Assessment of an existing prestressed concrete bridge according to the partial factor method for existing structures (fib Bulletin 80)

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Abstract. The assessment of existing reinforced concrete structures is a critical aspect 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, to perform the assessment of existing structural systems assuming the same criteria conceived for the design (i.e., partial factor method – EN 1990) can result to be too conservative and, sometimes, leads to unnecessary and costly structural interventions. In this context, fib Bulletin 80 defines a new partial factor method suitable for the assessment of existing reinforced concrete structures and infrastructures accounting for their 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 proposed in fib Bulletin 80 have been applied to the assessment of an existing precast box section pre-stressed reinforced concrete bridge built in 90s and located in north of Italy. The results are compared to the outcomes from the assessment performed according to EN1990 and, finally, limits and advantages of fib Bulletin 80 methodologies are discussed.

1. Introduction

The assessment of existing reinforced concrete structures is, nowadays, a crucial aspect 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. For this reason, several literature references are devoted to define methodologies useful to assess existing structures and infrastructures [1]-[5].

The partial factor method according to the semi-probabilistic approach [6]-[8] for evaluation of structural reliability is the most common methodology adopted by engineers and practitioners under static and dynamic loads [10]-[20].

fib Bulletin 80 [9] defines methodologies able to re-calculate the partial safety factors for existing reinforced concrete structures (i.e., buildings and bridges) according to updated reliability levels. Existing structures differ for many reasons from new ones. In fact, existing structures have already
fulfilled part of their service life and they should be assessed only for the remaining part. Moreover, costs for upgrading of existing structures are higher than costs for building a new structure. For these reasons, target reliability levels for existing structures should be re-assessed according to consequences of structural failure, economical optimization, individual and group risk criteria.

\textit{fib} Bulletin 80 [9] 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).

The present paper presents the assessment of an existing precast prestressed reinforced concrete bridge located in Italy applying the methodologies proposed by \textit{fib} Bulletin 80 [9]. The results, in terms of partial factors definition and safety verifications are compared to the ones obtained according to [6], [21] prescriptions for new structure also highlighting some critical aspects.

2. Fib Bulletin 80: partial factor methods for existing structures

The limit states semi-probabilistic method according to the partial factor format (PF format) defined by EN1990 [6] allows to perform the safety verification according to the following expression:

\[ R_d \geq E_d \]  

(1)

where \( E_d \) is the design value of the effect of external actions (e.g., internal forces) and \( R_d \) is the related design structural resistance. In the following, basic notions related to the partial factor format (PF format) and to the partial safety factors (PSFs) derivation according to EN1990 [6] are reported. Subsequently, the methods proposed by \textit{fib} Bulletin 80 are briefly described.

2.1. Basics of the partial factor format according to EN1990

The PF format [6]-[8] is defined according to the Level I (i.e., semi-probabilistic) methodology for evaluation of structural reliability. The safety measures are applied partially to loads and material resistances by means of PSFs. According to [6], in most of cases, the design value of resistance of a structural component or system \( R_d \) may be evaluated as:

\[ R_d = R \left\{ \eta_i \frac{X_{k,i}}{\gamma_{M,i}} \right\} \frac{a_d}{\gamma_{Rd1} \cdot \gamma_{Rd2} \cdot \gamma_{M,i}} \]

(2a,b,c)

where \( \eta_i \) is the conversion factor; \( X_{k,i} \) and \( X_{d,i} \) are the characteristic (i.e., 5% quantile) and the design value of the material property, respectively; \( V_{X_i} \) is the coefficient of variation of the material property; \( a_d \) is the design value of geometrical parameters; \( \gamma_m \) is the PSF for material uncertainty evaluated according to Eq.(2c) assuming a normal probabilistic distribution; \( \gamma_{M,i} \) is the PSF accounting for material, geometrical and model uncertainties evaluated according to Eq.(2b); \( \gamma_{Rd1} \) is the model uncertainty PSF set equal to 1.05 and 1.025 for concrete and reinforcement [21], respectively; \( \gamma_{Rd2} \) is the PSF accounting for geometrical uncertainties set equal to 1.05 [21].

The design value of the effect of external actions \( E_d \) can be evaluated as:
\[ E_d = E\{\gamma_G, j, \gamma_P, j; \gamma_Q, i; \psi, j\} \quad i > 1; \quad j \geq 1 \quad (3) \]

\[
\gamma_G, j = G_{d,j} / G_{k,j} = \frac{1 - \alpha_p BV_{G_j}}{1 - kV_{G_j}} \quad (4a,b) \\
\gamma_Q, j = F_{Q_{k,j}, t_{ref}}^{-1}\left[ \Phi\left(-\alpha_E \beta_{ref}\right)\right] / \gamma_{VE,j} \\
\gamma_G, j = \gamma_{Ed} \cdot \gamma_{G,j} \\
\gamma_P = 1.0 \\
\gamma_Q, j = \gamma_{Ed} \cdot \gamma_{Q,j} \quad (5a,b,c) \\
\]

where \( G_{k,j}, G_{d,j} \) are the characteristic (computed assuming \( k=0 \)) and design values of permanent actions; \( V_{Gj} \) is the coefficient of variation of 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; \( \psi_{Qd,Q_{k,q}} \) are the combination values of the non-dominant actions for the selected loading configuration; \( t_{ref} \) is the reference period (i.e., design service life for new structures and residual service life for existing structures); \( F_{Q_{k,j}, t_{ref}}^{-1} \) is the inverse of cumulative probabilistic distribution of maxima of variable action related to \( t_{ref} \) (e.g., Gumbel); \( \Phi \) is the standard normal distribution; \( \gamma_{Ed} \) is the model uncertainty PSF for actions evaluated according to [6]; \( \gamma_{G,j}, \gamma_P, \gamma_Q \) are the PSFs accounting for model and aleatory uncertainties for permanent, prestressing and variable actions, respectively, evaluated according to Eq. (4a-b) and Eq. (5a-c). The level of reliability is defined by the reliability index \( \beta \), 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 \( \alpha_p \) and \( \alpha_E \) 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 [6]. The methodologies proposed by fib Bulletin 80 [9] for existing structures perform the same assumptions related to the FORM sensitivity factors as in EN1990 [6].

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 differentiated with respect to the ones conceived for new structures [9]. Two target reliability indexes are defined: \( \beta_{0t} \), 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 \( \beta_{0t} \) is not satisfied, upgrading of the structure is required. Concerning existing reinforced concrete bridges, the target reliability levels are suggested by [9] accounting for consequences of structural failure, economic, human safety, individual and group risk criteria 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 \( \beta_{0t} \) and \( \beta_{up,t} \) can be, respectively, evaluated as:

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

2.3. The Design Value Method (DVM)

The Design Value Method (i.e., DVM) [9], [22] allows to recalculate the partial safety factors \( \gamma_{M,i}, \gamma_{G,j} \) and \( \gamma_{Q,j} \) from the actual probabilistic distribution of the variables \( X_i, G_j \) and \( Q_j \) under consideration (based on prior information, or results from tests or the combination of both). The new partial factors can be
defined according to target levels of reliability related to existing structures (i.e., buildings or bridges) and expected residual service life $t_{ref}$. This method is considered as the most refined and it is suggested for structures of particular relevance and may leads to results, in terms of values of partial factors, discordant to the ones derived by [6], [8], [21].

2.4. The Adjusted Partial Factor Method (APFM)

The Adjusted Partial Factor Method (i.e., APFM) [9], [22] allows to define partial factors for existing structures ($\gamma_{Existing}$) adjusting partial factors $\gamma_{M,i}$, $\gamma_{G,j}$, $\gamma_{Q,i}$ related to new structures ($\gamma_{New}$) proposed by EN1990 [6] and EN1992 [21] by means of adjustment factors $\omega$ as follows:

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

The adjustment factor $\omega$ accounts for the different target reliability and the residual service life $t_{ref}$ of the existing structure. The method is fully consistent with EN1990 [6] provisions when the same assumptions are performed, and it is considered as a simplification if compared to the DVM.

3. Study case: Avigliana’s Bridge

The study case of existing bridge selected for fib Bulletin 80 [9] application is located in Italy close to the city of Avigliana, along the junction between the SS25 and the “Torino-Bardonecchia” highway.

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]

The bridge, which crosses the river “Dora Riparia”, is a prestressed precast box section bridge built in 1990 through the balanced cantilever staged construction technique. It is 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. Reinforced concrete diaphragms are located close to supports regions. Isostatic restraints configuration is adopted along lateral and longitudinal directions.
The central 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 balanced cantilever static scheme. Bottom tendons have been tensioned after the hammer construction and closure of the midspan joint turning the overall static scheme in a continuous beam on four supports. Each tendon is composed of 12 strands of 0.6” of diameters.

General information about geometry, segments section and prestressing layout are reported in Figure 1. All the details about bridge geometry, material properties and construction stages has been derived from the original drawings and design reports. The linear elastic structural model has been defined using SAP2000 [23] software platform. Construction stages have been reproduced accounting for immediate and delayed prestressing losses [21] as well as creep and shrinkage effects [24]. The staged construction analysis has been carried out up to the end of the design service life of the bridge, as discussed in the next Section. Prestressing tendons has been modelled according to SAP2000 [23] elements library after preliminary validation process and their effect are considered from actions side within structural analysis and verifications. The permanent and variable (i.e., wind, traffic, foundation settlements, seasonal and daily thermal) actions have been defined and combined according to [25]. Concerning material characterization, concrete compressive strength class C37/45 and ordinary reinforcements FeB44k have been used. The prestressing strands with 0.6” of diameter present a characteristic yielding strength equal to $f_{p0.1k}=1600$ MPa and a characteristic ultimate strength $f_{puk}=1800$ MPa with an initial tension of $\sigma_{p0}=1428$ MPa.

4. Assessment of the Avigliana’s bridge according to fib Bulletin 80

4.1. Determination of the partial safety factors for assessment

fib Bulletin 80 [9] allows to assess 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 original design service life of 50 years; $t_{ref,2}=72$ years, accounting for an original service life of 100 years. According to [9], the target reliability in the hypothesis of CC2 can be evaluated assuming a potential collapsed span $S$ equal to the bridge length of 120m. In this way, according to Eq.(6), $\beta_{up,t}$ can be set equal to 3.73 and 3.42 assuming residual service life of 22 years and 72 years, respectively. Concerning material properties, prior information from literature [8] are available concerning 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. Since data from in situ and laboratory tests on concrete and reinforcements steel was not available, three significant scenarios concerning the ratios $V''/V'$ have been identified as reported in Table 1.

The symbol $V''$ represents the coefficient of variation of the posterior probabilistic distribution of the material property after Bayesian updating of the prior distributions characterised by the coefficient of variation $V'[9]$. The mean values of material properties have been assumed to be in compliance with the ones prescribed at the design stage, and remain constant after the updating of the prior distributions.
Table 1. Scenarios assumed for material properties

| Scenario | $V''/V'$ | $f_{cm}$ [MPa] | $f_{ck}$ [MPa] | $V_c$ [MPa] | $V''/V'$ | $f_{cm}$ [MPa] | $f_{ck}$ [MPa] | $V_s$ [MPa] |
|----------|----------|----------------|----------------|----------------|----------|----------------|----------------|------------|
| Scenario 1 | 1.00     | 37             | 430            | 1.00, lognormal | 0.15, lognormal |
| Scenario 2 | 0.50*    | 47             | 0.04, lognormal | 430            | 0.075, lognormal |
| Scenario 3 | 1.50     | 33             | 0.075, lognormal | 413            | 0.225, lognormal |

*lower bounds are defined according to fib Bulletin 80 [9] limits of applicability

Table 2. Summary of the partial factors for actions according to [9] and to [6]

| Residual service life $t_{ref}$ [y] | 22 | 72 | EN1990 [6] | Assumptions [9] |
|------------------------------------|----|----|------------|-----------------|
|                                     | DVM [9] | APFM [9] | DVM [9] | APFM [9] |
| $\gamma_g$ | 1.26 | - | 1.24 | - | - |
| $\gamma_q,Wind$ | 1.53 | - | 1.63 | - | - |
| $\gamma_q,Traffic$ | 1.13 | - | 1.19 | - | - |
| $\gamma_{Ed,g}$ | 1.07 | - | - | - | - |
| $\omega_g$ | 0.99 | - | 0.97 | - | - |
| $\omega_q,Wind$ | 1.10 | - | 1.14 | - | - |
| $\omega_q,Traffic$ | 0.96 | - | 1.00 | - | - |
| $\gamma_{P}$ | 1.35 | 1.34 | 1.33 | 1.31 | 1.35 |
| $\gamma_{P}$ | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| $\gamma_{Q,Imposed def.}$ | 1.20 | 1.20 | 1.20 | 1.20 | 1.20 |
| $\gamma_{Q,Wind}$ | 1.71 | 1.65 | 1.82 | 1.71 | 1.50 |
| $\gamma_{Q,Traffic}$ | 1.27 | 1.29 | 1.33 | 1.35 | 1.35 |

*values assumed according to EN1991[25], as are not given in fib Bulletin 80 [9]

In Tables 1-2, the partial factors derived in compliance with [9] and [6], 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. In Table 2 are also reported PSFs proposed by [6]. It can be noted that PSFs related to prestressing and imposed deformations (i.e., thermal actions and foundation settlements) are not discussed by [9] and are assumed in compliance with [6] 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 [6]. Significant differences are highlighted for the wind PSF adopting APF and DVM if compared to [6]. This is due to the fact that, as discussed by [26] and [9], the actual
levels of reliability [6] to compute the PSFs for wind actions are lower than the target reliability index commonly assumed for ordinary structures (i.e., 3.8).

Table 3. Summary of the partial factors for material properties according to [9] and to [6], [21]; scenario 1

| Residual service life | 22 | 72 | EN1990[6], EN1992[21] | Assumptions [9] |
|-----------------------|----|----|----------------------|-----------------|
| Partial factor        | DVM | APFM | DVM | APFM |                   |
| $\gamma_c$            | 1.22 | - | 1.18 | - | V$''_c$ = 0.15, lognormal |
| $\gamma_s$            | 1.07 | - | 1.06 | - | V$''_s$ = 0.05, lognormal |
| $\gamma_{Rd,c}$       | 1.21 | - | 1.21 | - | - |
| $\gamma_{Rd,s}$       | 1.08 | - | 1.08 | - | - |
| $\omega_c$            | - | 0.99 | - | 0.96 | - |
| $\omega_s$            | - | 1.00 | - | 0.99 | - |

| $\gamma_c$            | 1.48 | 1.49 | 1.43 | 1.43 | 1.50 | - |
| $\gamma_s$            | 1.15 | 1.15 | 1.14 | 1.13 | 1.15 | - |

Table 4. Summary of the partial factors for material properties according to [9] and to [6], [21]; scenario 2

| Residual service life | 22 | 72 | EN1990[6], EN1992[21] | Assumptions [9] |
|-----------------------|----|----|----------------------|-----------------|
| Partial factor        | DVM | APFM | DVM | APFM |                   |
| $\gamma_c$            | 1.11 | - | 1.09 | - | V$''_c$ = 0.075, lognormal |
| $\gamma_s$            | 1.06 | - | 1.04 | - | V$''_s$ = 0.04, lognormal |
| $\gamma_{Rd,c}$       | 1.21 | - | 1.21 | - | - |
| $\gamma_{Rd,s}$       | 1.08 | - | 1.08 | - | - |
| $\omega_c$            | - | 0.90 | - | 0.88 | - |
| $\omega_s$            | - | 0.98 | - | 0.97 | - |

| $\gamma_c$            | 1.34 | 1.35 | 1.31 | 1.32 | 1.50 | - |
| $\gamma_s$            | 1.14 | 1.13 | 1.12 | 1.12 | 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 [21], also depending on the different target level of reliability.
Table 5. Summary of the partial factors for material properties according to [9] and to [6],[21]; scenario 3

| Residual service life $t_{ref}[y]$ | 22  | 72  | EN1990[6], EN1992[21] | Assumptions [9] |
|-----------------------------------|-----|-----|----------------------|------------------|
| Partial factor                    | DVM | APFM| DVM                  | APFM             |                  |
| $\gamma_c$                        | 1.35| -   | 1.28                 | -                | $V''_{c}=0.225$, lognormal |
| $\gamma_s$                        | 1.11| -   | 1.09                 | -                | $V''_{s}=0.075$, lognormal |
| $\gamma_{rd,c}$                   | 1.21| -   | 1.21                 | -                | -                |
| $\gamma_{rd,s}$                   | 1.08| -   | 1.08                 | -                | -                |
| $\omega_c$                        | -   | 1.10| - 1.04              | -                | $V'_{c}=0.15; V''_{c}=0.225$, lognormal |
| $\omega_s$                        | -   | 1.03| - 1.01              | -                | $V'_{s}=0.05; V''_{s}=0.075$, lognormal |
| $\gamma_C$                        | 1.64| 1.65| 1.55                 | 1.56             | 1.50             | -                |
| $\gamma_S$                        | 1.19| 1.19| 1.17                 | 1.16             | 1.15             | -                |

4.2. Discussion and assessment results

The safety verifications have been defined according to [27]. The following longitudinal verifications has been carried out:

- ultimate limit states for bending and axial force, according to [27];
- ultimate limit states for shear and torsion, according to [27];
- ultimate limit state verification of joints and shear keys [27].

All the ultimate limit states are fulfilled in the different loading configurations [25] according to Eq.(1) adopting [6], APFM and DVM [9] methodologies. Criticisms are not recognized both in the assumption of remaining service life equal to 22 year and 72 years.

However, fib Bulletin 80 [9] is applicable to structures without evidence of fast ongoing deteriorating processes. In fact, in that case, the basic hypotheses which allow to recalculate target reliability accounting for remaining service life, economical optimization and human safety criteria are no more valid.

As the present structure presents a complex time dependent behavior due to the creep and shrinkage evolution, delayed restraints and prestressing losses, all the verifications have been performed considering the bridge as 28 years old in line with the year of construction (i.e., 1990). Then, the staged construction analysis and internal actions have been defined according to this hypothesis.

5. 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 to 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 to re-calibrate the safety partial factors.

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