

# Bridge Case Studies on the Assignment of Partial Safety Factors for the Assessment of Existing Structures



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**Abstract** Aging bridges in combination with an ever-growing traffic volume are a matter of concern all over the world. Consequently, the reassessment of existing bridges is gaining importance rapidly. This paper presents two bridge case studies considered within the IABSE Task Group 1.3 “Calibration of Partial Safety Factors for the Assessment of Existing Bridges”. The so-called design value method (DVM) and adjusted partial factor method (APFM), introduced in fib Bulletin 80 and both relying on a partial factor format, are considered in this paper. The objectives are (i) to illustrate how DVM and APFM can be used when specifying partial safety factors for assessment of existing bridges, and (ii) to discuss some of the assumptions that are implied by these methods. Two case studies are considered for illustration in

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this paper: a single span reinforced concrete slab and a 3-span continuous reinforced concrete slab.

**Keywords** Existing structures · Bridges · Case study · Reinforced concrete · Partial safety factors

## 1 Introduction

The approach for the assessment of existing structures is in many aspects different from that for the design of new structures [1, 2]. The potentially available information about geometry, material properties, loading and environmental conditions is not the same for new and existing structures and this implies different levels of uncertainty. Reliability requirements might also be different for new and existing structures based on societal, cultural, economic, and sustainability considerations. A partial safety format applied for existing structures should cover the aforementioned differences in an appropriate way, to avoid non-effective decisions that in the end may have severe economic, environmental and socio-political consequences [2].

With the objective to promote the fields of structural reliability at an inter-association level, the IABSE TG1.3 is currently investigating how semi-probabilistic formats can be used for assessment of existing structures. In particular, the so-called Design Value Method (DVM), and the Adjusted Partial Factor Method (APFM) that were introduced within *fib* COM3 TG3.1 [3] are considered herein for specifying partial safety factors for reassessment.

This paper presents some first results of an ongoing work within IABSE TG1.3 and is organised as follows. First, DVM and APFM concepts are briefly revisited. Second, the two methods are applied on two different bridge case studies to illustrate how partial safety factors are specified. In the first case study, some measurements on the bridge are available, while the second case study only considers project documentation, without additional on-site measurements. Then the potential and issues of the two methods are discussed.

## 2 Methods for Updating Partial Factors in Bulletin 80

### 2.1 General Considerations

At present, existing structures are mostly verified using a partial factor format with identical safety factors as commonly applied in the design of new structures. Such assessments are often conservative and may lead to expensive structural repairs and interventions. However, especially if the existing structure is associated to material deterioration that cannot be observed directly, the uncertainties in the assessment situation are larger compared to the design situation and the application of the design

partial safety factors might lead to non-conservative decision about the structure. The *fib* Bulletin 80 [3] provides a technical recommendation containing a format with adjusted partial factors for the assessment of existing concrete structures applicable to common situations re-assessment situations. The bulletin intends to be consistent with the background documents of the Eurocodes, ISO 2394:2015, CEB bulletins on the derivation of partial factors for concrete structures and JCSS publications [2, 4]. The following cases remain beyond the scope of the bulletin: structures severely affected by deterioration, non-linear FE models requiring specific safety formats, and compliance demonstrated by proof loading or monitoring. Such additional applications are however envisaged for the *fib* Model Code 2020 which is currently under development and takes basis in a similar philosophy.

When assessing existing structures, a partial factor format may be used, provided that additional information related to material parameters, loading conditions, localised structural defects, etc., is properly accounted for. Two methods relying on a partial safety factor format are provided in the bulletin: the design value method (DVM—general semi-probabilistic format for specifying partial factors, tied to ISO 2394 and EN 1990) and the adjusted partial factor method (APFM—partly a graphical procedure with pre-selected default values, based on DVM), both enabling the incorporation of specific semi-probabilistic aspects for existing structures.

The two methods utilise the generalised sensitivity factors for dominant and non-dominant resistance and load parameters,  $\alpha_R$  and  $\alpha_E$  respectively, as given in ISO 2394:2015 and EN 1990:2002 (see Table 1). Consideration of these generalised sensitivity factors makes it possible to adjust partial factors with respect to a modified reliability requirement  $\beta$  and the updated probabilistic representation of load and resistance variables. The simplifying assumption of the fixed sensitivity factors avoids the situation of having a wide range of different case-specific sensitivity factors which could make practical engineering applications cumbersome. The bulletin indicates  $\beta$ -values based on economic optimisation, and individual and group risk criteria on

**Table 1** Basic variables and corresponding sensitivity factors as treated in detail in *fib* Bulletin 80

Basic variable	Distribution	Sensitivity factor, $\alpha$
Concrete compressive strength, $f_c$	lognormal or normal (LN or N)	0.8
Yield strength of steel reinforcement, $f_y$	LN or N	0.8
Permanent action, $G$	N	-0.7 for unfavourable, 0.32 for favourable
Selected types of variable actions (snow, wind, imposed loads in buildings, and traffic loads on road bridges), $Q$	Gumbel (maxima)	-0.7
Resistance and load effect model uncertainties, $\theta_R$ and $\theta_E$ respectively	LN or N	0.32 for $\theta_R$ , -0.28 for $\theta_E$

a lifetime basis; these levels being systematically lower compared to those provided in EN 1990:2002 for structural design.

DVM and APFM offer generally applicable equations and graphs to update partial factors for the basic variables and specified generalised sensitivity factors as indicated in Table 1. Specific equations and graphs are provided in [3]. Partial factors should be specified assuming appropriate distributions of the variables under consideration (based on prior information, or results of tests or the combination of both). In the bulletin the coefficient of variation,  $V$ , is considered as the major parameter for representing the new state of information about the variable. Furthermore, the partial factors can also be specified on the basis of modified reliability requirement and reference periods. Partial factors for model uncertainty are taken into account, relying on the assumption that resistance and load effect model uncertainties are the same for new and existing structures (i.e. assuming that the same models with same (epistemic) uncertainties are applied in design and assessment). These factors also include uncertainty in a decisive geometrical property such as the effective depth of a beam or the sectional area of a column.

## 2.2 DVM Versus APFM

The DVM proposes the fundamental basis for specifying partial factors. In contrast, the APFM provides adjustment factors to be applied on the partial factors for new structures in EN 1990:2002. The key inputs are the reliability requirement  $\beta$ , the coefficient of variation of the basic variable  $X$ ,  $V_X$  and the reference period that corresponds to the reliability requirement  $\beta$ ,  $t_{ref}$ .

Both methods essentially derive the partial factors for a material property and loads in a similar way while differing slightly in treating of model uncertainty factors. As the DVM can incorporate more structure-specific information (including possible biases in basic variables, i.e. deviations from common ratios between the mean and characteristic value of the basic variable), a specified reliability  $\beta$  may be achieved more accurately than for the APFM. The partial factors obtained by the DVM may be considerably different from those given in the Eurocodes if additional choices are made with respect to input parameters, i.e. additional measurements or investigations are undertaken.

The APFM is tailor-made for everyday use in practice and provides adjustment factors to be applied to the partial factors used in design. The adjustment factors take additional information into account when it is available. However, the adjustment factors are fixed, based on calibrated options for the input variables and hence also relying on these assumptions, while emphasising that collecting structure-specific information is strongly recommended.

The APFM may be seen to yield a more robust approach with respect to practical applications, i.e. with reduced influence of subjectivity, but also with limited regard to structure-specific conditions. The *fib* Bulletin 80 [3] provides the representative values of statistical characteristics of the basic variables listed in Table 1.

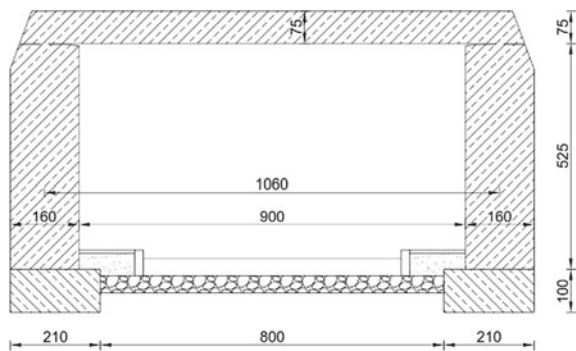
These simplified recommendations are deemed to cover most common cases and ensure the compatibility with Eurocodes. Due to the approximations accepted in the APFM, it often provides a more conservative format in comparison with the DVM. For the assessment of common structural elements, the APFM is particularly appropriate for normative purposes and calibration studies, considering the reduced amount of data required and the consistency with the Eurocode partial factors for the design of new structures. Nevertheless, recalibration is needed when the underlying Eurocode partial factors for structural design change. For important structures with large failure consequences and in the case of expensive upgrades, it is both justified and recommended to obtain additional case-specific information and to use a more advanced method for reliability verification. Use of the DVM is then recommended and if needed, higher level methods such reliability-based or risk-informed assessments might be conducted. Further to it, when reliability is significantly influenced by climatic actions, the use of the DVM is also recommended, provided that relevant measurements are available.

### 3 Case Study of Single Span Reinforced Concrete Slab

This case study corresponds to a highway bridge in Germany. The structure is a single span reinforced concrete slab of 75 cm thickness. The slab is connected to the abutments by concrete hinges, therefore it may be considered a simple supported beam with 10.6 m span (Fig. 1). The bridge was constructed approximately 50 years ago according to the German code in place at the time. Concrete grade corresponds to C20/25 according to the Eurocodes.

The bridge has been recalculated based on the reassessment guideline for existing bridges [5]. In addition to provisions regarding the consideration of outdated material grades, modified traffic load models and alternative design procedures, the guideline provides specifications for reduced partial safety factors. As a consequence among other alterations the traffic load model considered in the original structural design implied lower values than those currently required by the German national annex and

**Fig. 1** Longitudinal section of the bridge selected as the first case study



**Table 2** Partial factors for new structures and within German reassessment guideline [5]

Partial factors	New structure	German reassessment guideline
Concrete compressive strength	1.5	1.5
Reinforcing steel	1.15	1.05
Prestressing steel	1.15	1.1
Permanent loads	1.35	1.2
Traffic loads	1.35	1.5
Other live loads	1.5	1.5

some partial safety factors were also reduced. Table 2 details partial safety factors for new structures and those used for assessment of existing structures following [5]. It should be noted, that reduced partial safety factors for reinforcing and prestressing steel are only permitted for bending, if simultaneously the effective depth is modified unfavorably by 2.0 cm and 1.0 cm respectively. Furthermore, in the calculation of the design compressive strength the coefficient  $\alpha_{cc}$  is 0.85 for Germany instead of the recommended value of 1.0 in the Eurocodes. The adjusted partial safety factor for permanent loads requires that the cross-section dimension and concrete density have been determined by measurements on the investigated structure. The apparent increase in the partial safety factor for traffic loads is misleading, since the guideline is based on the traffic loads of previous generations of the German standard where lower traffic loads were combined with a partial safety factor of 1.5. In contrast, the latest German national annex of EN 1991-2 assumes significantly higher traffic loads for road bridges, but also a partial safety factor of 1.35 (instead of 1.5), thus effectively emphasizing the serviceability limit states.

With these provisions the structure showed a just sufficient load bearing capacity both for bending and shear and could be approved for continued operation. If the currently valid traffic load model would be applied, the actions would exceed the resistances by approximately 15%.

It is quite common in Germany to obtain concrete core samples from bridges subjected to recalculation. These samples provide more specific data on the material properties of concrete but also on the density and therefore allow for an update on the available information both for actions and resistances. Consequently, the provisions of the *fib* Bulletin 80 [3] are demonstrated for the bridge based on the concrete compressive strength and the concrete density.

The target reliability index may be adjusted for existing structures. When relying on an economic consideration only, the *fib* Bulletin 80 [3] mentions the reduction of reliability indices related for the design working life accepted for new structures by about  $\Delta\beta = 1.5$ . Since the failure of the present bridge would have severe economic consequences, it may be categorised into consequences class CC3 and the minimum target reliability is  $\beta_0 = 4.3 - 1.5 = 2.8$  (when considering as basic value the one provided in EN 1990:2002, see Table 3.2–1 in [3]). This value is considered in the

following for illustration. It is based on economic optimization with a reference period equal to the remaining lifetime and is independent of the latter [3].

The six core samples taken from the structure have shown a mean value for in-situ compressive strength of 66.5 N/mm<sup>2</sup> and high coefficient of variation of 0.21. An updated characteristic value is  $66.5 \times (1 - 2.18 \times 0.21) = 36.1$  N/mm<sup>2</sup>, considering statistical uncertainties due to a small sample size of only six core samples by considering a value of  $k_n = 2.18$  according to Table D1 in Annex D of EN 1990.

For DVM, the assumption of a lognormal distribution for concrete strength leads to the following partial safety factor:

$$\gamma_C = \gamma_{Rd} \cdot e^{\alpha_R \cdot \beta \cdot V_x - 1.645 \cdot V_x} = e^{0.4 \cdot 0.8 \cdot 2.8 \cdot 0.14} \cdot e^{0.8 \cdot 2.8 \cdot 0.21 - 1.645 \cdot 0.21} = 1.28 \quad (1)$$

where  $\gamma_{Rd}$  is the partial factor accounting for model uncertainty,  $\alpha_R$  is the sensitivity factor for resistances,  $\beta$  is the reliability index and  $V_x$  is the coefficient of variation for the material property. In Eq. (1), one assumes a lognormal distribution for the model uncertainties which are expressed as  $\gamma_{Rd} = \mu_{\theta_R} / \theta_{Rd} = e^{\alpha_{\theta_R} \cdot \beta \cdot V_{\theta_R}}$ , assuming that  $\mu_{\theta_R} = 1.0$  and that  $V_{\theta_R} \approx 0.14$  for concrete, when geometrical uncertainties are significant [3].

Since there was no actual data on the density of the core samples for the selected case study, a data set with a mean of 2362 kg/m<sup>3</sup> and a coefficient of variation of 0.021 was considered from another similar structure for the purpose of demonstration. It shall be noted, that in a different study [6] a comparison of values for the concrete density for five different bridges recalculated in Germany showed a remarkable consistency in the coefficient of variation staying around 0.02. The partial safety factor for a normal distribution in case of a dominant variable based on the design value method is calculated as:

$$\begin{aligned} \gamma_G &= \frac{G_d}{G_k} \gamma_{Ed,G} = \frac{(1 - \alpha_E \cdot \beta \cdot V_G)}{(1 - k \cdot V_G)} e^{-\alpha_E \cdot \beta \cdot V_{\theta_E}} \\ &= \frac{1 - (-0.7) \cdot 2.8 \cdot 0.021}{1 - 0 \cdot 0.021} e^{0.4 \cdot 0.7 \cdot 2.8 \cdot 0.10} = 1.13 \end{aligned} \quad (2)$$

where  $\gamma_{Ed,G}$  is the partial factor accounting for model uncertainty (assuming a lognormal distribution for model uncertainties and  $V_{\theta_R} = 0.10$  for bending and shear [3]). It is highlighted that  $k = 0$  in Eq. 2 as the *fib* Bulletin 80 [3] recommends this value for permanent actions (the mean value is chosen as the representative value). Note that commonly  $V_G = 0.05$  is considered for self-weight of concrete structures.

In case of the adjusted partial factor method (APFM), the regular partial safety factors are modified by an adjustment factor  $\omega_y$ . The adjustment factor is determined by the target reliability index and the ratio between the coefficient of variation for new structures and the actual value obtained on the structure. The adjustment factors for the current case study were calculated using the diagrams provided in the *fib* Bulletin 80 [3] as shown in Fig. 2 (and thus considering the limit for  $V_G''/V_G'$ ). In the case of the self-weight of the bridge, in accordance with the recommendations of the bulletin, the diagrams were cut off at the lower limiting values. The resulting adjusted partial

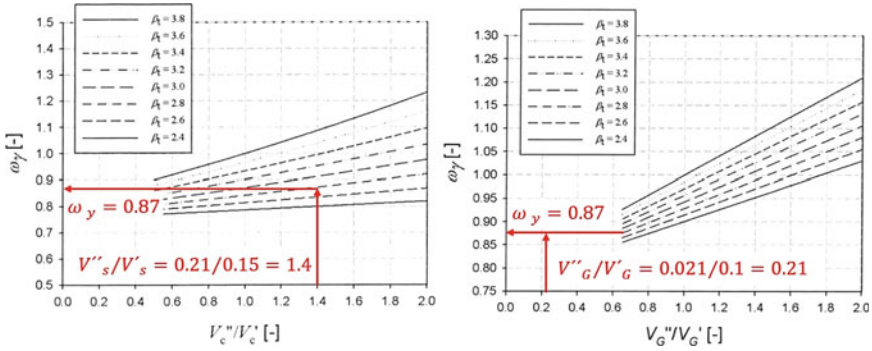


Fig. 2 Obtaining the adjustments factors for concrete and permanent actions for the APFM

safety factor for concrete is  $\gamma_M = 0.87 \times 1.5 = 1.30$  and for self-weight  $\gamma_G = 0.87 \times 1.35 = 1.18$ .

### 4 Case Study of 3-Span Continuous Reinforced Concrete Slab

The second case study is 3-span continuous reinforced concrete slab located on the Croatian state road E65 near Posedarje town (Fig. 3), built in 1961. Detailed original design plans are available for all structural elements and amount of built-in reinforcement is checked, but no in situ measurements are taken in this study. Bridge spans are  $9 + 15 + 9 = 33$  m. The slab is supported by concrete hinges on piers and the first (left) abutment, while a movable bearing is on the second abutment. Bridge

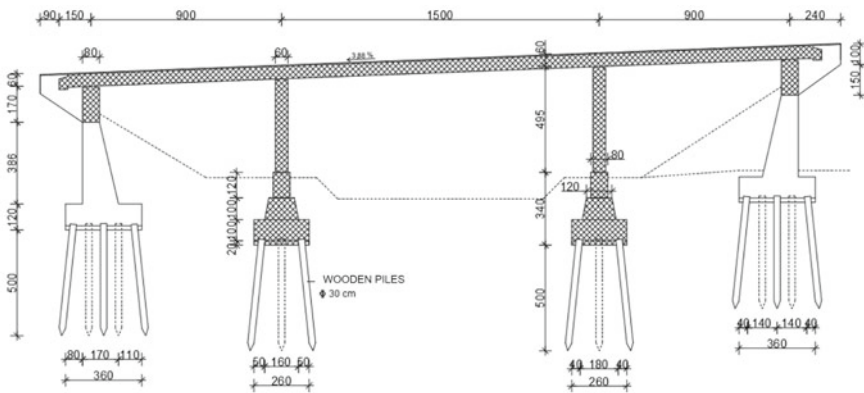


Fig. 3 Longitudinal section of the 3-span bridge selected as the second case study



slab is 0.60 m deep and 8.50 m wide. The bridge is designed according to the codes from 1940s. The designed concrete strength corresponds to C30/37 [7, 8].

In general, only visual inspections each six years are required for a bridge in good conditions without deficiencies (damage, cracks, corrosion, vibration). Additional destructive and non-destructive testing is needed in three cases: (i) for bridges of significant importance, large span or landmark bridges; (ii) deterioration or some deficiency is revealed by inspection, or (iii) after extreme events (e.g. earthquake, flood, etc.). This case study is focused on updating partial safety factors for the preliminary assessment based on no new material tests or traffic data.

The bridge is classified in Consequence Class CC3, the minimum target reliability is  $\beta_0 = 4.3 - 1.5 = 2.8$  and two reference periods ( $t_{\text{ref},1} = 40$  years and  $t_{\text{ref},2} = 15$  years; both equal to remaining lifetimes in two alternatives under investigation) are assumed. The material factors for concrete ( $\gamma_C$ ) and reinforcement ( $\gamma_S$ ) are determined as follows:

$$\gamma_C = \gamma_{Rd} \cdot \gamma_c = e^{\alpha_{\theta_{Rc}} \cdot \beta \cdot V_{\theta_{Rc}}} \cdot \gamma_c = e^{0.4 \cdot 0.8 \cdot 2.8 \cdot 0.14} \cdot 1.09 = 1.13 \cdot 1.09 = 1.24 \quad (3)$$

$$\gamma_S = \gamma_{Rd} \cdot \gamma_s = e^{\alpha_{\theta_{Rs}} \cdot \beta \cdot V_{\theta_{Rs}}} \cdot \gamma_s = e^{0.4 \cdot 0.8 \cdot 2.8 \cdot 0.06} \cdot 1.03 = 1.06 \cdot 1.03 = 1.08 \quad (4)$$

where  $\gamma_{Rd}$  is the value for model uncertainties (concrete or steel) calculated by considering a lognormal distribution [3], and  $\gamma_c$  and  $\gamma_s$  are estimated according to Fig. 4 (left), considering  $V_c = 0.15$  and  $V_s = 0.05$  – basic values in [3].

Permanent action factor  $\gamma_G$  is calculated for unfavourable action and favourable load effects:

$$\gamma_G = \gamma_{Ed,G} \cdot \gamma_g = e^{-\alpha_{\theta_E} \cdot \beta \cdot V_{\theta_E}} \cdot \gamma_g = e^{0.4 \cdot 0.7 \cdot 2.8 \cdot 0.10} \cdot 1.19 = 1.08 \cdot 1.19 = 1.29$$

for an unfavourable action (5)

$$\gamma_G = \gamma_{Ed,G} \cdot \gamma_g = e^{-\alpha_{\theta_{E, fav}} \cdot \beta \cdot V_{\theta_E}} \cdot \gamma_g = e^{-0.4 \cdot 0.8 \cdot 2.8 \cdot 0.10} \cdot 0.92 = 0.91 \cdot 0.92 = 0.84$$

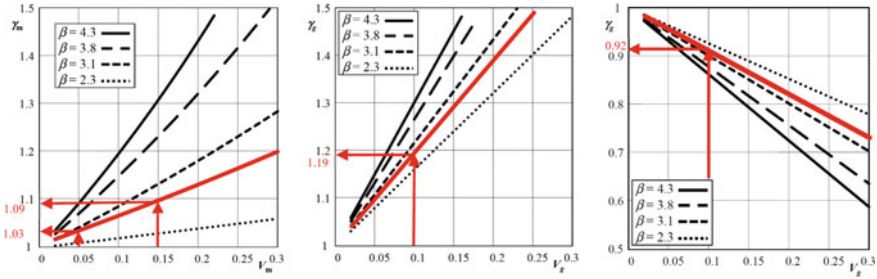
for a favourable action (6)

considering the recommended determination approach for  $\gamma_{Ed,G}$  and  $V_g = 0.1$  (Fig. 4, middle and right).

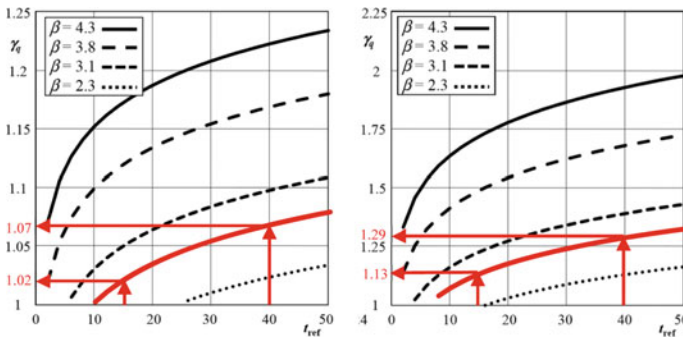
Partial safety factors for variable action,  $\gamma_Q$ , is given as:

$$\gamma_Q = \gamma_{Ed,Q} \cdot \gamma_q = e^{-\alpha_{\theta_E} \cdot \beta \cdot V_{\theta_E}} \gamma_q \quad (7)$$

where model uncertainty factor is  $\gamma_{Ed,Q} = 1.08$  for  $\beta_0 = 2.8$  as  $\gamma_{Ed,G}$  in Eq. 5 for the unfavourable permanent action effect. The factors  $\gamma_q$  are obtained from Fig. 5 considering  $\beta_0 = 2.8$  and  $t_{\text{ref},1} = 40$  years or  $t_{\text{ref},2} = 15$  years considering the recommended values for  $V_T$  and  $V_{vb}$  (related to annual maxima of traffic load effect and basic wind velocity, respectively).



**Fig. 4** Determination of  $\gamma_m$  ( $V_C = 0.15$ ;  $V_S = 0.05$ ;  $\alpha_R = 0.8$  and  $\beta = 2.8$ ) (left), the partial factor  $\gamma_g$  for unfavourable action ( $V_g = 0.1$ ;  $\alpha_E = -0.7$ ;  $\beta = 2.8$ ) (middle) and for favourable action ( $V_g = 0.1$ ;  $\alpha_{E,fav} = 0.32$ ;  $\beta = 2.8$ ) (right)



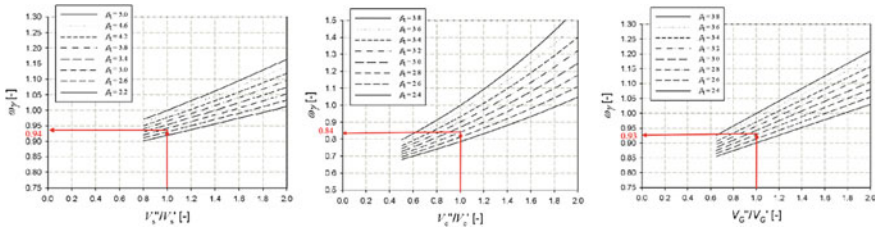
**Fig. 5** Determination of the partial factors for: the traffic load,  $\gamma_{q,t}$  ( $V_T = 0.075$ ) (left); the wind action,  $\gamma_{q,w}$  ( $V_{vb} = 0.12$ ) (right) for  $\beta = 2.8$ , and  $t_{ref,1} = 40$  years or  $t_{ref,2} = 15$  years

According to APFM, the partial safety factors for existing structures can be generally determined as:

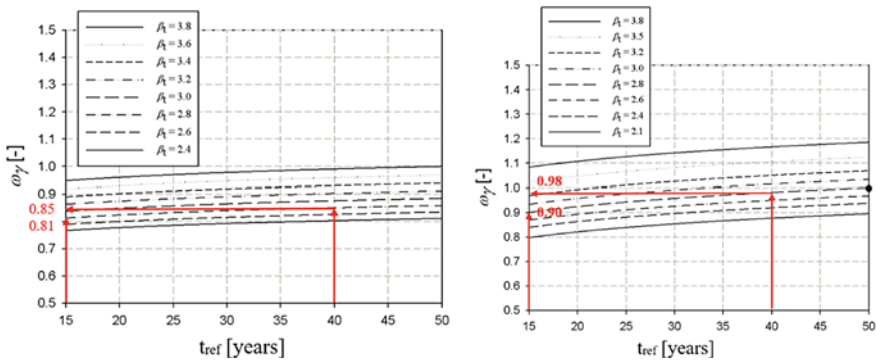
$$\gamma_X = \omega_\gamma \cdot \gamma_{X,new} \tag{8}$$

where  $\omega_\gamma$  is an adjustment factor and  $\gamma_{X,new}$  is the partial factor for new structures given in the Eurocodes. The values of ratio between the coefficient of variation for new structures and the actual value obtained on the structure are assumed in the case study. Adjustment factors are determined graphically (Figs. 6 and 7).

The obtained factors according to the DVM and APFM methods are summarised in Table 3. Partial factors are reduced compared to the corresponding values for new structures. Factors for unfavourable permanent actions obtained by DVM and APFM are almost equal, while the APFM gives more conservative values of partial factors for traffic load and wind action in comparison to the DVM due to a simplified conservative treatment of model uncertainties.



**Fig. 6** Determination of the adjustment factors for steel  $\omega_{y,s}$  (left), concrete  $\omega_{y,c}$  (middle) and unfavourable permanent actions  $\omega_{y,g}$  (right) according to the APFM ( $\alpha_R = 0.8$ ;  $\alpha_E = -0.7$ ;  $\beta = 2.8$ ;  $V_s''/V_s' = 1.0$ ;  $V_c''/V_c' = 1.0$ ;  $V_G''/V_G' = 1.0$ ;  $V_G' = 0.1$ )



**Fig. 7** Adjustment factors for variable actions: traffic loads  $\omega_{y,l}$  (left) and wind actions  $\omega_{y,sw}$  (right) according to the APFM – method A ( $\alpha_E = -0.7$ ;  $\beta = 2.8$ ;  $t_{ref,1} = 40$  years or  $t_{ref,2} = 15$  years;  $V_{Qclim,1}''/V_{Qclim,1}' = 1.0$ )

Analysis of the differences between  $\gamma_Q$ -factors of DVM and APFM (method A) reveals that the difference between model uncertainty factors  $\gamma_{Ed,Q}$  is negligible ( $\leq 0.5\%$ ) and the reasons for the difference needs to be found in the load factors  $\gamma_q$ .

DVM combines a time-variant component of the load (such as basic wind pressure) with time-invariant coefficients (e.g. shape, gust, roughness and other factors for wind pressure). The design value is then derived for given  $t_{ref}$  and  $\beta$  using recommended probabilistic models of the time-variant and time-invariant variables. The probabilistic models are simplified, but they are deemed to provide unbiased estimates, reflecting the existing knowledge. Consequently, use of the recommended models may not always lead to  $\gamma_Q = 1.5$  (or 1.35 for road traffic load) for a “basic case” with  $\beta = 3.8$  and  $t_{ref} = 50$  years.

As an example let us consider a wind action effect for which the largest difference in  $\gamma_Q$  between DVM and APFM is indicated in Table 3. For the basic case ( $\beta = 3.8$  and  $t_{ref} = 50$  years), DVM leads to  $\gamma_w = 1.75$  and  $\gamma_W = 1.11 \times 1.75 = 1.94$ . Changing  $\beta$  to 2.8 significantly decreases particularly  $\gamma_w$ . The partial factor becomes  $\gamma_W = 1.08 \times 1.32 = 1.43$  for  $\beta = 2.8$  and  $t_{ref} = 50$  years, i.e. decrease by about

**Table 3** Summarised results for modified partial factors according to the DVM and APFM methods ( $\beta = 2.8$ ;  $t_{ref,1} = 40$  years and  $t_{ref,2} = 15$  years)

Partial factors	DVM		Adjustment/partial factors	APFM			
	$t_{ref}$ (years)			Input assumptions	$t_{ref}$ (years)		Input assumptions
	40	15			40	15	
$\gamma_c$	1.09		$V_{m,C} = 0.15$	$\omega_{\gamma,c}$	0.84		$V''_C/V'_C = 1$
$\gamma_s$	1.03		$V_{m,S} = 0.05$	$\omega_{\gamma,s}$	0.94		$V''_S/V'_S = 1$
$\gamma_C$	1.24		$V_{\theta R,c} = 0.14$	$\gamma_C$	1.26		
$\gamma_S$	1.08		$V_{\theta R,s} = 0.06$	$\gamma_S$	1.08		
$\gamma_g$	1.19		$\alpha_E = -0.7$	$\omega_{\gamma,G}$	0.93		$V''_S = V'_S = 0.1$
$\gamma_{g,fav}$	0.92		$\alpha_{E,fav} = 0.32$				
$\gamma_{q,w}$	1.29	1.13	$V_{vb} = 0.12$ (annual max.)	$\omega_{\gamma,W}$	0.98	0.90	$V'_{Q_c \lim 1} = V''_{Q_c \lim 1}$
$\gamma_{q,t}$	1.07	1.02	$V_T = 0.075$ (annual max.)	$\omega_{\gamma,t}$	0.85	0.81	$\beta' = 3.8$
$\gamma_G$	1.29		$\gamma_{Ed,G} = 1.07$	$\gamma_G$	1.26		
$\gamma_{G,fav}$	0.84		$\gamma_{Ed,G} = 1.00$				
$\gamma_{Q,w}$	1.39	1.22	$\gamma_{ED,Q} = 1.12$	$\gamma_{Q,W}$	1.47	1.35	
$\gamma_{Q,t}$	1.16	1.10		$\gamma_{Q,t}$	1.28	1.22	

25% in comparison to the basic case. Decreasing  $t_{ref}$  to 40 years or 15 years yields additional reductions by 3% or 15%, respectively.

APFM makes no explicit distinction between the time-invariant and time-variant components of a variable load effect. For imposed and traffic loads, the method is tuned to obtain  $\gamma_Q = 1.5$  or 1.35, respectively, for the basic case. For snow and wind loads, it is recognised that  $\gamma_Q = 1.5$  is mostly insufficient to reach  $\beta = 3.8$ . The recommendations in APFM have thus been established in a way that  $\gamma_S = \gamma_W = 1.5$  is reached for  $\beta = 2.8$ , representing implicitly accepted risk in Eurocodes. The different assumptions in APFM lead to different sensitivity of  $\gamma_Q$  to changes in  $\beta$  and  $t_{ref}$ . For a wind load effect, APFM indicates  $\gamma_W = 1.77$  for the basic case ( $\omega_{\gamma,sw} = 1.18$  when  $\beta = 3.8$  and  $t_{ref} = 50$  years). Decreasing  $\beta$  to 2.8 gives a smaller reduction (15%) than in the case of DVM. Decreasing  $t_{ref}$  to 40 years or 15 years yields again smaller reductions (by 2% or 10%, respectively) than for DVM.

## 5 Discussion and Conclusions

The performance assessment of existing structures can take place at various times during their service life and for various reasons: change in the use of the structure, evolution of environmental variables and their uncertainties as well as material properties, regulatory developments, doubts about serviceability or safety levels, damage observed in situ, etc. Considering the difficulties in reliability assessments of existing structures, the assessment process needs to be based on a rational approach whose degree of sophistication and related assumptions depend structure-specific conditions.

The two methods DVM and APFM provided in [3] are based on the same principles, introduced in 1980s as the design value format. The methods aim to cover most common cases and ensure the compatibility with Eurocodes. In this sense, they provide an interesting framework to specify partial factors for practitioners who use a semi-probabilistic format when assessing existing structures. The two case studies presented in this paper illustrate how partial factors would be specified in each case. The DVM and the APFM are applied and lead to slightly different results, particularly due to simplified conservative treatment of model uncertainties and variable load effects in the APFM. The differences between the methods become more significant when the target reliability and reference period substantially differ from the reference values,  $\beta = 3.8$  and  $t_{\text{ref}} = 50$  years. Further studies should investigate how the choice between these methods would change the result of the assessment process (after determining the load bearing capacity) and where the use of fixed partial factors with updated characteristic values (the procedure often adopted in standards) is sufficient.

A major simplification of the two methods is that statistical uncertainty associated with the new measurements on the existing structure is considered only in the estimate of characteristic values of basic variables. Particularly in the case of absent prior information, the effect of this simplification might be large, especially for small sample sizes that are typical for reassessment situations. Here, the application of the methods could lead to non-conservative estimates of structural reliability. Finally, as simplified methods, DVM and APFM are based on some assumptions on standardised sensitivity factors or types of probabilistic distributions. For a specific limit state, the real sensitivity factors might deviate to a certain extent (in particular for complex limit states with several variables). Further analyses should consider the effect of the reference period on the target reliability index—the use of the two methods should be explored for reliability verifications based on an annual reference period (as introduced e.g. in ISO 2394). A full probabilistic analysis should provide reference reliability levels to check the consistency of the simplified approaches.

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