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ABSTRACT: *fib* Bulletin 80 proposes two methods for the recalibration of the partial safety factors specific for existing reinforced concrete structures and infrastructures taking into account different issues: the residual service life, information deriving from tests, measurements of variable actions and lower target reliability according to economical and human safety criteria. The methods presented in *fib* Bulletin 80 have been used to assess an existing prestressed concrete bridge built in the 90s. Successively, the results are compared to the outcomes achieved using the approach of EN1990.

KEYWORDS: partial safety factor method; reinforced conrete; existing bridge.

1 INTRODUCTION

Nowadays, a relevant challenge for engineers and practitioners is the assessment of existing reinforced concrete (RC) structures and infrastructures (Bertagnoli et al. 2012, 2014, 2017a,b, Castaldo and Alfano 2020, Giordano et al. 2008). In fact, the safety of existing infrastructures, as bridges and viaducts, is strongly affected by several parameters and phenomena such as environmental actions, materials aging, degradation and variation of magnitude of traffic loads during their service life. For this reason, several scientific studies are devoted to propose methodologies able to assess existing structures and infrastructures (Allen 1991, Mancini et al. 2018). The partial factor method according to the semi-probabilistic approach (EN 1990, *fib* Bulletin 80) to evaluate the structural reliability is the most common approach adopted by engineers and practitioners in presence of both static and dynamic loads (Castaldo et al. 2018a,b, 2019, 2020).

fib Bulletin 80 proposes two methods in order to recalculate the partial safety factors for existing RC structures (i.e., buildings and bridges) in line to updated reliability levels: the design value method (DVM) and the adjusted partial factor method (APFM). Specifically, existing structures have already fulfilled part of their service life and should be assessed only for the remaining part. Moreover, costs for upgrading of existing structures may be higher than costs for building new structures. In this way, the target reliability levels for existing structural systems should be revised.

This paper describes and comments the methods proposed by *fib* Bulletin 80 for assessment of existing bridges and proposes their application to the assessment of an existing precast prestressed RC bridge located in Italy. The re-calibrated partial safety factors as well as the results of the safety verifications are compared to the ones obtained according to the code prescriptions (EN 1990, EN 1992-1, EN 1991) for new structures, highlighting some critical aspects.

2 PARTIAL FACTOR METHODS IN *FIB* BULLETIN 80 FOR EXISTING STRUCTURES

The semi-probabilistic method in compliance with the partial factor format defined by EN 1990 and *fib* Bulletin 80 prescribes to perform the safety verification as follows:

$$R_d \geq E_d \tag{1}$$

where E_d is the design value of the effect of external actions (e.g., internal forces) and R_d denotes the related design structural resistance. In the following, basic notions related to the partial factor format and to the partial safety factors (PSFs) derivation according to EN 1990 are reported. Subsequently, the methods proposed by *fib* Bulletin 80 are briefly described.

2.1 Basics of the partial factor format according to EN1990

The partial factor format (EN 1990, *fib* Bulletin 80) is defined according to the Level I (i.e., semi-probabilistic) approach for evaluation of structural reliability, safety and resilience of infrastructure systems (Troisi and Alfano 2019, 2020). To this purpose, PSFs are applied to both loads and material resistances. In line with (EN 1990), in most of cases, the design value of resistance of a structural component R_d may be evaluated as:

$$R_d = R\left\{\eta_i \frac{X_{k,i}}{\gamma_{M,i}}; a_d\right\} \quad , \quad i \geq 1$$
$$\gamma_{M,i} = \gamma_{Rd1} \cdot \gamma_{Rd2} \cdot \gamma_{m,i} \qquad (2a,b,c)$$
$$\gamma_{m,i} = \frac{X_{k,i}}{X_{d,i}} = \frac{1-1.645V_{X_i}}{1-\alpha_R \beta V_{X_i}}$$

where η_i denotes the conversion factor; $X_{k,i}$, $X_{d,i}$ are the characteristic (i.e., 5% quantile) and the design value of the material property, respectively; V_{Xi} is the coefficient of variation of the material property; a_d is the design value of geometrical parameters; γ_m is the PSF for material uncertainty evaluated according to Equation (2c) assuming a normal probabilistic distribution (Garzino and Troisi, 2015, Golzio and Troisi 2013); $\gamma_{M,i}$ is the PSF accounting for material, geometrical and model uncertainties evaluated according to Equation (2b); γ_{Rd1} is the model uncertainty PSF set equal to 1.05 and 1.025 for concrete and reinforcement (EN 1992-1), respectively; γ_{Rd2} is the PSF accounting for geometrical uncertainties set equal to 1.05 (EN 1992-1).

The design value of the effect due to external actions E_d is evaluated as follows:

$$E_d = E\left\{\gamma_{G,j} G_{k,j}; \gamma_P P; \gamma_{Q,1} Q_{k,1}; \gamma_{Q,i} \psi_{0,i} Q_{k,i}\right\} \quad i > 1; \; j \geq 1 \qquad (3)$$

$$\gamma_{g,j} = \frac{G_{d,i}}{G_{k,i}} = \frac{1-\alpha_R \beta V_{G_j}}{1-kV_{G_j}}$$
$$\gamma_{q,j} = \frac{F_{Q,j,t_{ref}}^{-1}\left[\Phi\left(-\alpha_E \beta, t_{ref}\right)\right]}{Q_{k,j}} \qquad (4a,b)$$

$$\gamma_{G,j} = \gamma_{Ed} \cdot \gamma_{g,j}$$
$$\gamma_P = 1.0 \qquad (5a,b,c)$$
$$\gamma_{Q,j} = \gamma_{Ed} \cdot \gamma_{q,j}$$

where $G_{k,j}$, $G_{d,j}$ represent the characteristic (computed assuming k=0) and design values of the permanent actions; V_{Gj} is the coefficient of variation of the permanent actions; P is the prestressing action (i.e., mean value); $Q_{k,q}$ is the characteristic (i.e., 98th quantile of annual maxima distribution for climatic actions) value of the dominant external action for the selected loading configuration; $\psi_{0,i} Q_{k,j}$ are the combination values of the non-dominant actions for the selected loading configuration; t_{ref} is the reference period (e.g., design service life for new structures and residual service life for existing structures); $F^{-1}_{Q,j,tref}$ is the inverse of cumulative probabilistic distribution of maxima of variable action related to t_{ref} (e.g., Gumbel); Φ is the standard normal distribution; γ_{Ed} is the model uncertainty PSF for actions evaluated according to (EN 1990); $\gamma_{G,j}$, γ_P, $\gamma_{Q,i}$ are the PSFs accounting for model and aleatory uncertainties for permanent, prestressing and variable actions, respectively, evaluated according to Eq. (4a-b) and Equation (5a,c). The level of reliability is defined by the reliability index β, set equal to 3.8 for ordinary structures with 50 years of design service life (i.e., consequence class 2-CC2). The FORM sensitivity factors α_R and α_E are set equal to 0.8 and -0.7 for dominant variables, respectively, and equal to 0.4 and -0.32 for non-dominant variables (EN 1990).

2.2 *Target levels of reliability for existing bridges according to fib Bulletin 80*

As discussed in Section 1, target reliability values for existing structures should be revised with respect to the ones conceived for new structures (*fib* Bulletin 80). *fib* Bulletin 80 illustrates two target reliability indexes: β_{0t}, to be fulfilled by the existing structures as it is; $\beta_{up,t}$, to be fulfilled in case of upgrading of the structure. In general, if β_{0t} is not satisfied, upgrading of the structure is required. Concerning existing RC bridges, the target reliability levels are suggested by *fib* Bulletin 80 accounting for consequences of structural failure, economic, human safety, individual and group risk criteria (Diamantidis and Bazzurro 2007) by the definition of the possible collapsed span of the bridge S and the residual service life t_{ref}. Assuming CC2 in case of structural failure, the values of β_{0t} and $\beta_{up,t}$ can be, respectively, evaluated as follows:

$$\begin{aligned}
&\beta_{0t} = max\left(2.3;\ \beta_{0t,human\ safety}\right)\\
&\beta_{up,t} = max\left(3.3;\ \beta_{up,t,human\ safety}\right)\\
&\beta_{0t,human\ safety} = \beta_{up,t,human\ safety} = -\Phi^{-1}\left[\frac{2.75\cdot 10^{-5}\cdot\left(0.09\cdot S\right)^{-2}\cdot t_{ref}}{0.055}\right]
\end{aligned} \tag{6a,b,c}$$

2.3 *The design value method (DVM)*

The Design Value Method (i.e, DVM), (*fib* Bulletin 80, Caspeele et al. 2013) permits to re-evaluate the partial safety factors $\gamma_{M,i}$, $\gamma_{G,j}$ and $\gamma_{Q,i}$ starting from the actual probabilistic distribution of the variables X_i, G_j and Q_j (based on prior information, or results from tests or the combination of both). The updated partial safety factors can be defined according to target reliability levels related to existing structures (i.e., buildings or bridges) and expected reference life t_{ref}. This method is suggested for structures of importance and may lead to results, in terms of values of partial factors, discordant to the ones derived by EN 1992-1, *fib* Model Code 2010, EN 1990.

2.4 *The adjusted partial factor method (APFM)*

The Adjusted Partial Factor Method (i.e, APFM) (*fib* Bulletin 80, Caspeele et al. 2013) defines partial factors for existing structures ($\gamma_{Existing}$) adjusting the partial factors $\gamma_{M,i}$, $\gamma_{G,j}$, $\gamma_{Q,i}$ for new structures (γ_{New}) proposed by (EN 1990 and EN 1992-1) by means of adjustment factors ω as follows:

$$\gamma_{Existing} = \omega\cdot\gamma_{New} \tag{7}$$

The adjustment factor ω accounts for the target level of reliability and the reference life t_{ref} of the existing structure or infrastructure. The method is fully consistent with EN1990 provisions when the same assumptions are performed, and it is considered as a simplification in comparison to the DVM.

3 APPLICATION EXAMPLE: AVIGLIANA'S BRIDGE

The existing bridge selected for application of *fib* Bulletin 80 is located next to the city of Avigliana in Italy, along the junction between the SS25 and the "Torino-Bardonecchia" highway.

The bridge crosses the river "Dora Riparia" and has been built with a prestressed precast box section in 1990 by means the balanced cantilever methodology. The bridge presents three spans of 30+60+30m for a full length of 120m. The deck is 9.80m wide and the precast elements are 3m high and 3.05m long. Massive RC diaphragms are located in proximity of supports. Isostatic restraints configuration is adopted along lateral and longitudinal directions.

The central piers are high 15m from the river bed. Prestressing has been introduced by post-tensioning technique during the staged construction process.

The tendons are composed of 12 strands of 0.6" of diameters. Top tendons have been tensioned during the construction of the two hammers with balanced cantilever configuration. Bottom tendons have been tensioned after hammer construction and closure of the midspan joint. Finally, the static scheme turns in a continuous beam with four supports. Each tendon is composed of 12 strands of 0.6" of diameters.

More detailed information about the bridge geometry are given in Figure 1a,b,c. The information about bridge geometry, characterization of material and development of construction stages has been derived from the original design reports and drawings. The software SAP2000 (2002) has been adopted in order to define a linear elastic numerical model. The construction phases have been accounted for implementing immediate and delayed losses of prestressing (EN 1992-1), linear creep and shrinkage effects (CEB-FIP Model Code 1990). The analysis has been carried out up to the end of the reference service life of the bridge, as widely discussed in the following section. The effect of prestressing is considered from the actions side within structural analysis. The permanent and variable (i.e., wind, traffic, foundation settlements, seasonal and daily thermal) actions have been defined and properly combined ate ULS with reference to (EN 1991). The concrete compressive strength class for concrete segments is C37/45 and FeB44k steel have been used for ordinary reinforcements. Prestressing strands with 0.6" of diameter present a characteristic yielding strength equal to $f_{p0.1k}$=1600 MPa and a characteristic ultimate strength f_{ptk}=1800 MPa with an initial tension of σ_{p0}=1428 MPa.

4 ASSESSMENT OF AVIGLIANA'S BRIDGE IN LINE TO BULLETIN 80

4.1 Set of the partial safety factors for structural verifications

The *fib* Bulletin 80 suggests to perform the assessment of existing reinforced concrete structures accounting for reduced reliability levels, the residual reference life t_{ref} and information deriving from tests and inspections. In this investigation, two assumptions have been assumed related to the residual reference life of Avigliana's which has been built bridge built in the year 1990: $t_{ref,1}$=22 years, with reference to an original design service life of 50 years; $t_{ref,2}$=72 years, with the hypothesis of original service life of 100 years. In line to *fib* Bulletin 80, the target reliability of CC2 can be evaluated assuming a potential collapsed span S equal to the whole bridge length of 120m. In this way, according to Equation (6), $\beta_{0t} = \beta_{up,t}$ can be set equal to 3.73 and 3.42 assuming residual service life of 22 years and 72 years, respectively. Prior information for materials properties are available from literature CEB-FIP Model Code 1990 with reference to the coefficient of variation of the concrete compressive strength and the reinforcements yielding strength, that can be set equal to V'_c=0.15 and V'_s=0.05, respectively. As information from in situ and laboratory tests on concrete and reinforcements steel was not available for the investigated bridge, three significant scenarios concerning the ratios V"/V' have been assumed as reported in Table 1.

Table 1. Scenarios assumed for material properties.

V''_c/V'_c	f_{cm} [MPa]	f_{ck} [MPa]	V_c [MPa]	V''_s/V'_s	f_{ym} [MPa]	f_{yk} [MPa]	V_s [MPa]
1.00	47	37	0.15, lognormal	1.00	467	430	0.05, lognormal
0.50*		42	0.075, lognormal	0.8*		437	0.04, lognormal
1.50		33	0.225, lognormal	1.50		413	0.075, lognormal

*lower bounds are defined according to fib Bulletin 80 limits of applicability

First row: Scenario 1; Second row: Scenario 2: Third row: Scenario 3

Table 2. Summary of the partial factors for actions according to *fib* Bulletin 80 and to EN 1990.

Residual service life t_{ref} [y]	*22*		*72*		**EN1990**	**Assumptions**
Partial factor	**DVM**	**APFM**	**DVM**	**APFM**		
γ_g	1.26	-	1.24	-	-	V''_g= 0.1, Normal
$\gamma_{q,Wind}$	1.53	-	1.63	-	-	V''_{vb}= 0.12, Gumbel
$\gamma_{q,Traffic}$	1.13	-	1.19	-	-	V''_T= 0.075, Gumbel
$\gamma_{Ed,g}$	1.07	-	-	-	-	-
$\gamma_{Ed,q}$	1.12	-	-	-	-	-
ω_g	-	0.99	-	0.97	-	V'_g=V''_g = 0.1, Normal
$\omega_{q,Wind}$	-	1.10	-	1.14	-	V'_{vb}=V''_{vb}= 0.12, Gumbel
$\omega_{q,Traffic}$	-	0.96	-	1.00	-	V'_T= V''_T=0.075, Gumbel
$\boldsymbol{\gamma_G}$	1.35	1.34	1.33	1.31	1.35	-
$\boldsymbol{\gamma_P}$*	1.00	1.00	1.00	1.00	1.00	-
$\boldsymbol{\gamma_{Q,Imposed\ def.}}$*	1.20	1.20	1.20	1.20	1.20	-
$\boldsymbol{\gamma_{Q,Wind}}$	1.71	1.65	1.82	1.71	1.50	-
$\boldsymbol{\gamma_{Q,Traffic}}$	1.27	1.29	1.33	1.35	1.35	-

The symbol V” represents the coefficient of variation of the posterior probabilistic distribution of the material property after Bayesian updating of the prior distributions characterized by the coefficient of variation V’ *fib* Bulletin 80. The mean values of material properties have been assumed to be in line with the ones defined by design reports, and remains the same after the Bayesian updating process.

In Tables 1-2, the partial factors evaluated according to *fib* Bulletin 80 and EN 1990, adopting the mentioned above hypotheses, are reported. Concerning the permanent and variable actions, in Table 2 all the PSFs defined in according to DVM, APFM together with the related probabilistic assumptions are listed.

Table 3. Summary of the partial factors for material properties according to *fib* Bulletin 80 and to EN 1990, EN 1992-1, scenario 1.

Residual service life t_{ref} [y]	*22*		*72*		**EN1990, EN1992**	**Assumptions**
Partial factor	**DVM**	**APFM**	**DVM**	**APFM**		
γ_c	1.22	-	1.18	-	-	V''_c= 0.15, lognormal
γ_s	1.07	-	1.06	-	-	V''_s= 0.05, lognormal
$\gamma_{Rd,c}$	1.21	-	1.21	-	-	-
$\gamma_{Rd,s}$	1.08	-	1.08	-	-	-
ω_c	-	0.99	-	0.96	-	V'_c=0.15; V''_c= 0.15, lognormal
ω_s	-	1.00	-	0.99	-	$V's$=0.05; $V''s$= 0.05, lognormal
$\boldsymbol{\gamma_c}$	1.48	1.49	1.43	1.43	1.50	-
$\boldsymbol{\gamma_s}$	1.15	1.15	1.14	1.13	1.15	-

The PSFs for material properties are listed in Table 3, Table 4 and Table 5 for the different scenarios related to the outcomes of material tests. In function of the variation of the ratio V''/V', the PSF related to the concrete compressive strength and the reinforcements yielding strength can be significantly different from the ones defined by EN 1992-1, also depending on the different target reliability level. More details may be found in (Gino et al. 2019).

Table 4. Summary of the partial factors for material properties according to *fib* Bulletin 80 and to EN 1990, EN 1992-1, scenario 2.

Residual service life t_{ref} [y]	*22*		*72*		**EN1990, EN1992**	**Assumptions**
Partial factor	**DVM**	**APFM**	**DVM**	**APFM**		
γ_c	1.11	-	1.09	-	-	V''_c= 0.075, Lognormal
γ_s	1.06	-	1.04	-	-	V''_s= 0.04, Lognormal
$\gamma_{Rd,c}$	1.21	-	1.21	-	-	-
$\gamma_{Rd,s}$	1.08	-	1.08	-	-	-
ω_c	-	0.90	-	0.88	-	V'_c=0.15; V''_c= 0.075, Lognormal
ω_s	-	0.98	-	0.97	-	$V's$=0.05; $V''s$= 0.04, Lognormal
$\boldsymbol{\gamma_c}$	1.34	1.35	1.31	1.32	1.50	-
$\boldsymbol{\gamma_s}$	1.14	1.13	1.12	1.12	1.15	-

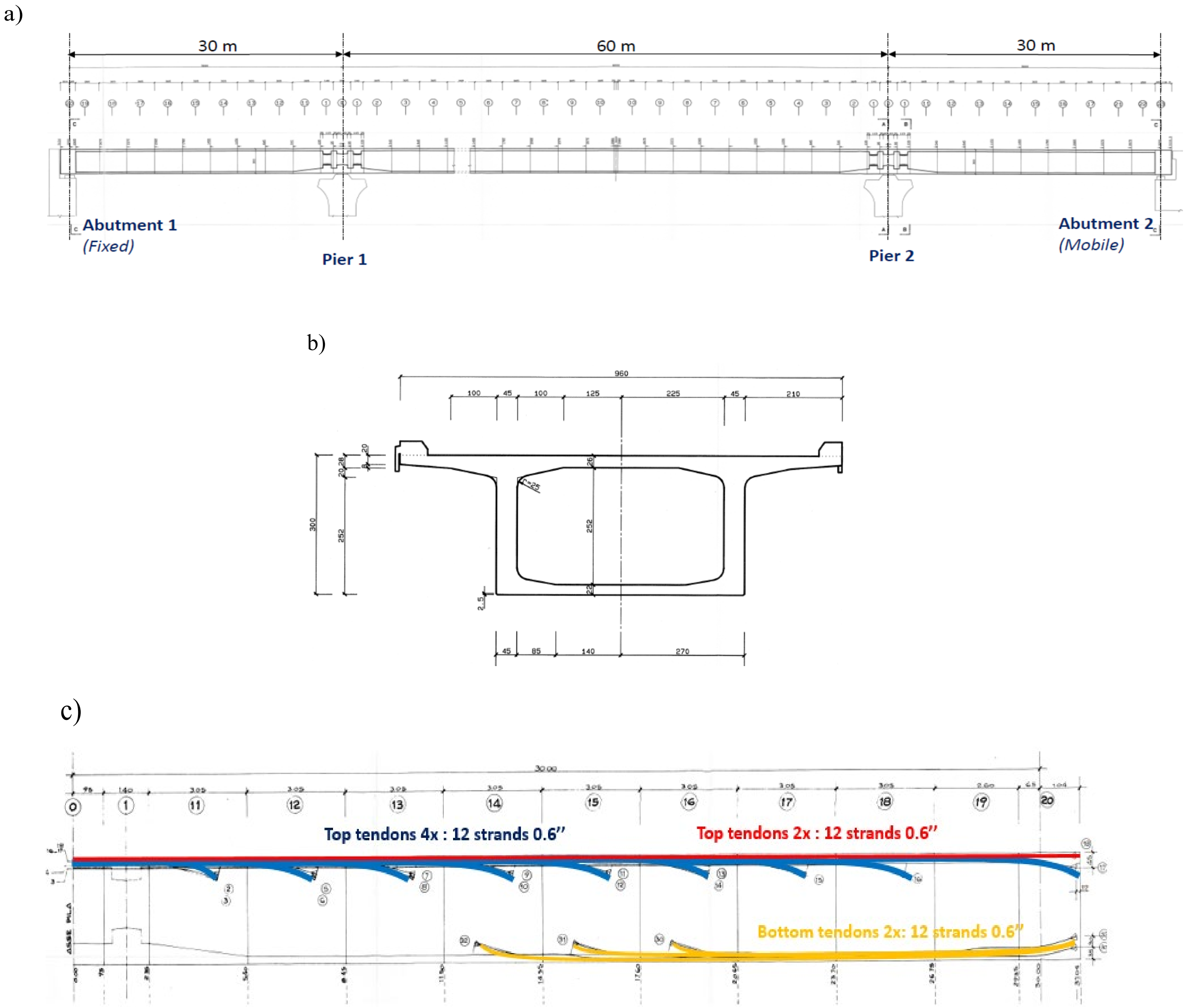

Figure 1. Geometrical configuration of the bridge (a); Typical segment cross section (b); prestressing tendons layout for the half hammer of span between abutment 1 and pier 1 (c) (dimensions in [cm]).

In Table 2, PSFs proposed by EN 1990 are also reported. It can be noted that PSFs related to prestressing and imposed deformations (i.e., thermal actions and foundation settlements) are not discussed by *fib* Bulletin 80 and are assumed in line to EN 1990 also for safety verification within the DVM and APFM methods. The APFM seems to be safer than DVM concerning the PSF for traffic loads. The PSFs for permanent actions are similar to ones provided by EN 1990. Important discrepancies are highlighted for the wind PSF adopting APFM and DVM if compared to EN 1990. This because, as discussed by (Steenbergen et al. 2010) and *fib* Bulletin 80, the actual levels of reliability of EN 1990 to compute the PSFs for wind actions are lower than the target reliability index commonly assumed for ordinary structures (i.e., 3.8). For instance, with reference to the case study, the partial safety factors for wind action evaluated in compliance to DVM and APFM turns out to be higher with respect to the ones of EN1990.

4.2 Discussion

The next longitudinal verifications have been performed:

- ultimate limit states for bending and axial force, according to EN 1992-1;
- ultimate limit states for shear and torsion, according to EN 1992-1;
- ultimate limit state verification of joints and shear keys EN 1992-1.

Table 5. Summary of the partial factors for material properties according to *fib* Bulletin 80 and to EN 1990, EN 1992-1, scenario 3.

Residual service life t_{ref} [y]	*22*		*72*		**EN1990, EN1992**	**Assumptions**
Partial factor	**DVM**	**APFM**	**DVM**	**APFM**		
γ_c	1.35	-	1.28	-	-	V''_c= 0.225, lognormal
γ_s	1.11	-	1.09	-	-	V''_s= 0.075, lognormal
$\gamma_{Rd,c}$	1.21	-	1.21	-	-	-
$\gamma_{Rd,s}$	1.08	-	1.08	-	-	-
ω_c	-	1.10	-	1.04	-	V'_c=0.15; V''_c= 0.225, lognormal
ω_s	-	1.03	-	1.01	-	$V's$=0.05; $V''s$= 0.075, lognormal
γ_C	1.64	1.65	1.55	1.56	1.50	-
γ_S	1.19	1.19	1.17	1.16	1.15	-

In general, the ultimate limit states verifications are fulfilled with reference to the different loading situations EN 1992-1 according to Equation (1) adopting EN 1990, APFM and DVM (*fib* Bulletin 80) methodologies concerning both residual service life equal to 22 year and 72 years.

In the details, *fib* Bulletin 80 is applicable to structures without evidence of fast deteriorating processes in progress. In that case, the fundamental hypotheses which allow to recalculate target reliability accounting for residual service life, economical optimization and human safety criteria are not valid.

In particular, the present structure shows a complex time dependent response due to the creep and shrinkage development, definition delayed restraints during construction stages and prestressing losses. For instance, the verifications have been performed considering the bridge as 28 years old in line with the year of construction (i.e., 1990). Then, the structural analysis and internal actions have been defined according to this hypothesis.

5 CONCLUSIONS

The aim of the present paper consists of assessing existing reinforced concrete structures with reference to the provisions of *fib* Bulletin 80. The methods (i.e., DVM and APFM) proposed in *fib* Bulletin 80 have been used for the assessment of an existing precast reinforced concrete bridge built in 1990. The results have been compared to the outcomes from the assessment carried out according to EN1990 demonstrating the advantages of the two methodologies (i.e., DVM and APFM) and the need to re-calibrate the safety partial factors. In particular, the adoption of reduced target level of reliability with reference to residual service life of the existing structure can lead to easier fulfilment of structural verifications. Moreover, the information related to tests results on material properties may be implemented with a straightforward approach within the assessment process. Finally, the use of semi-probabilistic methods, as APFM and DVM, can avoid the requirement of expensive intervention on existing bridges that, very often, are not able to fulfil target levels of reliability conceived for new structures.

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