deterioration on the structure.

A probabilistic approach can provide a more realistic and better-informed evaluation of existing structures by incorporating structure-specific measurements and explicitly quantifying uncertainty. Its principal disadvantage is the need for field investigations and material testing, which require specialized equipment and additional initial expenditure. Nevertheless, when the overall scope and cost of rehabilitation are considered, the resulting savings may be substantial, particularly when unnecessary strengthening measures can be avoided.

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