CALIBRATING MATERIAL PARTIAL SAFETY FACTORS FOR EXISTING BRIDGES IN ITALY

A. Grella
Università degli Studi di Napoli Federico II, Napoli, Italia

M. Giorgio
Università degli Studi di Napoli Federico II, Napoli, Italia

F. Noto
METIS srl, Roma, Italia

P. Franchin
Università La Sapienza, Roma, Italia

I. Iervolino
Università degli Studi di Napoli Federico II, Napoli, Italia
IUSS – Scuola Superiore di Pavia, Pavia, Italia

ABSTRACT
Partial safety factors are required in the *load-resistance factor design* (LFRD) context. This study outlines a probabilistic framework based on the (log-normal) *stress-strength* model to derive partial safety factors to be applied to safety verifications of existing bridges in Italy. A closed-form approach, which assumes that only the median of the strength is an estimated quantity, is employed. The design strength is defined as the ratio of the median strength from tests on the structural materials and the partial safety factor, which must be larger than the design load. The latter is derived from the load model and the reliability index used for calibration. The approach pursued in this work enables controlling the risk of *fallacious* safety structural assessment within LRFD. In fact, the calibration criteria are: (i) to control the risk that structures with acceptable reliability level do not pass the safety check, and – at the same time – the risk that those with unacceptable reliability level pass the check; (ii) assure that the risk of *fallacious* assessment reduces with the number of tests. This approach has been applied to calibrate partial factors for concrete and steel, for which two distinct values of partial safety factors have been derived.

Keywords: structural reliability, probabilistic analysis, partial safety factors.

Reference topic: Reliability, Risk, Robustness, and Resilience of Structures.

N. Augenti, F. Parisi (editors). *Atti IF CRASC '25 – VI Convegno di Ingegneria Forense e IX Convegno su Crolli, Affidabilità Strutturale, Consolidamento*. Doppiavoce, Napoli 2025.

1. INTRODUCTION

Following the collapse of the *Polcevera* bridge in Genoa (e.g., Calvi et al., 2019), the need for a systematic assessment of road infrastructure across Italy emerged. Special attention has been devoted to safety checks of existing bridges according to *Linee Guida per la Classificazione e Gestione del Rischio, la Valutazione della Sicurezza ed il Monitoraggio dei Ponti Esistenti* or LLGG (CS.LL.PP., 2020).
LLGG provides a formulation for defining design strengths to be used in the safety assessment of existing bridges, based on information obtained from in-situ investigations and partial safety factors to be used within the framework of *load-resistance factor design* or LRFD (e.g., Melchers and Beck, 2018).
This study aims at evaluating the partial factors used in LLGG, and possibly re-calibrate them, matching two criteria: (i) to control the risk of *fallacious* structural assessment, that is, assuring that bridges with acceptable reliability do pass the safety verification with high probability and that bridges with unacceptable reliability passes the verification with a low probability; (ii) to assure that increasing the number of material tests must further reduce the risk of *fallacious* assessment.
Material strength, failure mechanism, and effect of the applied loads are modelled as lognormal RVs. The partial factors calibration is an analytical and easily applicable procedure that is developed by assuming that in the safety check of existing bridges only the median of the strength of the material should be estimated from in-situ test data, while the probability distribution of failure mechanism and effect of the applied loads are assigned. In this context, it was found that re-calibration was needed for concrete and steel, yielding partial safety factors equal to1.54 and 1.22, respectively. The robustness of the resulting safety verification check with respect to the other parameter of the probability distribution of the strength of the material was verified by means of a parametric analysis.
This study is structured as follows: section 2 discusses the context of the study and the assumptions underlying the calibration procedure. Section 3 outlines the calibration criterion and the calculation of the probability of a positive safety check outcome. Section 4 presents the results that support the conclusions provided in section 5.

2. CONTEXT AND HYPOTHESIS OF THE WORK STRUCTURE

In LLGG (CS.LL.PP., 2020) the design strength, R_{LLGG}, is defined as:

$$R_{LLGG} = min\left(\frac{f_m}{\gamma_M \cdot FC}; \frac{f_k}{FC}\right), \quad (1)$$

where γ_M denotes the partial safety factor for the considered material equal to 1.26 and 1.10 for concrete and steel, respectively; FC represents the confidence factor (CS.LL.PP., 2019), which hereafter is not considered or set equal to one; f_m is the mean of the strength from tests on the structural materials; f_k is the characteristic value of the material strength distribution. The derivation of the current factors for steel and concrete is not known to the authors. One possibility is equation (1) from Massa (2020):

$$\gamma_M = \gamma_{Rd1} \cdot \gamma_{Rd2} \cdot \gamma_f = \frac{1}{1-0.4 \cdot \alpha_R \cdot \beta_t \cdot V_{\theta_{Rd1}}} \cdot \frac{1}{1-0.4 \cdot \alpha_R \cdot \beta_t \cdot V_{\theta_{Rd2}}} \cdot \frac{e^{\left(1-1.645 \cdot V_f\right)}}{e^{\left(1-\alpha_R \cdot \beta_t \cdot V_f\right)}}. \quad (2)$$

In the equation γ_{Rd1} and γ_{Rd2} are the partial factors accounting for model and geometric uncertainties, equal to 1.07 and 1.07 respectively for concrete and to 1.02 and 1.04 respectively for steel, while γ_f is the partial safety factor associated solely with the material strength, equal to 1.10 for concrete and 1.03 for steel. The terms α_R, β_t, $V_{\theta_{Rd1}}$, and $V_{\theta_{Rd2}}$ represent, respectively, the sensitivity coefficient, the *target reliability index* (with $\beta_t = 2.8$), and the coefficients of variation of the random variables (RVs), θ_{Rd1} and θ_{Rd2}, which account for model and geometry uncertainty. The coefficient of variation associated with material strength, V_f, is equal to 0.15 for concrete and 0.05 for steel (fib - Fédération Internationale du béton, 2016).
It appears from the $e^{1-1.645 \cdot V_f}$ term that the resulting partial factor is to be applied to f_k to obtain the design strength. On the other hand, according to equation (1), it is applied to f_m, creating an apparent inconsistency. In fact, values of γ_M calibrated to be applied to the mean f_m would be greater than the values derived from equation (2).
This issue is one of the motivations of this work, which led to the following goals: to recalibrate the partial safety factors based on structural reliability; to ease the calibration ensuring reproducibility; to cover a broad range of cases that the practitioner may encounter; to simplify as much as possible the expression given in equation (1).

2.1. Log-normal Stress-Strength Model

The calibration process developed in this study is based on a time-invariant *stress-strength* model, in which the strength, *R*, and the load effect or stress, *S*, are treated as stochastically independent log-normal RVs: $R \sim LN(\mu_{\ln R}, \sigma_{\ln R})$, $S \sim LN(\mu_{\ln S}, \sigma_{\ln S})$, where $\mu_{\ln R}$, $\mu_{\ln S}$ and $\sigma_{\ln R}$, $\sigma_{\ln S}$ are the mean and the standard deviation of $\ln R$ and $\ln S$, respectively. Therefore, the probability of failure p_f is:

$$p_f = P[R/S < 1] = \Phi\left(-\frac{\mu_{\ln R}-\mu_{\ln S}}{\sqrt{\sigma_{\ln R}^2+\sigma_{\ln S}^2}}\right) = \Phi(-\beta), \quad (3)$$

where $\Phi(\cdot)$ denotes the Gaussian cumulative distribution function, and $\beta = (\mu_{\ln R} - \mu_{\ln S})/\sqrt{\sigma_{\ln R}^2 + \sigma_{\ln S}^2}$ is the so-called *reliability index* (Cornell, 1969). Based on the definition of *sensitivity coefficients* for resistance, α_R, and load effect, α_S, as:

$$\begin{cases} \alpha_R = \frac{\sigma_{\ln R}}{\sqrt{\sigma_{\ln R}^2+\sigma_{\ln S}^2}} \\ \alpha_S = -\frac{\sigma_{\ln S}}{\sqrt{\sigma_{\ln R}^2+\sigma_{\ln S}^2}}, \end{cases} \quad (4)$$

it is possible to obtain the *safety check inequality* for LFRD:

$$\mu_{\ln R} - \alpha_R \cdot \beta \cdot \sigma_{\ln R} \geq \mu_{\ln S} - \alpha_S \cdot \beta \cdot \sigma_{\ln S}, \tag{5}$$

where $\mu_{\ln R} - \alpha_R \cdot \beta \cdot \sigma_{\ln R}$ is the design strength, R_d, and $\mu_{\ln S} - \alpha_S \cdot \beta \cdot \sigma_{\ln S}$ is the load design effect, S_d. Note that equation (5) is usually employed in design to determine the value of $\mu_{\ln R}$ necessary to satisfy the inequality with a target reliability β_t, for given values of α_S, $\mu_{\ln S}$, $\sigma_{\ln S}$ and $\sigma_{\ln R}$. In the following, equation (5) is used in a slightly different manner, to control the risk of *fallacious* structural assessment, in the sense already explained.

2.2. Characterization of uncertainties

In accordance with advanced international construction standards (CEN, 2004; CEN/TC250/SC2, 2021), the random variable R is assumed to be the product of two independent log-normal RVs: namely, $r \sim LN(\mu_{\ln r}, \sigma_{\ln r})$, which accounts for the uncertainties associated with the failure mechanism, and $f \sim LN(\mu_{\ln f}, \sigma_{\ln f})$, which models the material strength. Under this assumption it results:

$$R = f \cdot r \sim LN\left(\mu_{\ln f} + \mu_{\ln r}, \sqrt{\sigma_{\ln f}^2 + \sigma_{\ln r}^2}\right). \tag{6}$$

Consequently, the verification inequality (5) can be rewritten as:

$$\mu_{\ln f} + \mu_{\ln r} - \alpha_R \cdot \beta_c \cdot \sigma_{\ln R} \geq \mu_{\ln S} - \alpha_S \cdot \beta_c \cdot \sigma_{\ln S}, \tag{7}$$

where $\sigma_{\ln R} = \sqrt{\sigma_{\ln f}^2 + \sigma_{\ln r}^2}$. In the following, it is assumed that for the safety verification of existing bridges only $\mu_{\ln f}$ is estimated from data collected via in situ testing. The other parameters are assumed either to be known or (as in the case of $\sigma_{\ln f}$) to vary in an assigned range. The values used for the parameters $\sigma_{\ln f}$, $\mu_{\ln r}$, and $\sigma_{\ln S}$ in the calibration procedure are reported in Table 1, where three values are used for $\sigma_{\ln S}$ to span over a range of possible design situations (traffic, snow, wind).

In the calibration procedure the values of $\sigma_{\ln f}$ are used in a parametric analysis. The decision of defining a safety verification inequality that does not require an estimate of $\sigma_{\ln f}$ was mainly motivated by the circumstance that standard deviation estimates from small samples (as those collected via in-situ tests typically are) often fall outside the range that can be defined a-priori by experienced practitioners (i.e., those considered in Table 1). In addition, this choice significantly simplifies the implementation and the calibration of safety verification inequality. In the calibration procedure β was set to the value $\beta_c = 2.3$, which is set as the lower reliability threshold below which a bridge that passes the verification represents a *fallacious* assessment. Finally, to simplify the calibration process, in the verification inequality, the *sensitivity coefficients* α_R and α_s are set to the literature values $\alpha_R = 0.8$ and $\alpha_S = -0.7$

(König and Hosser, 1981), regardless of the specific values considered for standard deviations of $\ln R$ and $\ln S$. The aim of the calibration procedure was to define a unique value of the partial safety factor that simultaneously covers all the considered experimental scenarios, consistent with the findings reported in the Model Code 2020 (fib - Fédération Internationale du béton, 2023). The verification inequality that accounts for the circumstance that $\mu_{\ln f}$ is estimated is obtained by replacing $\mu_{\ln f}$ with the maximum likelihood estimate of the -quantile of order ϑ of the (Gaussian) logarithm of the material strength $m_{\ln f} - z_{1-\vartheta} \cdot \sigma_{\ln f,c}$, where $m_{\ln f} = \sum_{i=1}^{n} \ln(f_i)/n$ and $\sigma_{\ln f,c}$ is set to 0.150 in the case of concrete and 0.045 in the case of steel. Consequently, equation (7) becomes:

$$m_{\ln f} - z_{1-\vartheta} \cdot \sigma_{\ln f,c} + \mu_{\ln r} - \alpha_R \cdot \beta_c \cdot \sigma_{\ln R} \geq \mu_{\ln S} - \alpha_S \cdot \beta_c \cdot \sigma_{\ln S}, \quad (8)$$

which, setting $\gamma_M = e^{z_{1-\vartheta} \cdot \sigma_{\ln f,c} - \mu_{\ln r} + \alpha_R \cdot \beta_c \cdot \sigma_{\ln R}}$, reduces to:

$$e^{m_{\ln f}}/\gamma_M \geq e^{\mu_{\ln S} - \alpha_S \cdot \beta_c \cdot \sigma_{\ln S}}, \quad (9)$$

where $e^{m_{\ln f}}/\gamma_M$ is the design resistance, R_d, $e^{\mu_{\ln S} - \alpha_S \cdot \beta_c \cdot \sigma_{\ln S}}$ is design load effect, S_d, and γ_M is the partial safety factor.

R, S parameters	Concrete	Steel
$\sigma_{\ln f}$	0.100, 0.150, 0.200	0.045, 0.060
$\mu_{\ln r}$	-0.031	0.035
$\sigma_{\ln r}$	0.145	0.067
$\sigma_{\ln S}$	0.15, 0.25, 0.35 (spanning usual action effect uncertainty for traffic, snow, wind)	0.15, 0.25, 0.35 (spanning usual action effect uncertainty for traffic, snow, wind)

Table 1. Input data for the calibration procedure.

3. CALIBRATION OF THE PATIAL SAFETY FACTOR

3.1. Reliable and unreliable bridges

Given that $m_{\ln f}$ is a RV, the verification conducted through equation (8) may lead to two distinct undesirable outcomes: (i) a bridge that has an acceptable reliability does not pass the verification and (ii) a bridge that has an unacceptable reliability level passes the verification. The aim of the calibration procedure is to define a unique value γ_M in a manner that:

- a bridge with $\beta \leq \beta_c = 2.3$ (case ii) has an arbitrarily *low probability* of passing the verification, which decreases as the number n of experimental tests increases;
- a bridge with a $\beta \geq 2.8$ (case i) has an arbitrarily *high probability* of passing the verification, which increases as the number n of experimental tests increases.

3.2. Probability of passing the safety check for $\boldsymbol{\beta_c = 2.3}$

In the framework of the assumptions taken, given that $e^{m_{\ln f}} \sim LN\left(\mu_{\ln f}, \sigma_{\ln f}/\sqrt{n}\right)$, the probability of making a wrong decision can be computed in closed-form as:

$P[R_d \geq S_d] = P\left[e^{m_{\ln f}}/\gamma_M \geq e^{\mu_{\ln S} - \alpha_S \cdot \beta_c \cdot \sigma_{\ln S}}\right] = P\left[m_{\ln f} \geq \ln \gamma_M + \mu_{\ln S} - \alpha_S \cdot \beta_c \cdot \sigma_{\ln S}\right] = \Phi\left(-\frac{\ln \gamma_M + \mu_{\ln S} - \alpha_S \cdot \beta_c \cdot \sigma_{\ln S} - \mu_{\ln f}}{\sigma_{\ln f}/\sqrt{n}}\right)$,

which, being $\beta = (\mu_{\ln f} + \mu_{\ln r} - \mu_{\ln S})/\sqrt{\sigma_{\ln f}^2 + \sigma_{\ln r}^2 + \sigma_{\ln S}^2}$, can be rewritten as:

$$P[R_d \geq S_d] = \Phi\left(-\frac{\ln \gamma_M + \mu_{\ln r} - \alpha_S \cdot \beta_c \cdot \sigma_{\ln S} - \beta \cdot \sqrt{\sigma_{\ln f}^2 + \sigma_{\ln r}^2 + \sigma_{\ln S}^2}}{\sigma_{\ln f}/\sqrt{n}}\right), \qquad (10)$$

where β is the true reliability index of the bridge that is subjected to the check.
The calibration of γ_M is performed, computing the probability (10), setting the parameters $\mu_{\ln r}$, $\sigma_{\ln r}$, and $\sigma_{\ln S}$ to the values reported in Table 1, and verifying, separately for concrete and steel, that the value assigned to γ_M, satisfies the calibration goals mentioned above, in a range of β and $\sigma_{\ln f}$ values. The latter still from Table 1.

4. RESULTS

The partial safety factors are presented in the Table 2. They represent the intermediate values between the γ_M calculated for each combination of the parameters $P[R_d \geq S_d]$ depends on. They were obtained tuning ϑ in $\gamma_M = e^{-\mu_{\ln r} + z_{1-\vartheta} \cdot \sigma_{\ln f,c} + \alpha_R \cdot \beta_c \cdot \sigma_{\ln R}}$, via a trial-and-error approach to cover all cases. The resulting ϑ values are $\vartheta = 0.35$ and $\vartheta = 0.05$ for concrete and steel, respectively.

Concrete	Steel
1.54	1.22

Table 2. Partial safety factor values to be applied to the median material strength.

As examples of the effect of the partial factors in Table 2, the following tables provide the probabilities of passing the safety check, $P[R_d \geq S_d]$, for a range of reliability indices from $\beta = 2.0$ to $\beta = 3.5$, when $\sigma_{\ln S}$ is equal to 0.25 and $\sigma_{\ln f,c}$ is equal to 0.15 and 0.045 for concrete and steel, respectively. All other cases not shown here are consistent and available for consultation (Grella, 2025). Probabilities in the tables are provided varying the number, n, of in situ tests from two to thirty. It appears that $P[R_d \geq S_d]$ are (arbitrarily) low for unreliable bridges (reliability index not exceeding 2.3) and (arbitrarily) high for reliable bridges

(reliability index greater than or equal to 2.8).
For comparison the same tables are provided below when the safety factors are those current of LLGG, that is, 1.26 for concrete and 1.10 for steel (with $FC = 1$); it emerges that the probability of unreliable bridges passing safety check increases. In addition, in some cases, the probability of a *fallacious* safety assessment increases with the number of in situ tests.

n	*β=2.0*	*β=2.2*	*β=2.3*	*β=2.8*	*β=2.9*	*β=3.0*	*β=3.3*	*β=3.5*
2	0.076	0.206	0.304	0.847	0.908	0.949	0.995	0.999
5	0.012	0.098	0.209	0.947	0.982	0.995	1.000	1.000
15	0.000	0.012	0.080	0.997	1.000	1.000	1.000	1.000
30	0.000	0.001	0.024	1.000	1.000	1.000	1.000	1.000

Table 3. Probability of passing the safety check; material: concrete.

n	*β=2.0*	*β=2.2*	*β=2.3*	*β=2.8*	*β=2.9*	*β=3.0*	*β=3.3*	*β=3.5*
2	0.000	0.033	0.156	0.999	1.000	1.000	1.000	1.000
5	0.000	0.002	0.055	1.000	1.000	1.000	1.000	1.000
15	0.000	0.000	0.003	1.000	1.000	1.000	1.000	1.000
30	0.000	0.001	0.024	1.000	1.000	1.000	1.000	1.000

Table 4. Probability of passing the safety check; material: steel.

n	*β=1.6*	*β=1.9*	*β=2.0*	*β=2.1*	*β=2.2*	*β=2.3*	*β=2.8*	*β=3.0*
2	0.221	0.560	0.677	0.778	0.858	0.916	0.998	1.000
5	0.112	0.595	0.766	0.887	0.955	0.985	1.000	1.000
15	0.018	0.661	0.895	0.982	0.998	1.000	1.000	1.000
30	0.001	0.721	0.962	0.998	1.000	1.000	1.000	1.000

Table 5. Probability of passing the safety check with the safety factors of LLGG; material: concrete.

n	*β=1.6*	*β=1.9*	*β=2.0*	*β=2.1*	*β=2.2*	*β=2.3*	*β=2.8*	*β=3.0*
2	0.000	0.145	0.408	0.723	0.922	0.988	1.000	1.000
5	0.000	0.047	0.357	0.826	0.988	1.000	1.000	1.000
15	0.000	0.002	0.262	0.948	1.000	1.000	1.000	1.000
30	0.000	0.000	0.184	0.989	1.000	1.000	1.000	1.000

Table 6. Probability of passing the safety check with the safety factors of LLGG; material: steel.

5. CONCLUSIONS

The presented study refers to the re-calibration of material partial safety factors for concrete and steel, to be used in LRFD assessment of existing bridges in Italy. The analytical calibration procedure assumes a log-normal stress-strength model in which only the median strength from in-situ test is estimated. The characterization of the uncertainties affecting the safety check were assigned to cover a range of practical situations. The calibration criteria were to control the risk of *fallacious* structural safety assessments for acceptable (reliability index larger than $\beta \geq 2.8$) and unacceptable (reliability index lower than $\beta \leq \beta_c = 2.3$) bridges, which should always be lowered increasing the number of tests. The derived factors, which include the effect of the number of tests, are greater than those currently adopted, for explainable reasons. In fact, with the current values of the partial safety factors, even bridges with reliability index as low as 1.90 (for concrete) and 2.1 (for steel) have probability of passing the safety check that are large and increasing with the number of tests. It is to finally note that the study presented herein intentionally left out the discussion about the confidence factors, to be possibly employed in the safety verification, which requires a dedicated effort.

ACKNOWLEDGEMENTS

This study was developed within the activities of ReLUIS – Rete dei Laboratori di Ingegneria Sismica e Strutturale (cup E65F21001330005).

REFERENCES

Calvi, G.M., Moratti, M., O'Reilly, G.J., Scattarreggia, N., Monteiro, R., Malomo, D., Calvi, P.M., Pinho, R., 2019. Once upon a Time in Italy: The Tale of the Morandi Bridge. Structural Engineering International 29, 198–217. https://doi.org/10.1080/10168664.2018.1558033

CEN, 2004. Eurocode 2: Design of concrete structures-Part 1-1: General rules and rules for buildings. European Committee for Standardization.

CEN/TC250/SC2, 2021. Background to EC2: Design of concrete structures - Part 1-1: General rules and rules for buildings. European Committee for Standardization.

Cornell, C.A., 1969. A probability-based structural code, in: Journal Proceedings. pp. 974–985.

CS.LL.PP., 2020. Linee Guida per la classificazione e gestione del rischio, la valutazione della sicurezza e il monitoraggio dei ponti esistenti.

CS.LL.PP., 2019. CIRCOLARE 21 gennaio 2019, n.7.

fib - Fédération Internationale du béton, 2023. Model Code for Concrete Structures 2020, fib. ed. Lausanne, Svizzera.

fib - Fédération Internationale du béton, 2016. Partial factor methods for existing concrete structures. Lausanne, Svizzera.

Grella, A., 2025. Affidabilità e sicurezza strutturale dei ponti esistenti (in revisione). Università degli Studi di Napoli Federico II, Napoli.

König, G., Hosser, D., 1981. The simplified level II method and its application on the derivation of safety elements for level I. CEB bulletin 147.

Massa, G., 2020. La verifica di sicurezza dei ponti esistenti. Università Politecnica delle Marche.

Melchers, R.E., Beck, A.T., 2018. Structural reliability analysis and prediction. John Wiley & Sons Ltd.