



**HAL**  
open science

# Adjustment of sensitivity factors for the assessment of reinforced concrete Open Frame underpass short span bridge

Midula Alam, F. Lavergne, Silvia Ientile, André Orcesi, Franziska Schmidt

## ► To cite this version:

Midula Alam, F. Lavergne, Silvia Ientile, André Orcesi, Franziska Schmidt. Adjustment of sensitivity factors for the assessment of reinforced concrete Open Frame underpass short span bridge. 15th fib PhD Symposium, fib (Fédération international du béton), Aug 2024, Budapest, Hungary. pp.325-332. hal-04774686

**HAL Id: hal-04774686**

**<https://hal.science/hal-04774686v1>**

Submitted on 20 Jan 2025

**HAL** is a multi-disciplinary open access archive for the deposit and dissemination of scientific research documents, whether they are published or not. The documents may come from teaching and research institutions in France or abroad, or from public or private research centers.

L'archive ouverte pluridisciplinaire **HAL**, est destinée au dépôt et à la diffusion de documents scientifiques de niveau recherche, publiés ou non, émanant des établissements d'enseignement et de recherche français ou étrangers, des laboratoires publics ou privés.

# Adjustment of sensitivity factors for the assessment of reinforced concrete Open Frame underpass short span bridge

Midula Alam<sup>1</sup>, Francis Lavergne<sup>2</sup>, Silvia Ientile<sup>1</sup>, André Orcesi<sup>2</sup>, Franziska Schmidt<sup>2</sup>

<sup>1</sup>EMGCU, MAST

Gustave Eiffel University

14-20 Boulevard Newton, Cité Descartes, Champs-sur-Marne (77420), France

<sup>2</sup>Dtech ITM

CEREMA

8 Boulevard Newton, Champs-sur-Marne (77420), France

## Summary

Structural assessment can be performed using semi-probabilistic methods, as is the case in design codes. However, these codes are primarily calibrated for the design of new structures and do not account for existing structures parameters, which need to be determined more precisely. To adapt the formalism of these codes, adjustment of partial factors is possible through the updating of the distribution of random variables. Challenges in reliability analysis of existing structures notably include data collection and the comprehensive analysis of structures. This study focuses on a family of small-span reinforced concrete bridges, known as Open Frame Underpasses (OFU), designed with the CHAMOA-P calculation model. This study particularly analyses the adjustment of partial factors using sensitivity factors obtained by reliability calculation rather than the default values used in the Eurocodes. Although this approach is intended to provide a more rigorous evaluation for the selected OFU existing structures, it presents challenges related to the analysis of these structures. The effectiveness of the parameter adjustment is evaluated by comparing the results with case studies and existing codes.

## 1 INTRODUCTION

Adjusting partial factors (PFs) is crucial for assessing the reliability of existing structures. Although safety margins in Eurocode 2 typically present no issue at the design stage, they can lead to significant economic and environmental consequences during assessment of existing structures, especially if specifications are overly conservative. Excess material usage of 5 to 10% at the design stage is usually inconsequential, but failing to meet certain criteria in existing structures might necessitate costly rehabilitations or demolitions. For this reason, refining models based on empirical data [1] and advanced evaluation methods are needed [2].

This study proposes a framework compliant with Eurocode standards [3]–[4] (NF EN 1991-2, 2004 [5]; NF EN 1992-1-1, 2005 [6]–[7]) and TS 17440 [8], employing engineering software for comprehensive structural modelling. It aims to optimize PFs by adjusting uncertainty levels and targeting specific reliability standards [9], focusing on Open Frame Underpasses (OFU) bridges, a prevalent type of reinforced concrete structure in France known for their robustness [10]. The approach assesses the impact of sensitivity factors on various random variables, comparing standard Eurocode values against those derived from reliability analyses. This method enhances structural evaluations and contributes to sustainable infrastructure management by applying theoretical advancements in structural reliability to practical scenarios.

## 2 SYSTEMATIC ADJUSTMENT OF SAFETY COEFFICIENTS USING FORM

The adjustment of PFs can be based on the reliability concepts described in the First Order Reliability Method (FORM) [11]–[12]. In addition, the Model Code 2020 (MC2020) [10], [13] takes these elements and reduces the probability laws and partial coefficients according to the assumptions of the

Eurocode to propose updated PFs [3]–[7]. The partial coefficients are calculated as the ratio between a quantile (e.g. at 5%) and the working point determined by the FORM method (e.g. for materials in (1)).

$$\gamma_X = \frac{X_k}{X_d} \quad (\text{for resistance variables}) \quad (1)$$

where  $\gamma_X$  is the partial coefficient of the random variable  $X$ ,  $X_k$  a fractile (e.g. at 5%) of the random variable  $X$  and  $X_d$  is the operating point of the random variable  $X$ .

In accordance with the prevailing methodology, PFs may be adjusted through two distinct approaches. The first approach considers the uncertainty variables separately, resulting in the determination of PFs for each source of uncertainty [12]. A second approach involves the use of a single variable encompassing several sources of uncertainty, via an overall uncertainty coefficient [4], [12]. In this case, a distinction is made between uncertainties in resistance and uncertainties in stress. These two approaches are based on different methods for adjusting the PFs to take account of the uncertainties in structural safety calculations [13]. In the case of material variables, the PFs can be expressed according to equation (2). The separate PF approach will be used in the present study.

$$\gamma_M = \gamma_{Rd} \times \gamma_m = \gamma_{Rd1} \times \gamma_{Rd2} \times \gamma_m \quad (2)$$

with  $\gamma_m$  the partial coefficient on the physical characteristics of a material,  $\gamma_{Rd}$  the overall partial coefficient on uncertainties,  $\gamma_{Rd1}$  the partial coefficient allowing model uncertainties to be considered and  $\gamma_{Rd2}$  the partial coefficient to take account of geometric uncertainties.

Table 1 shows the calculation of CP for different types of random variables.

Table 1 Details of the equations for calculating partial safety factors using FORM as presented in *fib* Bulletin 80 [12]

PF	Formula	Distribution	Comment
$\gamma_m$	$\frac{\exp(-1,645V_X)}{\exp(-\alpha_R\beta V_X)}$	LN	-
$\gamma_g$	$\frac{(1 - \alpha_S\beta V_G)}{(1 + k V_G)}$	N	k=0 for current expenses
$\gamma_q$	$\frac{1 - \frac{0,5772}{1,282} V_{Q,50} - \frac{V_{Q,50}}{1,282} \ln\{-\ln[\Phi(-\alpha_S \times \beta)]\}}{1 - \frac{\ln(2) - \ln[-\ln(0,90)] - 0,5772}{1,282} \times V_{q50}}$	G	50-year reference period and 1000-year return period
$\gamma_{Rd}$	$\frac{1}{1 - \alpha_{ND}\alpha_R\beta V_{\theta_R}}$ OR $\frac{1}{\exp(-\alpha_{ND}\alpha_R\beta V_{\theta_R})}$	LN	without bias
$\gamma_{Rd}$	$\frac{\exp(-1,645 V_{\theta_R})}{\exp(-\alpha_{ND}\alpha_R\beta V_{\theta_R})}$	LN	-
$\gamma_{Sd}$	$1 - \alpha_{ND}\alpha_S\beta V_{\theta_S}$	N	-

PFs can be adjusted by modifying various parameters such as: (i) the coefficient of variation when more accurate data is available, thereby reducing uncertainties (measurements, geometry and method), (ii) the target reliability index, which reflects the expected level of structural safety [15] and (iii) the sensitivity factors, which account for the weight of random variables in the calculation model. As the PFs are defined in relation to the simplified expression of  $X_k$  and  $X_d$ , they depend on sensitivity factors. For PFs as defined in the Eurocodes [3]–[4], the sensitivity factors were initially determined using the FORM approach based on the values of CoV (Coefficient of Variation) for the variables of resistance and load considering identified reference periods. For a reference period of 50 years, fixed values of  $\alpha_S = -0.7$  and  $\alpha_R = 0.8$ , along with a multiplying coefficient  $\alpha_{ND} = 0,4$  of  $\alpha_S$  and  $\alpha_R$  for secondary variables (e.g. for uncertainties), are used to describe the sensitivity factors [16]. The objective of the subsequent phases of this research is to evaluate the influence of selected sensitivity factors on the adjustment of PFs.

### 3 FRAMEWORK FOR MODELING AND ANALYSIS OF REINFORCED CONCRETE BRIDGES

#### 3.1 Establishing the Structural Model for Bridges

Updating the partial factors, particularly by adjusting the sensitivity factors, necessitates the creation of a specific model for the structures under study, here OFU bridges [10]. The design and assessment of structures requires the use calculation software as Cerema's Chamoia-P calculation chain [17]. In Chamoia-P, the frame is represented by a planar model in which the crossbeam, uprights, and footings are represented with a bar model. The effect of transverse distribution is accounted for using the Guyon Massonnet method. The loads in the sections are evaluated as superpositions of weighted linear elasticity solutions to obtain the combinations described by the Eurocodes and their French annexes [3]–[7]. On the basis of these loads, Chamoia-P [17] calculates the cross-sectional area to assess the quantity of steel required to ensure that the structure complies with the criteria defined by the Eurocodes for ULS (Ultimate Limit State) and SLS (Serviceability Limit State). Chamoia-P relies on the bar calculation software ST1v2.23 [18] which is used here to evaluate the loading forces as presented in Fig. 1. Given a set quantity of steel, the numerical application for section calculation, CDS 6.35 [19], is utilized to draw the N-M interaction diagram of the section at each limit state, thus describing the domain of forces the section can withstand.

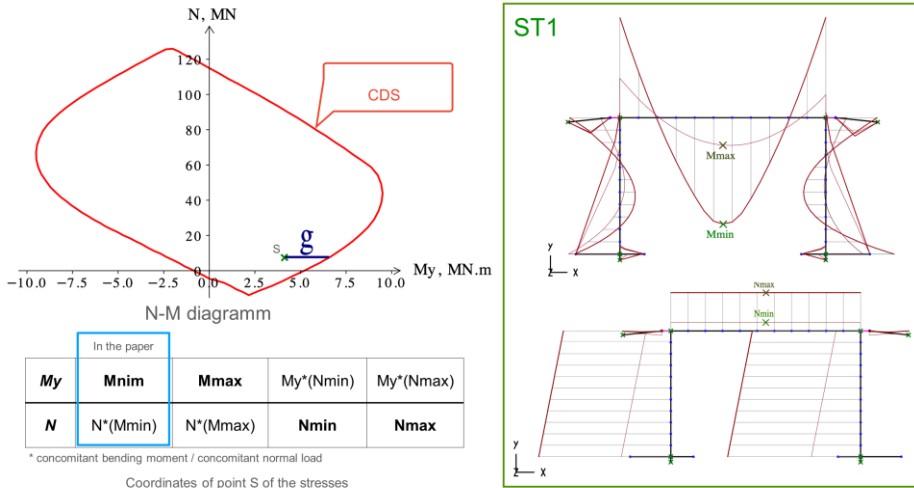


Fig. 1 Schematic diagram of the limit state function  $G$

The limit state function for this study is derived from the N-M interaction diagrams of the section located at mid-span. For each limit state, precise knowledge is obtained of the combinations of normal forces (N) and bending moments (M) required for the section to meet the resistance criteria. When the applied normal force is within acceptable limits, the corresponding resisting force is identified on the interaction diagram as the point with the same normal force and a moment having the same sign as the applied moment (Fig. 1) [20]. Given a specific set of random variables (e.g., steel strength, concrete cover thickness, material density, traffic intensity, etc.), the comparison between the loading force and the resisting force (denoted as  $g$ ) facilitates the definition of the limit state function (denoted as  $G$ ). This function is positive if the structural safety is sufficient and negative if it is inadequate. The limit state function  $G$  is formulated as equation (3).

$$G(X) = M_R(X_R, N_S) - M_S(X_S, N_S) \quad (3)$$

where  $G > 0$  : in the field of safety and  $G < 0$  in the field of failure with  $G$  the limit state function,  $g$  the value of  $G$  for a given print run,  $X$  the vector of random variables,  $M_R$  the moment of resistance of the interaction diagram determined from CDS 6.35 [19],  $X_R$  the vector of random variables linked to resistance,  $M_S$  the soliciting moment associated with the point S coordinates  $(M_{y_S}, N_S)$  of the section subjected to the most critical loads, determined by ST1v2.23 [18],  $X_S$  the vector of random variables

linked to the stresses. The failure probability, the design point and the sensitivity factors  $\alpha_S$  and  $\alpha_R$  can be estimated with the RSM (Response Surface Method) [21]. It is thus possible to estimate the probability of failure associated with the limit state G. The related design point and the sensitivity factors  $\alpha_S$  and  $\alpha_R$  are also determined.

The study was conducted at the fundamental ULS. Four failure modes are defined for each of the limit states: (i) bending moment failure under maximum moment with tension of the lower fibres of the span (Mmin), (ii) bending moment failure under moment with the lower fibres of the span in compression (Mmax), (iii) bending moment failure associated with the simultaneous moment at the maximum or minimum axial force (Nmax and Nmin). The principal mode of failure is bending moment failure at maximum moment with the lower fibres of the span in tension (illustrated in blue in Fig. 1), which is the case that will be discussed in the following sub-section. This preliminary study was conducted on a limited number of structures.

### 3.2 Structural Characteristics of Bridges Under Study

The four structures studied share several characteristics. The urban profiles, U10 and U15, feature openings of 10m and 15m, respectively with a 7.50m carriageway, including a 2cm waterproof layer and a 10cm asphalt mix, alongside two 1.25m pavements. These roads are designed for LM1, LM2, and LM4 type traffic in traffic class 2, excluding exceptional or military convoys. Additionally, two highway profiles, H10 and H15, also with 10m and 15m openings respectively, include a 0.5m guardrail, an 11m carriageway, and a 0.8m BN4 with a transition slab, accommodating traffic types LM1 and LM2 in traffic class 1, as well as MC120 convoys. Each structure has a free height of 5.5m, a 1.5m plug in the ground, a pedestal height of 6.4m, and 60cm thick footings. Reinforcement details are specified in Table 2. Steel quantities at mid-span were calculated using Eurocode coefficients, confirming that the primary design limit state corresponds to LM1 traffic at the ULS.

Table 2 Details of reinforcement for structures

Structure	Dimension	U10	U15	H10	H15
Steel below mid-span	-	67 HA25	100 HA25	113 HA25	88 HA32
Slab thickness	[m]	0.45	0.6	0.5	0.65
Upper steel at mid-span	-	67 HA8	100 HA8	113 HA8	88 HA8
Spacing	[cm]	15	10	10	14,3

The structures are modelled on loose, normally consolidated soil whose properties are as follows: The  $E_{Ménard}$  value is 100 MPa. The limit pressure is 1 MPa, and a rheological parameter accounting for soil deformability of 0.5. It is assumed that the exposure class is XC2. Furthermore, a thermal effect representative of the Seine et Marne, in accordance with EN1991-1-5/NA [5], is considered, with a dark coating. The maximum uniform temperature is 30°C, while the minimum uniform temperature is -27°C. The positive gradient is 18°C, while the negative gradient is -6°C. In this study, temperature is not considered a random variable.

### 3.3 Defining Random Variables for the Structural Analysis

In the context of reliability calculations based on case studies and the literature, dominant random variables with significant sensitivity factors have been identified [2], [12], [13], [22]. It is crucial to limit the number of random variables to those that have the greatest impact on the variability of the assessment result. The calculation method employed in the study is that presented in the Eurocodes [3]–[7]. Consequently, the uncertainties defined in the Eurocodes may be applied.

The variables under consideration for the study are presented in Table 3. These include  $f_c$ , the compressive strength of the concrete at 28 days;  $f_y$ , the tensile strength of the steel;  $\Delta c$ , excess or defective coating;  $\rho_{RC}$ , the density of the concrete; and  $q_1$ , the distributed loading associated with LM1 for traffic class CC2 on lane 1. The model uncertainty on the compressive strength of the concrete at 28 days is denoted by  $\theta_{Rd,c}$ , while the model uncertainty on the tensile strength of the steel is represented by  $\theta_{Rd,s}$ .

Similarly,  $\theta_{sd,g}$  represents the model uncertainty in the dead weight, while  $\theta_{sd,q}$  denotes the model uncertainty on traffic loads at LM1. Given that the objective of the approach was to address structures designed in accordance with less rigorous regulations or affected by pathologies such as corrosion, the strength of the steels was deliberately reduced. The structures were designed to withstand using class B500B reinforcement, with a characteristic yield strength of  $f_{yk}=500$  MPa. It is assumed here that the elastic limit of the degraded steel has an average value of 375 MPa. In the model, the weighting coefficient  $q_1$  is employed to scale the other components of LM1. For a structure of traffic class CC2, the weighting coefficient for the distributed load on lane 1 is  $a_{q1,EN}=0.7$ , and the distributed load on lane 1 is  $q_{1,EN}=9\text{kN/m}^2$ . The weighting coefficient is then calculated as  $a_{LM1} = \frac{q_1}{0.7 \times 9}$ .

Table 3 Details of the random variables considered and the related parameters as the dimension, the distribution law, the mean value, the standard deviation, the Coefficient of Variation (CoV) and the source where the random variable's parameters were calibrated

Variable	Dimension	Distribution	Mean	Standard deviation	CoV	Source
$f_c$	[MPa]	Lognormal	36	5,4	15%	[22]
$f_y$	[MPa]	Lognormal	375	11,7	4,5%	[9][13]
$\rho_{RC}$	[kN/m]	Normal	25	2	8%	[13]
$q_{LM1}$	[kN/m <sup>2</sup> ]	Gumbel	7,9	0,61	7,8%	[23]
$\Delta c$	[m]	Normal	0	0,005		[12]
$\theta_{Rd,c}$	-	Lognormal	1,2	0,18	15%	[22]
$\theta_{Rd,s}$	-	Lognormal	1	0,05	5%	
$\theta_{sd,g}$	-	Normal	1	0,06	6%	
$\theta_{sd,q}$	-	Normal	1	0,08	8%	

A probabilistic analysis of the structures is conducted using the response surface method (RSM) [21]. RSM is employed to develop surrogate models that approximate the complex behaviour of systems, thereby facilitating iterative reliability predictions [21], [24]. A linear response surface is employed to explore the linear relationships between the independent variables and the dependent variable(s), without consideration of complex interaction effects or non-linearities (such as quadratic terms). The response surface algorithm approximates a plane orthogonal to the calculation point that is closest to the system's actual operating point. The selection of a linear response surface is consistent with the FORM model, which represents an orthogonal plane through the operating point. This approach aligns with the underlying assumptions considered in the formulae presented in Table 1.

In order to efficiently traverse the space of variables, a method called experimental design is employed to determine the most relevant sets of variable values to assess the impact of variables on a response. This optimises the information obtained while minimising the resources used. The experimental design proposed in the ReliabTbx toolbox [25] include the star design, the modified star design and the central composite experimental design. A star experimental design comprises  $2n+1$  experiments conducted around a calculation point, which enables the estimation of the differential of the limit state function  $G$  at the point of study to be centred. The RSM algorithm employs an iterative process to identify a new point that is more closely aligned with the limit state. The selection of the star plane is sufficient to provide an adequate approximation of the target reliability index in the case of a linear response surface. The criterion of convergence is set such that  $\Delta\beta=0.01$ .

#### 4 RESULTS AND DISCUSSION ON ADJUSTMENTS TO SAFETY COEFFICIENTS

On the basis of the variables considered, it is possible to perform reliability analyses for each of the structures. The reliability indices are presented in Table 4, along with the related operating points obtained for the four structures, shown in Table 5.

Table 4 Reliability indices of the studied structures determined using RSM

Reliability	H10	H15	U10	U15
$\beta$	3.7	3.7	2.8	3.8
$p_f$	0.0094%	0.010%	0.28%	0.0070%

Table 5 Design points of the studied structures determined using RSM

Variable	Dimension	H10	H15	U10	U15
$f_c$	[MPa]	34.05	34.48	34.26	33.41
$f_y$	[MPa]	353.49	355.03	358.53	351.72
$\rho_{RC}$	[kN/m]	25.59	25.75	25.80	26.54
$q_1$	[kN/m <sup>2</sup> ]	10,73	10.53	9.34	10.45
$\Delta c$	[m]	0.0019	0.0013	0.0015	0.0006
$\theta_{Rd,c}$	-	1.14	1.15	1.15	1.15
$\theta_{Rd,s}$	-	0.94	0.93	0.95	0.93
$\theta_{Sd,g}$	-	1.01	1.02	1.01	1.04
$\theta_{Sd,q}$	-	1.09	1.11	1.10	1.09

The results of the reliability indices for structures H10, H15 and U15 are all within a narrow range of the target reliability index of 3.8. Given that the sensitivity factors are contingent upon the reliability index, the rest of this study will concentrate only on these three structures, for the sake of consistency.

The structural designs examined across the four cases demonstrated considerable uniformity at their respective design points. Although variations in reliability indices were evident (Table 4), the robustness and reliability of the computational model could be discerned, suggesting a high degree of predictability in load response behaviours for the OFU structures. Furthermore, by analysing the obtained sensitivity factor values, an average sensitivity factor for three of the four structures under consideration is determined (see Fig. 2). The sensitivity factor analysis performed on the H10, H15, U10, and U15 bridge configurations facilitates the identification of key variables that influence the structural response to loading.

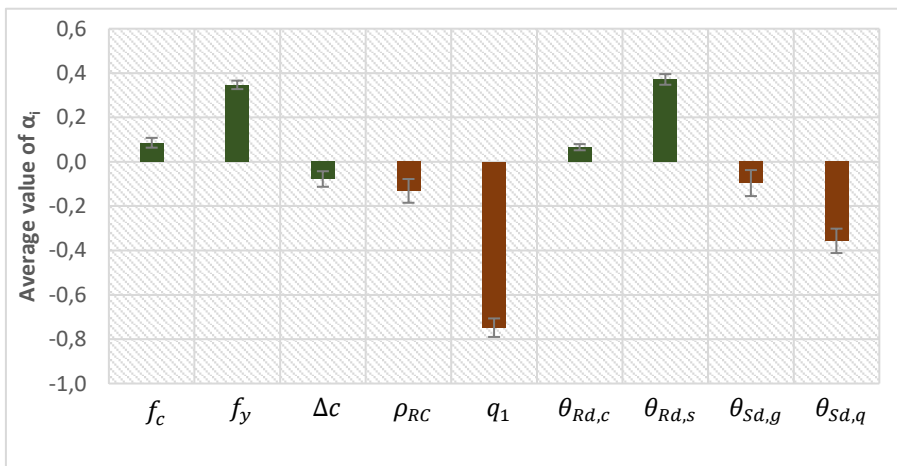


Fig. 2 Average values of the sensitivity factors at the fundamental ULS Mmax at mid-span (resistance variables in green and load variables in red)

Subsequently, two distinct methodologies are outlined. Initially, the sensitivity factors of the nine random variables are categorized into strength and stress groups to determine  $\alpha_R$  and  $\alpha_S$  according to equation (4).

$$\alpha_R = \sqrt{\frac{1}{n} \sum_{i,j} \alpha_{R,ij}^2}; \quad \alpha_S = -\sqrt{\frac{1}{n} \sum_{i,j} \alpha_{S,ij}^2} \quad (4)$$

where  $\alpha_{R,ij}$  is the sensitivity index obtained for the structure  $i$  for the variable associated with resistance  $j$ ,  $\alpha_{S,ij}$  is the sensitivity index obtained for the structure  $i$  for the variable associated with stress  $j$  and  $n$  the number of studied structures.

This approach reduces the limit state to a fundamental one of R-S. The sensitivity factors, determined from (4), are as follows:  $\alpha_S = -0.85$  and  $\alpha_R = 0.52$ . These values diverge slightly from the Eurocode assumptions ( $\alpha_S = -0.7$ ,  $\alpha_R = 0.8$ ). The PFs for the first two dominant variables ( $q_1$ ,  $\theta_{sd,q}$ ,  $f_y$  and  $\theta_{Rd,s}$ ) are calculated with the new coefficient by applying the multiplication coefficient  $\alpha_{ND} = 0.4$  for the secondary variables ( $\beta_t = 3.8$ ), resulting in  $\gamma_Q = 1.23$  and  $\gamma_S = 1.06$ .

A second approach (approach 2) identifies a dominant variable,  $q_1$ , with a coefficient of -0.75, and three secondary variables,  $f_y$ ,  $\theta_{Rd,s}$ , and  $\theta_{sd,q}$ , with coefficients of 0.35, 0.37 and -0.33 respectively. These results are derived from the reliability calculation presented in Fig. 2. These values are then used to calculate PFs such as  $\gamma_Q=1,23$  and  $\gamma_S=1,06$  for the three structures studied (considering  $\beta_t=3,8$ ). The other variables  $\gamma_C$  and  $\gamma_G$ , which have a less significant influence, do not need to be adjusted, allowing the calibration to focus on the most influential elements with regard to structural reliability for the case of a failure mode due to steel tension in the lower fibre.

In this study, the values of  $\gamma_Q$  and  $\gamma_S$ , which have been adjusted according to the two approaches, yield identical results. Furthermore, the values found are also in close proximity to those that have been adjusted in *fib* MC2020, which proposes  $\gamma_Q=1.25$  and  $\gamma_S = 1.10$  [9].

## 5 CONCLUSION

This study examines the adjustment of PFs, with particular attention to the sensitivity coefficients specific to Open Frame Underpass existing structures, which have been modelled using the Chaoa-P calculation chain. The approach adopted allows for the identification of distinct partial coefficients for each variable. This reveals that the uncertainties on some variables are more significant than others. In particular, the traffic variable emerges as a dominant variable in calculations of reliability.

Further investigation, specifically an examination of the sensitivity coefficients' variability in relation to the statistical parameters (mean and standard deviation), could lead to the identification of suitable coefficients for a diverse range of distributions. Additionally, the impact of uncertainty variables on certain parameters is evident, emphasizing the necessity of considering the primary sources of uncertainty within the model. Consequently, the analysis must be based on empirical data, and its results must be verified against experimental data to ensure their consistency with observed reality.

Finally, the *fib* MC2020 recommends adopting different values for sensitivity coefficients for existing structures. This recommendation is based on the observation that the expected lifetime of an existing structure may differ from that of a new structure. The reference period for this recommendation is 1 year rather than 50 years, in line with the observation that the expected lifetime of an existing structure differs from that of a new structure. Consequently, this modification impacts the target reliability index. Accordingly, the proposed sensitivity coefficients are  $\alpha_S = -0.8$  and  $\alpha_R = 0.7$  [9].

## References

- [1] Crémona, C., Poulin, B., Colas, A.-S., Michel, J., Marcotte, C., Vion, B., Orcesi, A., Sadone, R. 2013. "Calibration of partial safety factors for the assessment of existing bridges." Paper presented at the IABSE Conference, Rotterdam 2013: Assessment, Upgrading and Refurbishment of Infrastructures, Rotterdam, The Netherlands, November 21–24, pp. 290–291. doi:10.2749/222137813806479133.
- [2] Diamantidis, D., Tanner, P., Holicky, M., Madsen, H.O., Sykora, M. 2024. "On reliability assessment of existing structures." *Structural Safety*, 102452. doi:10.1016/j.strusafe.2024.102452.
- [3] CEN 2005. "EN1990: Basis of structural design." Brussels: European Committee for Standardization. URL <https://www.phd.eng.br/wp-content/uploads/2015/12/en.1990.2002.pdf> (accessed 12.4.21).



- [4] Association française de normalisation. 2007. “Eurocode - Bases de calcul des structures - Annexe nationale - NF EN 1990/A1:2006.”
- [5] CEN 2004. “Eurocode 1: Actions on structures - Part 2: Traffic loads on bridges.” Brussels: European Committee for Standardization. EN 1991-2:2004.
- [6] CEN 2005. “Eurocode 2: Design of concrete structures - Part 1-1: General rules and rules for buildings.” Brussels: European Committee for Standardization. EN 1992-1-1:2005.
- [7] CEN 2021. “Eurocode 2: Design of concrete structures - Part 1-1: General rules and rules for buildings.” Brussels: European Committee for Standardization. Annex A. prEN 1992-1-1:2021.
- [8] CEN 2020. “Evaluation and Renovation of Existing Structures - FprCEN/TS 17440:2020 (F).” Brussels: European Committee for Standardization.
- [9] Caspeele, R., Van Den Henden, K. 2023. “Validation of the harmonized partial factor method for design and assessment of concrete structures as proposed for fib model code 2020.” *Structural Concrete*, 24, 4368–4376. doi:10.1002/suco.202201109
- [10] Service d'Études Techniques des Routes et Autoroutes. 2002. “Programme PIPO 74 - Notice d'emploi.” PIPO.EL. Accessed December 8, 2023. <https://www.cerema.fr/fr/centres-ressources/boutique/portiques-beton-arme-programme-pipo-74-mise-jour-1991>.
- [11] Caspeele, R., Sýkora, M., Allaix, D.L., Steenbergen, R.D.J.M. 2013. “The Design Value Method and Adjusted Partial Factor Approach for Existing Structures.” *Structural Engineering International*, 23, 386–393. doi:10.2749/101686613X13627347100194.
- [12] fib. 2016. “Partial factor methods for existing concrete structures.” fib Bulletin 80. Lausanne: The International Federation for Structural Concrete. doi:10.35789/fib.BULL.0080.
- [13] Orcesi, A., Diamantidis, D., O'Connor, A., Palmisano, F., Sýkora, M., Boros, V., Caspeele, R., Chateaneuf, A., Mandić Ivanković, A., Lenner, R., Kušter Marić, M., Nadolski, V., Schmidt, F., Skokandić, D., Van der Spuy, P. 2023. “Investigating Partial Factors for the Assessment of Existing Reinforced Concrete Bridges.” *Structural Engineering International*. doi:10.1080/10168664.2023.2204115
- [14] Hajek, P. 2023. “Sustainability perspective in fib MC2020: Contribution of concrete structures to sustainability.” *Structural Concrete*, 24, 4352–4361. doi:10.1002/suco.202300022.
- [15] Holický, M., Markova, J., Sýkora, M. 2008. “Partial factors for assessment of existing reinforced concrete bridges.” Paper presented at the 6th International Probabilistic Workshop.
- [16] König, G., Hosser, D. 1981. “The simplified level II method and its application on the derivation of safety elements for level I.” In: Conceptual Preparation of Future Codes - Progress Report. *CEB Bulletin*. fédération internationale du béton, Technische Hochschule Darmstadt.
- [17] CEREMA. 2023. “CHAMOA-P: CHAîne Algorithmique Modulaire Ouvrages d'Art Programme de calcul des ouvrages de type PIPO.” *Les Outils*, Centre d'études et d'expertise sur les risques, l'environnement, la mobilité et l'aménagement.
- [18] CEREMA. 2020. “ST1v2.233.” [Software]. Available at: <https://logiciels-oa.cerema.fr/LogicielsOA/ST1/st1.html>. Paris: Centre d'Études et d'Expertise sur les Risques, l'Environnement, la Mobilité et l'Aménagement.
- [19] CEREMA. 2022. “CDS.” Version 6.35, [Software] Available at: <https://logiciels-oa.cerema.fr/LogicielsOA/CDS/cds.html>. Paris: Centre d'Études et d'Expertise sur les Risques, l'Environnement, la Mobilité et l'Aménagement.
- [20] Pugliese, F., De Risi, R., Di Sarno, L. 2022. “Reliability assessment of existing RC bridges with spatially-variable pitting corrosion subjected to increasing traffic demand.” *Reliability Engineering and System Safety*, 218(A), article 108137. doi:10.1016/j.res.2021.108137.
- [21] Mohammadkhani-Shali, S. 2007. “Study of System Redundancy in Bridges: Failure Mechanism Analysis Using Response Surface Methods.” PhD diss., Ecole Nationale des Ponts et Chaussées.
- [22] Joint Committee on Structural Safety. 2021. “JCSS Probabilistic Model Code.” Accessed [29/04/2024]. <https://www.jcss-lc.org/jcss-probabilistic-model-code/>.
- [23] Markova, J., Holický, M., Sýkora, M., Kral, J. 2013. “Probabilistic assessment of traffic loads on bridges.” In *Safety, Reliability and Risk Analysis*, CRC Press, pp. 2613–2618. doi:10.1201/b15938-393.
- [24] Moustapha, M., Marelli, S., Sudret, B. 2022. “Active learning for structural reliability: Survey, general framework and benchmark.” *Structural Safety*, 96, article 102174. doi:10.1016/j.strusafe.2021.102174.
- [25] Crémone, C. 2010. “ReliabTbx Release 1.5 - Structural Reliability Toolbox.”