Partial factor methods for existing structures according to fib Bulletin 80: Assessment of an existing prestressed concrete bridge

PARTIAL FACTOR METHODS FOR EXISTING STRUCTURES ACCORDING TO *fib* BULLETIN 80: ASSESSMENT OF AN EXISTING PRESTRESSED CONCRETE BRIDGE

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ASSESSMENT OF AN EXISTING BRIDGE WITH *fib* BULLETIN 80

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ABSTRACT

The assessment of existing reinforced concrete structures is one of the major aspects for engineers and practitioners. In particular, existing infrastructures, as bridges and viaducts, are extensively exposed to environmental actions, materials aging, degradation and variation of magnitude of traffic loads during their service life. Hence, the assessment of existing structural systems assuming the same criteria conceived for the design (i.e., partial factor method – EN 1990) can be too conservative and, sometimes, may lead to unnecessary and expensive structural interventions. In this context, *fib* Bulletin 80 defines the partial factor methods suitable for the assessment of existing reinforced concrete structures accounting for their 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. The methodologies proposed in *fib* Bulletin 80 have been applied to assess the safety of an existing pre-stressed reinforced concrete bridge built in 90s and located in Italy. The results are compared to the outcomes from the assessment performed according to EN1990 and, finally, limits and advantages of the methodologies proposed by *fib* Bulletin 80 are discussed.

KEYWORDS: existing structures; partial factor method; reinforced concrete; prestressed concrete; bridges; residual service life.

1. INTRODUCTION

The assessment of existing reinforced concrete structures and infrastructures is one of the major aspects for engineers and practitioners. In particular, existing bridges and viaducts are extensively exposed to environmental actions, time dependent phenomena (e.g., creep, shrinkage, prestressing steel relaxation), degradation (e.g., corrosion due to carbonation and/or chloride penetration) and increasing magnitude of traffic loads. For these reasons, several investigations have been devoted to define methodologies to assess efficiently existing structures and infrastructures also including refined analysis methods (Allen, 1991; Diamantidis and Bazzurro, 2007; Castaldo et al., 2018a,b, 2019; Mancini et al., 2018). Moreover, national Authorities are strongly interested in the assessment of existing structures, with particular care for existing bridges and viaducts, due to the high costs for maintenance, interventions and upgrading.

The partial factor method (PFM) (EN 1990, 2002; ISO 2394, 2015; *fib* Model Code 2010, 2013) according to the semi-probabilistic approach (i.e., level I) for the evaluation of structural reliability is the most efficient methodology adopted by engineers and practitioners for the design and assessment of structures in presence of static and dynamic loads. Partial safety factors are applied both to material properties and actions in order to respect the target reliability levels according to reference service life, economical and human safety criteria (EN 1990, 2002; ISO 2394, 2015; *fib* Model Code 2010, 2013).

The assessment of existing structures and infrastructures differs from the design of new systems due to several reasons. First of all, the reference service life t_r related to an existing structure is, reasonably, represented by the residual service life (e.g., lower than 50-100 years) for which the structure should carry out its functionality, that may differ significantly from the design service life (e.g., 50-100 years (EN 1990, 2002; ISO 2394, 2015; *fib* Model Code 2010, 2013)). Moreover, the costs for upgrading of safety measures for existing structures are higher than the costs required to provide the same measures in the design of new structures. Hence, the target reliability levels related to both economical optimization and human safety criteria should be differentiated in case of assessment of existing structures with respect to the design of new systems. Furthermore, existing structures have provided very often "satisfactory past performances" with regard to relevant loading configurations occurred during their service life. This knowledge should be included in the assessment process, although it is very complex to account for this aspect (Allen, 1993).

In order to perform the assessment according to the limit semi-probabilistic states approach defined by EN 1990 (EN 1990, 2002), an appropriate updating of the partial safety factors is required considering the following aspects: the residual service life, modified target reliability levels for existing structures, the actual condition of the structure as well as the magnitude of variable actions (e.g., material characterization from in situ and laboratory test results, monitoring and measurements of environmental actions and traffic loads for road bridges).

Due to the mentioned above reasons, *fib* Bulletin 80 (*fib* Bulletin N°80, 2016) proposes advanced and efficient methodologies devoted to:

- define target reliability levels suitable for the assessment of existing structures accounting for human safety (i.e., individual and group risk) and economical optimization criteria;

- re-calibrate the partial safety factors for existing structures which should be compatible with the semi-probabilistic framework defined by EN1990 (EN 1990, 2002).

In detail, *fib* Bulletin 80 (*fib* Bulletin N°80, 2016) proposes two methodologies devoted to the re-definition of the partial safety factors for existing structures: the Design Value Method (DVM) and the Adjusted Partial Factor Method (APFM). Both methods are able to evaluate the partial safety factors according to both the residual service life and updated target reliability levels for existing structures. The DVM allows to recalculate the partial safety factors for both material resistances and actions by means of consistent probabilistic models derived by the prior knowledge, test results and observations related to the existing structure under investigation. The APFM, which can be considered as the simpler approach, allows to update the partial safety factors defined by Eurocodes (EN 1990, 2002; CEN EN 1992-1, 2005; CEN EN 1991, 2005; CEN EN 1992-1, 2005) for new structures, by means of "adjustment coefficients" accounting for the prior knowledge, test results and observations related to the existing structure.

In compliance to the two methodologies defined by *fib* Bulletin 80 (*fib* Bulletin N°80, 2016), the present work deals with the assessment of an existing concrete road bridge realized close to Avigliana city in the north of Italy (Piedmont, Turin) in 1990. The bridge is located along the connection between A32 Torino-Bardonecchia highway and SS25 road and crosses the river Dora Riparia. It is a three span (30+60+30 m) precast box section prestressed concrete bridge built through the balanced cantilever technique.

In the following, the basic notions related to partial factor method (EN 1990, 2002) are described together with the methodologies able to evaluate the updated target reliability levels for existing concrete bridges (*fib* Bulletin N°80, 2016). Then, the approaches devoted to define the appropriate partial safety factors for the existing bridges are described (i.e., DVM, APFM (*fib* Bulletin N°80, 2016)) and commented. In addition, the description of the Avigliana's bridge is reported highlighting both the time dependent behaviour of the structural response and the staged construction process. Finally, the results in terms of the redefined partial factors and of safety verifications according to the limit states semi-probabilistic approach are described and compared to the ones achieved according to Eurocodes (EN 1990, 2002; CEN EN 1992-1, 2005; CEN EN 1991, 2005; CEN EN 1992-1, 2005) prescriptions for new structures highlighting advantages, limits and deficiencies of the methodologies proposed by *fib* Bulletin 80 (*fib* Bulletin N°80, 2016).

2. BASIC NOTIONS RELATED TO BOTH THE PARTIAL FACTOR METHOD AND *fib* BULLETIN 80

The limit states semi-probabilistic method, in line with the partial factor format of EN1990 (EN 1990, 2002) and with *fib* Model Code 2010 (*fib* Model Code 2010, 2013), allows to perform the safety verification according to the following equation:

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

where E_d is the design value of the effect of external actions (e.g., internal forces) and R_d is the corresponding design structural resistance. Next, basic notions related to the partial factor format and to the derivation of the partial safety factors according to EN1990 (EN 1990, 2002) and to *fib* Model Code 2010 (*fib* Model Code 2010, 2013) are reported. Finally, the methods proposed by *fib* Bulletin 80 (*fib* Bulletin N°80, 2016) are briefly described.

2.1 The partial factor method according to EN1990

The partial factor method (EN 1990, 2002; ISO 2390, 2015; *fib* Model Code 2010, 2013) is based on the level I (i.e., semi-probabilistic) approach for the evaluation of the structural reliability. The safety measures are applied partially to actions and material resistances by means of appropriate partial safety factors. According to both EN1990 (EN 1990, 2002) and *fib* Model Code 2010 (*fib* Model Code 2010, 2013), concerning the most of the cases, the design value of the resistance of a structural component or system R_d may be evaluated as:

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

$$\gamma_{M,i} = \gamma_{Rd1} \cdot \gamma_{Rd2} \cdot \gamma_{m,i} \tag{3}$$

$$\gamma_{m,i} = \frac{X_{k,i}}{X_{d,i}} = \begin{cases} \dfrac{1-1.645V_{X_i}}{1-\alpha_R \beta V_{X_i}}, & \textit{Normal distribution} \\ \dfrac{exp\left(-1.645V_{X_i}\right)}{exp\left(-\alpha_R \beta_t V_{X_i}\right)}, & \textit{Lognormal distribution} \end{cases} \tag{4}$$

where η_i is the conversion factor in terms of the material resistance relating the test results and the actual structural member value; $X_{k,i}$, $X_{d,i}$ 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 (e.g., 0.15 for concrete cylinder compressive strength and 0.05 for reinforcement yielding strength (*fib* Model Code 2010, 2013)); a_d is the design value of geometrical parameters; γ_m is the partial safety factor for material uncertainty evaluated according to Eq.(4) assuming normal or lognormal probabilistic distributions; $\gamma_{M,i}$ is the partial safety factor accounting for material, geometrical and model uncertainties evaluated according to Eq.(3); γ_{Rd1} is the model uncertainty partial safety factor set equal to 1.05 and 1.025 for concrete and ordinary reinforcement (*fib* Model

Code 2010, 2013), respectively; γ_{Rd2} is the partial safety factor accounting for geometrical uncertainties set equal to 1.05 (*fib* Model Code 2010, 2013).
The design value of the effect of external actions E_d can be evaluated as:

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

$$\gamma_{g,j} = \frac{G_{d,i}}{G_{k,i}} = \frac{1-\alpha_R\beta V_{G_j}}{1-kV_{G_j}} \quad \textit{Normal distribution} \tag{6}$$

$$\gamma_{q,j} = \frac{F_{Q,j,t_{ref}}{}^{-1}\left[\Phi\left(-\alpha_E\beta,t_{ref}\right)\right]}{Q_{k,j}} \tag{7}$$

$$\gamma_{G,j} = \gamma_{Ed}\cdot\gamma_{g,j};\quad \gamma_P = 1.0;\quad \gamma_{Q,j} = \gamma_{Ed}\cdot\gamma_{q,j} \tag{8}$$

where $G_{k,j}$, $G_{d,j}$ are the characteristic (computed assuming k=0 in Eq.(6)) and design values of 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 the annual maxima distribution for climatic actions) value of dominant external action for the selected loading configuration; $\psi_0 Q_{k,j}$ are the combination values of the non-dominant actions for the specific loading configuration; t_{ref} is the reference period (i.e., design service life for new structures and residual service life for existing structures); $F^{-1}_{Q,tref}$ is the inverse of cumulative probabilistic distribution of maxima of the variable action related to t_{ref} (e.g., Gumbel); Φ is the standard normal distribution; γ_{Ed} is the model uncertainty partial safety factor for actions evaluated according to (EN 1990, 2002); $\gamma_{G,j}$, γ_P, $\gamma_{Q,i}$ are the partial safety factors accounting for model and aleatory uncertainties for permanent, prestressing and variable actions, respectively, evaluated according to Eq.(8).
The reliability level is defined through the reliability index β (Hasofer and Lind, 1974), set equal to 3.8 for ordinary structures with 50 years of design service life (EN 1990, 2002; ISO 2390, 2015; *fib* Model Code 2010, 2013). 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, 2002; Hasofer and Lind, 1974; Konig and Hosser, 1962). The methodologies proposed by *fib* Bulletin 80 (*fib* Bulletin N°80, 2016) for existing structures present the same assumptions related to the FORM sensitivity factors as in EN1990 (EN 1990, 2002).

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

As discussed in Section 1, the target reliability levels for existing structures should be differentiated with respect to the ones conceived for new structures. According to (*fib* Bulletin N°80, 2016), the target reliability indices are evaluated accounting for both economic and human safety criteria. In detail, *fib* Bulletin 80 (*fib* Bulletin N°80, 2016) performs the assumption of constant annual target probability of failure within the residual service life of the structure t_{ref}. This hypothesis makes *fib* Bulletin 80 (*fib* Bulletin N°80, 2016) not directly applicable for cases of bridges subjected to active deterioration processes, where, the probability of failure over the years cannot be considered as a constant value. Within *fib* Bulletin 80 (*fib* Bulletin N°80, 2016) short notices for deteriorating bridges are also provided but not exhaustively. Specifically, the fully probabilistic analysis is suggested in the case of assessment of deteriorating structures and structures dominated by time-invariant variables. However, the extension of the partial factor format for existing structures proposed by *fib* Bulletin N°80 (2016) to deteriorating reinforced concrete structures is required in the next future and further research on this topic is necessary. For instance, as discussed in the next sections, the bridge selected for the present investigation does not present any evident deterioration process due to corrosion of both tendons and ordinary reinforcements. This makes it possible to properly apply the methodologies proposed by *fib* Bulletin N°80 (2016).

The optimization concerning the economic criteria is performed accounting for the consequence of structural failure in terms of economic losses, social inconvenience, environmental effects and costs of safety measures adopted in order to reduce the probability of failure. The consequences related to structural failure are accounted for by means of the reliability differentiation approach proposed by (EN 1990, 2002; ISO 2390, 2015; *fib* Model Code 2010, 2013). For instance, three consequence classes (i.e., CC1, CC2 and CC3) are defined with increasing economic, social and environmental consequences due to structural collapse. The target reliability levels concerning the economical optimization can be considered as independent on the residual service life t_{ref} , as discussed by (Vrouwenvelder, 2012). Then, *fib* Bulletin 80 (*fib* Bulletin N°80, 2016) suggests that target reliability levels for economic optimization for existing structures can be defined differentiating between two different values of the target reliability index β_t:

- β_{0t}, target reliability index associated to the existing structure assessed as it is, without any intervention or upgrade. In the case of this target level of reliability is not fulfilled, an upgrade of the structure is required;

- $\beta_{up,t}$, target reliability index to be satisfied after the upgrading of the existing structure.

The human safety requirements for existing bridges are identified in compliance to the individual and group risk criteria (*fib* Bulletin N°80, 2016). The group risk criteria represent one of main advances proposed by *fib* Bulletin 80 (*fib* Bulletin N°80, 2016), in particular, concerning the assessment of existing bridges. Specifically, the group risk criteria allow to take into account the possible number of people at risk as a function of the potential collapsed span of the bridge S and the residual service life t_{ref}. The potential collapsed span S should not be confused with the total span of the existing bridge.

In fact, the potential collapsed span S should be identified by means of an engineering judgment accounting for the actual static scheme of the bridge and the predicted failure modes considering also potential disproportionate mechanisms of collapse due to lack of structural robustness. For increasing potential collapsed spans S, higher is the number of people at risk who can be present at the same time on the bridge. Hence, the related target reliability index to be satisfied turns out to be higher. However, the group risk criteria accounting for human safety is extremely approximate as it is derived from the relationship between the collapsed span of the bridge S and the number of casualties N (i.e, N=0.09S), which is calibrated on the basis of limited data descending from the analysis regarding the failures of ten bridges occurred around the world. The methodology proposed by (*fib* Bulletin N°80, 2016) in order to define the target reliability levels for existing bridges is summarized in Table 1. Note that similar values can be achieved adopting the Life Quality Index (LQI) approach (ISO 2394, 2015; Nathwani et al., 2009; Sykora et al., 2017). In the following, the methodologies proposed by *fib* Bulletin 80 (*fib* Bulletin N°80, 2016) devoted to the evaluation of the partial safety factors for existing structures are described.

2.3 The Design Value Method (DVM)

The Design Value Method (i.e, DVM) (*fib* Bulletin N°80, 2016; Caspeele et al., 2013) allows to recalculate the partial safety factors $\gamma_{M,i}$, $\gamma_{G,j}$ and $\gamma_{Q,i}$ from the actual probabilistic distribution of the variables X_i, G_j and Q_j under consideration (based on prior information or on results from a test or on the combination of both ones). The partial safety factors can be defined according to target reliability levels related to existing structures (i.e., buildings or bridges) and to the expected residual service life t_{ref}. This method is considered as more refined if compared to the APFM, even if for structures of particular relevance higher level methods (e.g., fully probabilistic and risk assessment) are also suggested (ISO 2394, 2015 - *fib* Bulletin N°80, 2016). The DVM leads to results, in terms of values of partial safety factors, discordant to the ones derived by (EN 1990, 2002; *fib* Model Code 2010, 2013; CEN EN 1992-1, 2005) performing the same assumptions concerning the probabilistic model for the involved random variables, especially, concerning the partial safety factors for variable actions.

2.4 The Adjusted Partial Factor Method

The Adjusted Partial Factor Method (i.e, APFM) (*fib* Bulletin N°80, 2016; Caspeele et al., 2013) allows to define the partial factors for existing structures ($\gamma_{Existing}$) adjusting the partial factors $\gamma_{M,i}$, $\gamma_{G,j}$, $\gamma_{Q,i}$ related to new structures (γ_{New}) proposed by EN1990 (EN 1990, 2002), EN1992 (CEN EN 1992-1, 2005) and *fib* Model Code 2010 (*fib* Model Code 2010, 2013) by means of adjustment factors ω as follows:

$$\gamma_{Existing} = \omega \cdot \gamma_{New} \tag{9}$$

The adjustment factor ω accounts for the different target reliability level, the residual service life t_{ref} of the existing structure, prior ad new information from in situ and laboratory tests and measurements of variable actions (e.g., actual traffic loads). The method is fully consistent with (EN 1990, 2002; *fib* Model Code 2010, 2013; CEN EN 1992-1, 2005) provisions if the same hypotheses concerning the probabilistic model for the involved random variables are assumed.
Comparing this method to the previous one, it is observed that the both ones are based on the same fixed values of the FORM sensitivity factors (EN 1990, 2002). The APFM may be considered as the easiest one to be applied in practice and often leads to conservative values of partial safety factors if compared to the DVM. In particular, the DVM provides results that may differ significantly from the APFM ones in the case of significant variation of the probabilistic model for the variable actions with respect to the suggestion of *fib* Bulletin 80. On the contrary, similar results between the two methods are provided concerning partial safety factors for material properties and permanent loads.

3. CASE STUDY OF A PRESTRESSED CONCRETE BRIDGE

The existing bridge herein assessed following the methodologies proposed by *fib* Bulletin 80 (*fib* Bulletin N°80, 2016) is situated in the north of Italy close to Avigliana city. The bridge crosses the river “Dora Riparia” and is located along the connection between SS25 road and “Torino-Bardonecchia” highway (Figures 1-3).
The Avigliana’s bridge is a prestressed precast box section bridge built in 1990 through the balanced cantilever staged construction technique. It is a bridge built with continuous beam static scheme, composed of three spans of 30+60+30m for a total length of 120m. The deck is 9.80m wide and the typical precast segments are 3m high and 3.05m long. The scheme representing the typical segment with the related ordinary reinforcement arrangement is reported in Figure 3. Reinforced concrete diaphragms are located close to the support regions. Statically determined restraints configuration is adopted along lateral and longitudinal directions as showed in Figure 2. The two piers have 15m height from the river bed. Prestressing has been introduced by post-tensioning technique during the different construction stages. Top tendons have been tensioned during the hammers construction with a balanced cantilever static scheme. Bottom tendons have been tensioned when the hammers construction and the closure of the midspan joint were completed turning the overall static scheme in a continuous beam on four supports. Each tendon is composed of 12 strands with diameters of 0.6”.
The main steps representing the construction procedure are reported in Figure 4. All the tendons have been appropriately grouted at the end of the construction process. General information about geometry, segments section and prestressing layout are reported in Figure 1 and Figure 3. All the

details about bridge geometry, material properties and construction stages have been derived from the original drawings and design reports which are completely available. Precisely, it has been possible to identify accurately the construction process which strongly affects the structural behavior during the service life of the bridge due to time dependent phenomena (i.e., concrete creep and shrinkage and relaxation of tendons).

The linear elastic structural model has been defined using SAP2000 (SAP2000, 2002) software platform. Figure 5 reports the location of the beam elements according to the centroid of the cross section. Construction stages have been reproduced accounting for both the immediate and delayed prestressing losses (CEN EN 1992-1, 2005) as well as the creep and shrinkage effects (CEB-FIP Model Code 1990, 1993). The staged construction analysis has been carried out up to the end of the design service life of the bridge, as discussed in Section 4. Prestressing tendons have been modelled according to SAP2000 elements library (SAP2000, 2002) after a preliminary validation process (i.e., comparing the results of a simple model with one straight tendon to the results provided by a hand calculation). The structural effect of the prestressing is considered as an external action within the structural analysis. The permanent (i.e., dead weight, kerbs, barriers) and variable (i.e., wind, traffic, foundation settlements, seasonal and daily thermal) actions have been defined and appropriately combined according to (CEN EN 1991, 2005). The Load Model 1 (CEN EN 1991, 2005) has been adopted in order to model the traffic loads to perform the longitudinal verifications. The multi-component of actions has been considered for the group of actions 1 and 2 in line with (CEN EN 1991, 2005).

The basic mechanical properties are derived from the original design reports and modified according to the different assessment methodologies as described in the following sections. The compressive strength of concrete is related to the concrete strength class C37/45, FeB44k steel has been used for ordinary reinforcements. Prestressing strands with diameter of 0.6” present a characteristic yielding strength equal to $f_{p0.1k}$=1600 MPa and a characteristic ultimate strength f_{ptk}=1800 MPa with and initial stress after tensioning of σ_{p0}=1428 MPa.

4. APPLICATION OF THE METHODS PROPOSED BY *fib* BULLETIN 80

In this section, the two methodologies proposed by *fib* Bulletin 80 (*fib* Bulletin N°80, 2016) are applied in order to perform the assessment of the Avigliana’s bridge.

4.1 Evaluation of the target reliability level for the Avigliana’s bridge

fib Bulletin 80 (*fib* Bulletin N°80, 2016) allows to perform the assessment of existing structures accounting for reduced levels of reliability, residual service life t_{ref} and information deriving from tests and inspections. In the present paper, two assumptions have been performed related to the residual service life of Avigliana’s bridge built in 1990: $t_{ref,1}$=22 years, accounting for an overall design service life of 50 years; $t_{ref,2}$=72 years, accounting for an overall design service life of 100 years. The consequence class selected for the Avigliana’s bridge is CC2. In fact, the bridge is not located along the main axis of the “Torino-Bardonecchia” highway but along the connection with SS25 road, hence, the direct and indirect consequences of structural failure can be considered as ordinary.

On the contrary, if the bridge was located along the main axis of the highway, direct and indirect consequences of structural failure may be significantly higher (i.e., long-time interruption of traffic, deviations on alternative paths), then, CC3 should be recommended.

According to (*fib* Bulletin N°80, 2016), the target reliability level in the hypothesis of CC2 can be evaluated defining the potential collapsed span S accounting for the group risk criteria. Concerning the Avigliana’s bridge, the hypothesis of S set equal to the whole bridge length of 120m is assumed. In fact, this turns out to be a safe and reasonable assumption in case of structural failure of the bridge considered for the investigation.

Finally, in line with both Table 1 and Figure 6, $\beta_{0t} = \beta_{up,t}$ can be set equal to 3.73 and 3.42 assuming a residual service life of 22 years and 72 years, respectively, and the potential collapsed span of 120m.

4.2 Available information related to the bridge

Visual and external inspections have been carried out in order to determine the actual condition of the Avigliana's bridge. One of the most important sources of deterioration of existing post-tensioned bridges is related to corrosion of prestressing tendons that may lead to catastrophic collapses without any significant warning. In fact, when internal bonded tendons are adopted as in the case of the Avigliana's bridge, the quality of the grouting of the ducts is responsible for the protection of the prestressing steel over the service life. If the grouting process is not performed with appropriate care, a local uncomplete grouting of the tendons (in particular in the regions where tendons are subjected to strong curvature and deviations) may create weak points where corrosion may take place. Concerning the present investigation, the Avigliana's bridge is not affected by progressive deterioration processes such as corrosion of both ordinary reinforcement and post-tensioned tendons. Then, the methodologies proposed by *fib* Bulletin 80 (*fib* Bulletin N°80, 2016) can be properly applied (as discussed in Section 2) in order to assess the bridge under consideration.
fib Bulletin 80 (*fib* Bulletin N°80, 2016) suggests to update prior information (') on materials properties accounting for test results in order to get to posterior information ('') by means of a Bayesian approach. Prior information from literature (*fib* Model Code 2010, 2013) are available concerning the coefficient of variation of the concrete compressive strength and of the reinforcement yield strength, that can be set equal to V'_c=0.15 and V'_s=0.05, respectively. Since data from in situ and laboratory tests on concrete and reinforcements are not available, five different significant scenarios concerning the ratios V''/V' have been assumed as reported in Table 2. 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 N°80, 2016). The mean values of material properties have been assumed to be in compliance with the ones reported by the original design reports and remain constant after the updating of the prior distributions. The five scenarios have been selected in order to meet the limits of applicability for the methodologies proposed by *fib* Bulletin N°80 (2016). Specifically, the lowest bounds for V''/V' (i.e., 0.5 for concrete compressive strength and 0.8 for reinforcement yielding strength) have been defined in order to satisfy the hypothesis of dominant random variables for material properties with respect to the model uncertainty. In fact, for lower values of the ratios V''/V' for both concrete and reinforcement, the model uncertainty becomes a dominant variable and some recommendations reported by *fib* Bulletin N°80 (2016) need to be modified for the assessment of existing structures. The highest bounds for V''/V' are set equal to 1.5 as reasonable assumption for existing structures without evidence of deterioration process.
The ordinary reinforcement yield strength and concrete cylinder compressive strength can be modelled as lognormally distributed variables (*fib* Model Code 2010, 2013; *fib* Bulletin N°80, 2016; JCSS, 2001) with parameters adopted in compliance with Table 2. Then, the design values of material properties have been derived in agreement to Eq.(2) adopting the partial safety factors reported in the next sub-section.
As introduced previously, the effect of prestressing tendons is accounted for by means of an external action on the bridge and the additional contribution of resistance due to yielding of the tendons at ultimate condition has been neglected in structural verifications leading to a safe approximation. It can be noted that *fib* Bulletin 80 (*fib* Bulletin N°80, 2016) does not provide any information about probabilistic modelling of mechanical properties of prestressing reinforcements and the related partial safety factors.

4.3 Partial safety factors for material properties according to DVM, APFM and EN1990

The partial safety factors for material properties are listed in Table 3 in compliance with the different scenarios related to the knowledge of the actual condition of the structure. In function of the ratio V"/V', the partial safety factors related to the concrete compressive strength and the reinforcement yield strength can be significantly different from the ones defined by (CEN EN 1992-1, 2005), also depending on the different target level of reliability.
The Scenario 1 is evaluated in agreement with the assumptions performed by (CEN EN 1992-1, 2005) concerning the probabilistic model for variables representing the material properties (Table 2). In this case, the partial safety factors γ_C and γ_S (comprehensive of the related model uncertainties) are consistent with the ones provided by (CEN EN 1992-1, 2005), although the latter ones are evaluated with different target level of reliability (i.e., β=3.8).
The Scenario 2 may represent the situation where, after testing, both concrete and reinforcements show a significant reduction of the coefficient of variation of the related resistances. As a result, the partial safety factors γ_C and γ_S are significantly reduced with respect to the ones provided by (CEN EN 1992-1, 2005) and this may lead to a simpler satisfaction of the structural verifications performing the assessment.
The Scenario 5 is conceived as the opposite to the Scenario 2. For instance, the coefficient of variation of both concrete and reinforcement strengths are higher than the expected ones and the partial safety factors γ_C and γ_S turn out to be higher if compared to the ones given by (CEN EN 1992-1, 2005).
The Scenarios 3 and 4 are derived from the Scenario 2 and 5 providing intermediate results.
The partial safety factors for material properties are derived as a function of the target level of reliability according to Eq.(3) (i.e., lognormal distribution) which is influenced by the reference service life, the consequence class (i.e., CC2) and the potential collapsed span *S* of the bridge.

4.4 Evaluation of the partial factors for actions according to EN1990, DVM and APFM

In the present section, the partial safety factors according to DVM, APFM (*fib* Bulletin N°80, 2016) and EN1990 (EN 1990, 2002) for actions are defined and compared. The methodologies proposed by *fib* Bulletin 80 (*fib* Bulletin N°80, 2016) has been applied according to the assumptions described in the previous sections. The partial safety factors for actions are reported in Table 4 accounting for the two different target reliability levels which refer to 22 and 72 years of residual service life t_{ref}, respectively.
The probabilistic model and the assumptions for the derivation of the partial safety factors for actions are in line with the DVM and APFM (*fib* Bulletin N°80, 2016). Concerning the permanent and variable actions, in Table 4 all the partial safety factors defined according to both the DVM and APFM together with the related probabilistic assumptions are listed. In Table 4 the partial safety factors proposed by (EN 1990, 2002) are also reported.
It can be noted that the partial safety factors related to prestressing and imposed deformations (i.e., thermal actions and foundation settlements) are not discussed by (*fib* Bulletin N°80, 2016) and are assumed in compliance with (EN 1990, 2002) also for safety verification within the DVM and APFM methods.
The APFM seems to be safer than the DVM concerning the partial safety factor for traffic loads. The partial safety factors for the permanent actions are similar to ones provided by (EN 1990, 2002). Significant differences are highlighted for the partial safety factor for wind adopting both the APFM and DVM in comparison to (EN 1990, 2002). In fact, as discussed by (Steenbergen and Vrouwenvelder, 2010) and (*fib* Bulletin N°80, 2016), the actual levels of reliability adopted by (EN 1990, 2002) to compute the partial safety factors for wind actions are lower than the target reliability index commonly assumed for ordinary structures (i.e., β_t=3.8). However, as discusses by (Zitny et al., 2018), the so-called "hidden safety" inherent to the design Codes is able to compensate the mentioned above un-favourable effect, in particular, for large light-weight structures and infrastructures. Then, neglecting the "hidden safety" may lead to a lower level of reliability.

5. ASSESSMENT OF THE BRIDGE: RESULTS AND DISCUSSION

The safety verifications have been developed according to (CEN EN 1992-1, 2005). The following longitudinal verifications have been carried out:
- ultimate limit states for bending and axial force;
- ultimate limit states for shear and torsion;
- ultimate limit states verification of the joints and of the shear keys.

In the following, the design envelope diagrams for internal actions are reported differentiating between the two assumptions related to the residual service life of the bridge (i.e, $t_{ref,1}$=22 years and $t_{ref,2}$=72 years).
The internal actions have been evaluated according to CEN EN 1990 (2005) accounting for prestressing, group of actions 1 – 2, wind, thermal (seasonal and daily) and settlements actions. They have been combined at ultimate limit state (ULS) in compliance with the partial safety factors reported in Table 4. In Figures 7-8 the results in terms of envelope diagrams of internal actions along the span axis are reported according to ULS combination of actions.
The structural verifications and the values of internal actions as well as the time dependent phenomena have been evaluated at the end of the design service life of the bridge (i.e., 50 years for $t_{ref,1}$ and 100 years for $t_{ref,2}$).
Small differences can be recognized between the design internal actions evaluated following the different methodologies. The reason of this result may be explained observing the partial safety factors reported in Table 4. In fact, it can be noted that the differences between the partial safety factors of the different methodologies are quite small (between 1-5%).
In fact, moving from a residual service life of 22 years to a residual service life of 72 years, a decrease of the partial safety factors related to permanent actions corresponds to an increase of the partial safety factors related to variable actions. Moreover, the influence of prestressing, thermal actions and settlements are taken into account with the same partial safety factors for APFM, DVM and EN1990 independently from the choice of the residual service life. It could be noted that the guidance to address these effects is missing and should be provided in a further extension of the *fib* Bulletin 80 (DVM/ APFM). As the traffic loads and their disposition along the transverse section of the deck affect the torsional response of the bridge, Figures 7(c) shows stronger differences between the different methods concerning the torsional internal action. In fact, considering a residual service life of 22 years the partial safety factors for traffic loads differs significantly between APFM, DVM and EN1990. On the contrary, concerning a residual service life of 72 years, the partial safety factors for traffic loads converges to the value of 1.35 and Figure 8(c) does not show any appreciable difference. All the ultimate limit states are fulfilled in the different loading configurations (CEN EN 1991, 2005) according to Eq.(1), following both the APFM and DVM (*fib* Bulletin N°80, 2016) methodologies in the assumption of remaining service life equal to 22 year and 72 years.
The structural verifications turn out to be satisfied also adopting the set of partial safety factors suggested by both EN1990 and EN1992.
In Figure 9, the structural verification in terms of axial force (N) and bending moment (M) is reported for all the 30 typical segments along the bridge (i.e., segment with same cross section with the exclusion of the pier and abutment segments). The points representing the internal actions report the results of the of the application of group of actions 1 considered as dominant and properly combined with prestressing, wind, thermal and settlements actions in line with CEN EN 1991 (2005). The M-N domains have been evaluated accounting for only the concrete transversal section neglecting the additional resistance contribution due to yielding of tendons at ULS. As a result, the sectional resistance for bending and axial force is influenced by the value assumed by the partial safety factor γ_C. The influence of the scenarios selected in order to simulate the information from in situ and laboratory tests on material properties is highlighted in Figure 9. In fact, higher coefficient

of variation for concrete compressive strength implies a higher partial safety factor γ_C with a significant reduction of the M-N domain.
Comparing the results of the DVM and APFM with respect to the residual service life, it can be recognised that the structural verification is more easily satisfied when the reference service life is set equal to 22 years rather than 72 years.
In Figure 10, the comparison between the M-N domains evaluated according to EN1990, APFM and DVM are reported for the different scenarios for material properties and residual service life t_{ref}. It can be observed that for assumptions about the residual service life and some scenarios (i.e., Scenario 5) the M-N domains computed through the APFM and DVM approaches are larger than the domain evaluated according to EN1990 demonstrating, in this specific case study, the usefulness of the approaches proposed by *fib* Bulletin 80.

6. CONCLUSIONS

The aim of the present study consists of assessing existing reinforced concrete structures in compliance with the provisions of *fib* Bulletin 80 by means of the definition of new partial factors able to account for the residual service life, information from in situ and laboratory tests, measurements of variable actions and reduced target reliability levels according to economical and human safety criteria. The methodologies (i.e., DVM and APFM) proposed in *fib* Bulletin 80 have been applied for the assessment of an existing precast box section pre-stressed reinforced concrete bridge built in 1990 and located in north of Italy. The results have been compared to the outcomes from the assessment performed according to EN1990 demonstrating the advantages of the two methodologies (i.e., DVM and APFM) and the need for the re-calibration the partial safety factors. The limits of applicability of *fib* Bulletin 80 have been highlighted and some deficiencies have been recognized. In detail, *fib* Bulletin 80 does not provide probabilistic models in order to update partial safety factors for prestressing and imposed deformations in general (i.e., foundation settlements and thermal actions). The partial safety factor related to the action of prestressing has been set equal to 1.00 according to EN1990 provision. It may be considered as a safe assumption and can be suggested to be adopted in general with the methodologies proposed by *fib* Bulletin 80 (because lower levels of reliability with respect to new structures may lead to values lower than one). Also concerning partial safety factors for settlements and thermal actions, the same values proposed by EN1990 have been adopted. In this case, adopting reduced target levels of reliability for existing structures may lead to significantly lower values for the related partial safety factors if compared to EN1990. In this context, an appropriate calibration is necessary and further research is needed. However, as it is demonstrated within the present paper, *fib* Bulletin 80 is already applicable to the assessment of existing reinforced concrete and prestressed concrete bridges. It may be adopted in order to avoid expensive and un-useful immediate interventions, for example, by means of a reduction of the residual service life of the bridge accepted by the Authorities (i.e., defining a lower target of reliability to be fulfilled in a shorter residual lifetime) and planning over the years the maintenance, the upgrading and/or the demolition with replacement.

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Tables

Table 1. Evaluation of target reliability index for existing bridges according to (*fib* Bulletin N°80, 2016).

a. Economical aspects (*fib* Bulletin N°80, 2016) – valid for building and bridges		
Consequence classes	$\beta_{0t,\ economical}$ [-]	$\beta_{up,t,economical}$ [-]
CC1	1.8	2.8
CC2	2.3	3.3
CC3	2.8	3.8
b. Human safety requirement (*fib* Bulletin N°80, 2016) – valid for bridges		
Consequence classes	$\beta_{0t,human\ safety}$ [-]	$\beta_{up,t,human\ safety}$ [-]
CC1/CC2	$-\Phi^{-1}\left[\frac{2.75\cdot10^{-5}\cdot\left(0.09\cdot S\right)^{-2}\cdot t_{ref}}{0.055}\right]$	
Target reliability index for existing bridges (*fib* Bulletin N°80, 2016)		
Consequence classes	β_{0t} [-]	$\beta_{up,t}$ [-]
CC1/CC2	$max(\beta_{0t,economical},\beta_{0t,human\ saefty})$	$max(\beta_{up,t,economical},\beta_{up,t,human\ saefty})$
CC3 (very small bridges)	1.8	2.8

*t_{ref} is the residual service life

*S is the collapsed span of the bridge accounting for individual and group risk criteria (*fib* Bulletin N°80, 2016)

* Φ is the cumulative standard normal distribution function

Table 2. Scenarios assumed to represent test results for characterization of the material properties.

Scenario	**Concrete**					**Reinforcement**				
	$\frac{V_c''}{V_c'}$	f_{cm}	f_{ck}*2	V''_c	**Probabilistic distribution**	$\frac{V_s''}{V_s'}$	f_{ym}	f_{yk}*2	V''_s	**Probabilistic distribution**
	[-]	[MPa]	[MPa]	[-]	[-]	[-]	[MPa]	[MPa]	[-]	[-]
1	1.00		37	0.15		1.00		430	0.05	
2	0.50*1		42	0.075		0.80*1		437	0.04	
3	1.50	47	33	0.225	*lognormal*	0.80*1	467	437	0.04	*lognormal*
4	0.50*1		42	0.075		1.50		413	0.075	
5	1.50		33	0.225		1.50		413	0.075	

*1lower bounds are defined according to *fib* Bulletin 80 (*fib* Bulletin N°80, 2016) limits of applicability

*25% characteristic values evaluated according to lognormal distribution from the knowledge of the mean value f_m of the coefficient of variation V as follows: $f_k=f_m\cdot exp(-1.645\cdot V)$

Table 3. Summary of the partial factors for material properties.

Partial factor	Residual service life t_{ref}=22ys ($\beta_{0,t}$=3.72)		Residual service life t_{ref}=72ys ($\beta_{0,t}$=3.42)		EN1990 (EN 1990, 2002), EN1992 (CEN EN 1992-1, 2005)	Probabilistic distribution	Assumptions
	DVM (*fib* Bulletin N°80, 2016)	APFM (*fib* Bulletin N°80, 2016)	DVM (*fib* Bulletin N°80, 2016)	APFM (*fib* Bulletin N°80, 2016)			
	[-]	[-]	[-]	[-]	[-]	[-]	[-]
Scenario 1							
γ_c	1.22	-	1.18	-	-	*lognormal*	$V_c = V''_c = 0.15$
γ_s	1.07	-	1.06	-	-	*lognormal*	$V_s = V''_s = 0.05$
$\gamma_{Rd,c}$	1.21	-	1.21	-	-	-	-
$\gamma_{Rd,s}$	1.08	-	1.08	-	-	-	-
ω_c	-	0.99	-	0.96	-	*lognormal*	V'_c=0.15; V''_c=0.15
ω_s	-	1.00	-	0.99	-	*lognormal*	V'_s=0.05; V''_s= 0.05
$\boldsymbol{\gamma_C}$	1.48	1.49	1.43	1.43	1.50	-	-
$\boldsymbol{\gamma_S}$	1.15	1.15	1.14	1.13	1.15	-	-
Scenario 2							
γ_c	1.11	-	1.09	-	-	*lognormal*	$V_c = V''_c = 0.075$
γ_s	1.06	-	1.04	-	-	*lognormal*	$V_s = V''_s = 0.04$
$\gamma_{Rd,c}$	1.21	-	1.21	-	-	-	-
$\gamma_{Rd,s}$	1.08	-	1.08	-	-	-	-
ω_c	-	0.90	-	0.88	-	*lognormal*	V'_c=0.15; V''_c=0.075
ω_s	-	0.98	-	0.97	-	*lognormal*	V'_s=0.05; V''_s=0.04
$\boldsymbol{\gamma_C}$	1.34	1.35	1.31	1.32	1.50	-	-
$\boldsymbol{\gamma_S}$	1.13	1.13	1.12	1.12	1.15	-	-
Scenario 3							
γ_c	1.35	-	1.28	-	-	*lognormal*	$V_c = V''_c = 0.225$
γ_s	1.06	-	1.04	-	-	*lognormal*	$V_s = V''_s = 0.04$
$\gamma_{Rd,c}$	1.21	-	1.21	-	-	-	-
$\gamma_{Rd,s}$	1.08	-	1.08	-	-	-	-
ω_c	-	1.10	-	1.04	-	*lognormal*	V'_c=0.15; V''_c=0.225
ω_s	-	0.98	-	0.97	-	*lognormal*	V'_s=0.05; V''_s= 0.04
$\boldsymbol{\gamma_C}$	1.64	1.65	1.55	1.56	1.50	-	-
$\boldsymbol{\gamma_S}$	1.13	1.13	1.12	1.12	1.15	-	-
Scenario 4							
γ_c	1.11	-	1.09	-	-	*lognormal*	$V_c = V''_c = 0.075$
γ_s	1.11	-	1.09	-	-	*lognormal*	$V_s = V''_s = 0.075$
$\gamma_{Rd,c}$	1.21	-	1.21	-	-	-	-
$\gamma_{Rd,s}$	1.08	-	1.08	-	-	-	-
ω_c	-	0.90	-	0.88	-	*lognormal*	V'_c=0.15; V''_c=0.075
ω_s	-	1.03	-	1.01	-	*lognormal*	V'_s=0.05; V''_s=
$\boldsymbol{\gamma_C}$	1.34	1.35	1.31	1.32	1.50	-	-
$\boldsymbol{\gamma_S}$	1.19	1.19	1.17	1.16	1.15	-	-
Scenario 5							
γ_c	1.35	-	1.28	-	-	*lognormal*	$V_c = V''_c = 0.225$
γ_s	1.11	-	1.09	-	-	*lognormal*	$V_s = V''_s = 0.075$
$\gamma_{Rd,c}$	1.21	-	1.21	-	-	-	-
$\gamma_{Rd,s}$	1.08	-	1.08	-	-	-	-
ω_c	-	1.10	-	1.04	-	*lognormal*	V'_c=0.15; V''_c=0.225
ω_s	-	1.03	-	1.01	-	*lognormal*	V'_s=0.05; V''_s=
$\boldsymbol{\gamma_C}$	1.64	1.65	1.55	1.56	1.50	-	-
$\boldsymbol{\gamma_S}$	1.19	1.19	1.17	1.16	1.15	-	-

Table 4. Summary of the partial safety factors for actions according to *fib* Bulletin N°80 (2016) and to EN1990 (2002).

Partial factor	**Residual service life** t_{ref} *=22ys* ($\beta_{0,t}$=3.72)		**Residual service life** t_{ref} *=72ys* ($\beta_{0,t}$=3.42)		**EN1990** (EN 1990, 2002)	**Probabilistic distribution**	**Assumptions (DVM, APFM)** (*fib* Bulletin N°80, 2016)
	DVM (*fib* Bulletin N°80, 2016)	**APFM** (*fib* Bulletin N°80, 2016)	**DVM** (*fib* Bulletin N°80, 2016)	**APFM** (*fib* Bulletin N°80, 2016)			
	[-]	[-]	[-]	[-]	[-]	[-]	[-]
γ_g	1.26	-	1.24	-	-	*Normal*	V''_g= 0.1
$\gamma_{q,Wind}$	1.53	-	1.63	-	-	*Gumbel*	V''_{vb}= 0.12
$\gamma_{q,Traffic}$	1.13	-	1.19	-	-	*Gumbel*	V''_T= 0.075
$\gamma_{Ed,g}$	1.07	-	-	-	-	-	-
$\gamma_{Ed,q}$	1.12	-	-	-	-	-	-
ω_g	-	0.99	-	0.97	-	*Normal*	$V'_g=V''_g$ =0.1
$\omega_{q,Wind}$	-	1.10	-	1.14	-	*Gumbel*	$V'_{vb}=V''_{vb}$= 0.12
$\omega_{q,Traffic}$	-	0.96	-	1.00	-	*Gumbel*	$V'_T= V''_T$=0.075
$\boldsymbol{\gamma_G}$	1.35	1.34	1.33	1.31	1.35	-	-
$\boldsymbol{\gamma_{Q,Wind}}$	1.71	1.65	1.82	1.71	1.50	-	-
$\boldsymbol{\gamma_{Q,Traffic}}$	1.27	1.29	1.33	1.35	1.35	-	-
$\boldsymbol{\gamma_P}$	1.00*					-	-
$\boldsymbol{\gamma_{G,Settlements.}}$	1.20*					-	-
$\boldsymbol{\gamma_{Q,Thermal\ actions}}$	1.50*						

*values assumed according to EN1991 (CEN EN 1991, 2005), as no information in *fib* Bulletin 80 (*fib* Bulletin N°80, 2016) are provided.