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Comparison of partial factor methods for existing concrete structures: Application to a cable-stayed bridge

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ABSTRACT: The assessment of existing reinforced concrete structures is currently a critical task to be dealt with by engineers and practitioners. Specifically, bridges and viaducts, composing important links in the infrastructure system, are subjected to changes in structural capacity and demand in time, e.g., due to aging and structural deterioration processes as well as the evolution of intensity and frequency of structural loadings. Moreover, climate change may have a significant impact on this evolution. In this context, partial factor methods calibrated for the design of new structures may lead to unnecessary and expansive interventions when used for the assessment of existing structures. Therefore, the exploitation of semi-probabilistic methodologies tailored for existing structural systems is necessary. In this paper, methodologies developed for the evaluation of existing structures are presented and applied to the safety assessment of a cable-stayed bridge located in France. The obtained results are eventually compared with the outcomes of the assessment performed according to the original design and the current Eurocodes regulations, with a discussion on benefits and limits of the presented methodologies.

1 INTRODUCTION

Civil engineering structures and infrastructures represent a fundamental part of the built environment. Nowadays, a large part of these systems is approaching the end of the design lifetime and needs to be re-assessed in order to evaluate the best strategy to be implemented, e.g., maintenance, reparation, or demolition (Biondini & Frangopol 2016). In particular, the safety and serviceability assessment of existing Reinforced Concrete (RC) bridges is a challenge for practitioners and engineers.

In order to assess the performance of existing structures, it is important to highlight the differences with respect to the design of new systems. Structures are subjected to an evolution in time of the structural resistance R , e.g., due to aging and deterioration processes such as corrosion (Bertolini et al. 2013), as well as the modification of the originally assumed structural demand S , e.g., due to the increase in traffic volume (Dolcemascolo et al. 2015) and to the change of climatic conditions (Retief 2022). Moreover, the remaining service life t_{ref} associated with existing structures may be significantly different from the original design service life, usually equal to 50-100 years (ISO 2394 2015), and the cost of reparation or upgrading to provide certain safety performance is higher respect to the cost of applying the same measure in the design phase of a new structure (CEN/TS 17440 2020). For these reasons, the assessment of existing structures should be

performed with appropriate methodologies, different from the conservative approaches tailored for the design of new structures, considering the different type of uncertainties involved and avoiding possible expensive and unnecessary repair activities (JRC 2015).

In this context, the semi-probabilistic method is a practical approach to account for uncertainties in a simplified and implicit way, providing a tool to practitioners in case of common situations in terms of uncertainties and consequences. This approach is proposed by several national and international design standards, such as the Eurocodes (CEN EN1990 2002), generally for the design of new structures. Nevertheless, this methodology may be calibrated for the assessment of existing structures (ISO 2394 2015), and over the last decade, a significant effort has been provided along these research lines (Diamantidis & Bazzurro 2007; Steenbergen & Vrouwenvelder 2010; Caspeele et al. 2013; Sykora et al. 2017). Guidance for the exploitation of semi-probabilistic approaches tailored for the assessment of existing structures can be found in the *fib* Bulletin N°80 (2016). The proposed methodologies explicitly consider the residual service life, information from in situ and laboratory tests, measurements of variable actions, and reduced target reliability levels according to both economical and human safety criteria, focusing on standard RC structures.

This paper provides basic concepts of the semi-probabilistic approach and an overview of two methodologies for the assessment of existing RC structures. The methodology used for the calibration of the Partial Safety Factors (PSF) of an existing cable-stayed RC viaduct located in France is also discussed. Finally, the results of the safety assessment, exploiting tailored methodology for existing structures, are compared, in terms of different PSF calibration, with both the original design (CCTG 1983a; CCTG 1983b) and Eurocodes prescriptions for new structures (CEN EN1990 2002; CEN EN 1991. 2005; CEN EN1992-2. 2005). The benefits and drawbacks of the presented methodologies are discussed with emphasis on possible future developments.

2 BACKGROUND ON THE SEMI-PROBABILISTIC APPROACH

The semi-probabilistic approach, also addressed as the partial factor method, relies on the satisfaction of the safety criterion stating that the design resistance R_d must be larger than the design demand E_d :

$$R_d \geq E_d \quad (1)$$

The design values are generally determined through multiplication or division of the characteristic values, e.g., R_k and E_k , by the partial safety factors γ , which are calibrated to meet the prescribed reliability requirements, usually represented by the target reliability index β_t . For ordinary structures with a lifetime of 50 years, β_t is assumed equal to 3.8 (CEN EN1990 2002). This approach is proposed by several national and international design standards, such as the Eurocodes (CEN EN1990 2002) when dealing with common situations in terms of uncertainties and consequences.

In the following, a general description of the partial factor method is provided consistently to design of new structures. Eventually, two methodologies tailored for the assessment of existing structures, i.e., the Design Value Method (DVM) (ISO 2394 2015) and the Adjusted Partial Factor Method (APFM) (Caspeele & Taerwe 2012) are introduced.

2.1 Partial factor method for design of new structures

The satisfaction of the criterion, defined by Equation 1, should be checked for all the significant scenarios through the evaluation of R_d and E_d for the different structural components.

The design resistance R_d may be evaluated as function of different parameters:

$$R_d = R \left\{ \eta_i \frac{X_{k,i}}{\gamma_{M,i}}; a_d \right\}, i \geq 1 \quad (2)$$

where η_i are conversion factors, exploited to relate possible test results to the actual behavior of the considered member, $X_{k,i}$ are the characteristic values of the materials property, the a_d is

the value for geometrical data, and $\gamma_{M,i}$ is the partial safety factor incorporating model $\gamma_{Rd,1}$, geometrical $\gamma_{Rd,2}$ and material $\gamma_{m,i}$ uncertainties. Indeed, the factor $\gamma_{M,i}$ may be defined as:

$$\gamma_{M,i} = \gamma_{Rd,1} \cdot \gamma_{Rd,2} \cdot \gamma_{m,i} \quad (3)$$

Particularly, assuming the material strength described by a lognormal distribution, $\gamma_{m,i}$ can be defined as follows:

$$\gamma_{m,i} = \frac{X_{k,i}}{X_{d,i}} = \frac{\exp(-1.645 V_{X_i})}{\exp(-\alpha_R \beta_t V_{X_i})} \quad (4)$$

where V_{X_i} is the coefficient of variation of the material property X_i and the 5% quantile is considered for the characteristic values.

The design demand E_d , considering the effect of external loadings, is generally evaluated as a function of different parameters:

$$E_d = E \left\{ \gamma_{G,i} G_{k,i}; \gamma_P P; \gamma_{Q,1} Q_{k,1}; \gamma_{Q,j} \psi_{0,j} Q_{k,j} \right\} \quad i \geq 1; j \geq 2 \quad (5)$$

where $G_{k,i}$, P , $Q_{k,1}$ and $Q_{k,j}$ are respectively the characteristic values of permanent actions, prestressing, dominant variable loading, and non-dominant variable loadings; $\psi_{k,j}$ are the combination factors related to the selected loading configuration; eventually, $\gamma_{G,i}$, γ_P and $\gamma_{Q,j}$ are respectively the partial safety factors accounting for model, e.g., $\gamma_{Ed,G}$ and $\gamma_{Ed,Q}$, and intrinsic aleatory uncertainties for permanent actions $\gamma_{g,i}$, prestressing γ_p and variable loadings $\gamma_{q,j}$. Specifically, the factors $\gamma_{G,i}$ and $\gamma_{Q,j}$ may be defined as follows:

$$\gamma_{G,i} = \gamma_{Ed,G} \cdot \gamma_{g,i} \quad (6a)$$

$$\gamma_{Q,i} = \gamma_{Ed,Q} \cdot \gamma_{q,i} \quad (6b)$$

Particularly, assuming the partial factor for permanent loading uncertainties $\gamma_{g,i}$ following a normal distribution, it can be defined as:

$$\gamma_{g,i} = \frac{G_{d,i}}{G_{k,i}} = \frac{1 - \alpha_E \beta_t V_{G_i}}{1 - k V_{G_i}} \quad (7)$$

where V_{G_i} is the coefficient of variation of the permanent loading and value $k = 0$ is usually assumed for the permanent actions.

The partial factor for variable loading uncertainties $\gamma_{q,i}$ may be computed as follows:

$$\gamma_{Q,i} = \frac{Q_{d,i}}{Q_{k,i}} = \frac{F_{Q,i,tref}^{-1} \left[\Phi(-\alpha_E \beta_t V_{Q_i}) \right]}{Q_{k,i}} \quad (8)$$

where $F_{Q,i,tref}^{-1}$ is the inverse of the distribution of the maxima over the period t_{ref} , Φ is the standard normal distribution and V_{Q_i} is the coefficient of variation of the variable loading.

The calculation of the partial safety factor is related to the sensitivity factors for resistance α_R and demand α_E , which are considered equal to 0.8 and -0.7 for dominant variables and to 0.32 and -0.28 for non-dominant variables respectively (CEN EN1990 2002; ISO 2394 2015).

2.2 Target reliability for existing structures

The definition of a reliability target β_t represents a crucial step in the semi-probabilistic approach. Indeed, the definition of the fundamental elements of the code, e.g. partial safety factors γ and combination factors ψ , are strictly dependent on the assumed reliability target. In Baravalle & Köhler (2017) and Köhler et al. (2019), a discussion regarding the selection of β_t according to different strategies, addressing the benefits and drawbacks of each methodology, is presented.

For the sake of the analysed case study, the reliability target is defined according to the formulation proposed by the *fib* Bulletin n°80 (2016), which is based on both human safety and economic considerations for bridges. The reliability target is computed according to the

Consequence Class (CC), the remaining lifetime or reference period t_{ref} , and the length of the span that may collapse S , as reported in Table 1. The Consequence Classes are conceived considering increasing consequences, from CC1 to CC3, for structural collapse at economic, social and environmental levels (CEN EN1990 2002; *fib* 2016).

Table 1. Reliability target for existing bridges considering different consequence classes (*fib* 2016).

Consequence Class	Reliability Target β_t
CC1	1.8
CC2	$\text{Max}(2.3; \beta_{t,hs})$
CC3	$\text{Max}(2.8; \beta_{t,hs})$

The reliability target for human safety $\beta_{t,hs}$ is defined as:

$$\beta_{t,hs} = -\Phi^{-1} \left[\frac{2.75 \cdot 10^{-5} \cdot (0.09 \cdot S)^{-2} \cdot t_{ref}}{0.055} \right] \quad (9)$$

2.2 Design value method

The Design Value Method is formulated to compute partial safety factors based on the available information regarding the considered structure. Indeed, based on prior experience or test results it is possible to characterize the random variables connected to the resistance and the demand, reducing the uncertainty in the evaluation. Moreover, the partial safety factors should be defined considering the reliability target β_t related to the existing structure and the expected remaining lifetime t_{ref} .

2.3 Adjusted partial factor method

The Adjusted Partial Factor Method proposes the definition of the partial factors related to existing structure $\gamma_{X,exist}$ multiplying the partial factors related to new structures $\gamma_{X,new}$ using an adjustment parameter ω :

$$\gamma_{X,exist} = \omega \gamma_{X,new} \quad (10)$$

The adjustment parameter accounts for the expected remaining lifetime t_{ref} of the considered structure, the target reliability index β_t , and possible available information from tests, measurement, and prior and new knowledge.

3 CASE STUDY OF A CABLE-STAYED BRIDGE

3.1 Description of the bridge

The assessed existing viaduct is a prestressed cable-stayed bridge built in 1991 in the Isère district in France. The structure consists of a five-span bridge, supported by two abutments, three piers, and one pylon. The structural typology was selected in order to overpass a gap of approximately 300 m in correspondence with the Isère River. Considering an initial design lifetime of 100 years, the remaining lifetime of the bridge is equal to 67 years at the current date.

For the specific case study, the focus is dedicated to the deck of the viaduct. The deck is composed of two types of transversal sections: the box-girder section, having the central web 30 or 45 cm thick, and the plain section, located near the abutments, specifically the first 16 m from each end. The shape of the section is triangular, adopted at the time of the design for its aerodynamic, architectural, and structural characteristics, with 21.4 m width and 2.4 m height, as reported in Figure 1. The section is built in reinforced concrete and prestressed both longitudinally and transversely. The concrete is designed to have a characteristic compressive strength $f_{ck} = 40$ MPa, a characteristic tensile strength $f_{tk} = 3$ MPa, and Young modulus $E_c = 37.6$ GPa, as reported in the document detailing the hypothesis of the original design.

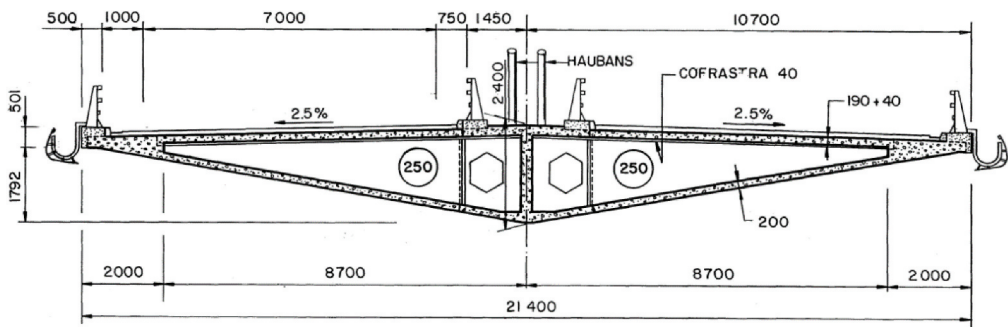


Figure 1. Cross-section of the bridge deck.

3.2 Proposed methodology

The methodology proposed for the assessment of the cable-stayed viaduct is based on three steps:

- Analysis of the original design;
- Assessment according to Eurocode;
- Assessment according to methodology for existing structures.

Firstly, the original design is studied in order to understand the assumption and the regulation exploited at the time of the conception of the viaduct. From the available documents, it is possible to retrieve the data about the materials and the loading considered for the bridge deck design, as well as the partial safety factors proposed by the regulations (CCTG 1983a; CCTG 1983b). The internal action and the stresses along the deck have been deduced from the original design report.

Secondly, the Eurocodes (CEN EN1990 2002; CEN EN 1991. 2005; CEN EN1992-2. 2005) are considered in order to assess the safety of the viaduct.

Eventually, the partial safety factors for the assessment of the viaduct are evaluated using two different methodologies which are tailored for the assessment of the existing structure, namely DVM, and APFM, and considering three different scenarios in terms of existing knowledge and uncertainties. Moreover, the *fib* Bulletin n°80 (2016) is exploited as guidance through the definition of the Reliability Target β_i and the partial safety factors γ .

The loadings considered for the analysis are the dead load G (self-weight and permanent actions) and the traffic load Q .

The methodology proposed for the assessment is developed considering that the Eurocodes are currently calibrated for the design of new structures instead of the assessment of existing ones, leading in some cases to over-conservative results in the case of the evaluation of existing systems.

3.3 Reliability target and partial safety factors

The reliability target β_i is computed according to Equation 4 and Table 1, assuming Consequence Class CC3, remaining lifetime $t_{ref} = 67$ years, and span length $S = 148$ m, which corresponds to the most critical of the viaduct. Consequently, $\beta_i = 3.55$ is defined for the case study.

The sensitivity factors for resistance α_R and demand α_E are assumed to be constant and equal to the ones defined for new structures for the calculation of the partial factors according to DVM and APFM.

The values in terms of coefficient of variation V for model and geometrical uncertainties, both for the material and loading evaluations, are assumed compliant with the values suggested in *fib* Bulletin n°80 (2016). The partial factors for concrete and steel materials as well as the ones for permanent and traffic loading are evaluated according to the DVM and the APFM. Concerning the traffic load, a basic reference time of 15 years is selected.

Three different scenarios related to the existing knowledge of the structure are considered for the calculation:

1. Scenario 1 (reference scenario) is evaluated in agreement with the assumptions performed by Eurocodes (CEN EN 1991, 2005) concerning the probabilistic distributions although a different reliability target is considered.
2. Scenario 2 is conceived assuming a reduction in the uncertainties related to both loading and material and consequently reducing the coefficient of variation V considered in the reference scenario by 20%; this scenario may represent a situation in which testing and measurement led to a reduction of the uncertainties considered in the initial design.
3. Scenario 3 is conceptually opposite with respect to scenario 2, i.e., V is increased by 20% concerning considered in the reference scenario.

The results obtained in terms of partial factors (PF) are summarized in Table 2 for the DVM and Table 3 for the APFM. The results in terms of the partial safety factors for the three scenarios and considering the original design, the Eurocodes, the DVM, and the APFM are reported in Table 4.

Table 2. Partial factor according to different scenarios exploiting the DVM.

	Scenario 1		Scenario 2		Scenario 3		Probabilistic Distribution
	PF	V [-]	PF	V [-]	PF	V [-]	
γ_c [-]	1.20	0.15	1.15	0.12	1.24	0.18	Lognormal
γ_s [-]	1.06	0.05	1.05	0.04	1.07	0.06	Lognormal
γ_g [-]	1.25	0.1	1.20	0.08	1.30	0.12	Normal
γ_q [-]	1.08	0.075	1.04	0.06	1.14	0.09	Gumbel

$\gamma_{Rd,1,c} = 1.05$, $\gamma_{Rd,1,s} = 1.025$, $\gamma_{Rd,2} = 1.05$, $\gamma_{Ed,G} = 1.07$ and $\gamma_{Ed,Q} = 1.12$ (fib 2016).

Table 3. Partial factor according to different scenarios exploiting the APFM.

	Scenario 1		Scenario 2		Scenario 3		Probabilistic Distribution
	V' [-]	PF	V''/V' [-]	PF	V''/V' [-]	PF	
ω_C [-]	0.15	0.958		0.924		0.993	Lognormal
ω_S [-]	0.05	0.985		0.973		0.997	Lognormal
ω_G [-]	0.1	0.982	1.00	0.943	0.8	1.021	Normal
ω_Q [-]	0.075	0.975		0.948		1.003	Gumbel

V' and V'' are the initial or reference and the updated coefficient of variation.

Table 4. Partial safety factor according to different semi-probabilistic approaches.

	Original Design	Eurocodes	Scenario 1		Scenario 2		Scenario 3	
			DVM	APFM	DVM	APFM	DVM	APFM
γ_C [-]	1.50	1.50	1.31	1.44	1.26	1.39	1.37	1.49
γ_S [-]	1.15	1.15	1.14	1.13	1.13	1.12	1.16	1.15
γ_G [-]	1.35	1.35	1.34	1.33	1.28	1.27	1.39	1.38
γ_Q [-]	1.35	1.35	1.21	1.31	1.16	1.28	1.27	1.35

3.4 Discussion

Based on the results listed in Table 4, it is possible to observe that the partial safety factors considered in the regulations of the original design (CCTG 1983a; CCTG 1983b) and the ones related to Eurocodes (CEN EN1990 2002) are equivalent. The difference between the two regulations would be related to the possible discrepancies in the characteristic loading, which may lead to an increase in demand for structural verification.

The partial safety factors computed with the two methodologies tailored for the assessment of existing structure, i.e. DVM and APFM, are significantly different than the one used for the design of new structural systems. Firstly, considering scenario 1, the results obtained with the DVM and APFM are slightly lower than the exploited in the original design and suggested by the Eurocodes. For the examined case study, this is due to the definition of a reliability target for existing structures $\beta_t = 3.55$, which is lower than the one that should be guaranteed for a new structure. Secondly, considering scenario 2, the partial safety factors obtained by the two methodologies for existing structures are significantly lower than the results obtained for the reference scenario. This is related to the assumed reduction of the coefficient of variation, representing a case in which information on the material strength and loading would be available to properly characterize the uncertainty of the variables involved. Eventually, scenario 3 depicts an opposite case with respect to scenario 2, leading to higher partial safety factors obtained by exploiting the DVM and the APFM with respect to scenario 1, due to the increase of the uncertainties.

In general, for all the three investigated scenarios, the results obtained with APFM are more conservative than the ones retrieved exploiting the DVM considering concrete and traffic loading, due to the inherent approximation of the APFM. This aspect is highlighted in several studies, e.g., by Orcesi et al. (2021). Consequently, it is generally suggested to assess the existing structures firstly exploiting the APFM, secondly the DVM and eventually, if needed, to perform a full probabilistic verification. Furthermore, concerning the partial safety factors related to reinforcement and permanent loading, the results achieved with the two methodologies are almost equivalent.

The two approaches are both compatible with the Eurocode format, although it is observed that the DVM is more accurate in incorporating new information with respect to the APFM (Caspelle et al. 2013; Lara et al. 2021). Indeed, a recurrent Percent Deviation (PD) is observed between the results obtained with DVM and APFM for all the scenarios: the PD between the two methodologies is 8-10% for γ_C , 1% for γ_S , 1% for γ_G , and 6-10% for γ_Q . This is due to the differences in the incorporation of uncertainties between the two methodologies. Nevertheless, the obtained results highlight the importance of using appropriate methodology with proper treatment and characterization of uncertainties when dealing with the assessment of existing structures.

4 CONCLUSIONS AND FUTURE DEVELOPMENTS

Different semi-probabilistic methodologies for the assessment of existing structures have been investigated. Specifically, the definition of the partial safety factors for the evaluation of an existing reinforced concrete viaduct located in France has been firstly performed according to the original design of the structure, secondly exploiting the structural safety prescriptions according to the Eurocodes, and eventually considering two different methodologies tailored for existing structures, i.e., DVM and APFM. The results in terms of partial safety factor have been compared to the outcomes of the original design and the assessment performed according to Eurocodes, demonstrating the importance of the exploitation of appropriate methodologies for existing structures to avoid unnecessary and expensive interventions. Moreover, three different scenarios in terms of uncertainties have been defined and compared, showing the significance of a proper characterization of the structural capacity and demand.

Further investigation is necessary to properly characterize the actual information for the materials and loadings regarding the considered structure. Moreover, the same methodology may be applied for the calculation of the partial safety factors for other loadings affecting the structure, such as temperature and wind loading, and used for proper structural verification of the system. To this purpose, appropriate structural modeling at the sectional and structural level of the deck is fundamental to obtain reliable results. In addition, climate change effects are expected to significantly impact on both structural capacity and demand over time. The assumption of stationarity at the base of the current structural design codes is no longer well representing reality and this should be considered when designing or assessing the structural system. For this reason, the impact of climate change should be also integrated into the calibration of the partial safety framework. Specifically, concerning the investigated viaduct, the temperature gradient and the wind load are reported to have significant impact and their evolution due to climate change should be estimated and then incorporated into the safety assessment.

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