Decisions under uncertainty: *fib* Bulletin 80 versus prCEN/TS

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ABSTRACT

In the past few years, various studies have been conducted to develop approaches for the assessment of existing structures, compatible with the Eurocodes, which include the same verification format as normally applied for designing new structures. The transformation of an industrial building from the 1940's into a cultural centre requires safety verification of the reinforced concrete structure. The application of different partial factor approaches according to, respectively, *fib* Bulletin 80 and CEN Technical Specification on the Assessment of Existing Structures, is illustrated by means of the verification of one of the main beams of this building. The results obtained are discussed and compared to the findings of a full probabilistic analysis.

RESUMEN

En los últimos años, se han llevado a cabo varios estudios para el desarrollo de enfoques para la evaluación de estructuras existentes, compatibles con los Eurocódigos, que incluyan el mismo formato de verificación que se emplea para el dimensionamiento de estructuras nuevas. La transformación de una edificación industrial de los años 1940 en un centro cultural requiere la verificación de la seguridad de su estructura de hormigón armado. La aplicación de diferentes enfoques basados en el método de los coeficientes parciales, de acuerdo con el *fib* Bulletin 80 y la Especificación Técnica del CEN sobre Evaluación de Estructuras Existentes, respectivamente, se ilustra mediante la verificación de una de las vigas principales de la edificación. Los resultados obtenidos son analizados y comparados con los de la verificación mediante un método probabilista.

KEYWORDS: existing structures, reinforced concrete, assessment, probabilistic methods, partial factors.

PALABRAS CLAVE: estructuras existentes, hormigón, evaluación, métodos probabilistas, coeficientes parciales.

1. Introduction

The main difference between assessing the performance of existing and designing new structures is that many characteristics whose values are merely anticipated in the latter can be measured in the former. The accuracy of the load and resistance models needed for the assessment can always be enhanced by collecting more data about the structure studied. Therefore, assessment is conducted by stages [1,2], raising the quality of the information available from stage to stage. The most accurate way to incorporate site data in structural safety verifications would be to conduct a probabilistic analysis. This is a time-consuming process, however, calling for a working knowledge of probabilistic methods that may not be suited to everyday use by practising engineers. Therefore, simplified methods for assessing the reliability of existing structures are normally applied. In such methods, based on the same partial factor formulation as adopted in structural design codes, the representative values for the variables and the partial factors can be modified on the basis of updated information.

Recently, various studies [2,3,4,5] have been conducted to develop assessment approaches, compatible with the Eurocodes, that include the same verification format as normally applied for designing new structures and that allow for taking into account the influence of updated information on the characteristic values of the variables and the respective partial factors. An overview of three of these studies is presented in section 2. Practical tools for reliability assessments of existing reinforced concrete (RC) structures, using the partial factor method (PFM) have been developed in [3,4]. These tools, established for sound RC structures, are discussed in section 2.2.

The *fib* Bulletin 80 [5], provides a basis for rational decision making with respect to safety-related issues associated with existing structures, as well as an extensive body of background information. Two simplified procedures for applying partial factor methods to the assessment of existing RC structures are proposed in [5]: the design value method (DVM) and the adjusted partial factor method (APFM), briefly summarized in section 2.3.

For the past few years, the CEN/TC 250 Working Group 2 has been engaged in the development of a Technical Specification (TS) on the Assessment of Existing Structures [2]. The TS establishes as initial method for the verification of the structural safety of an existing structure the PFM. The partial factor format included in [2] is based on the assessment values of effects of actions and resistance, which are equivalent to the design values according to Eurocode [6]. The values of the design parameters are substituted by the corresponding values for assessment, taking into account the available information on the actual properties of the structure. Section 2.4 provides an overview of the approach included in [2].

The main purpose of this paper is the presentation of a practical application of the aforementioned methods, by means of the reliability verification of an existing beam belonging to an industrial reinforced concrete building. The example has been solved by the authors applying both partial factor approaches (DVM, APFM) proposed in [5]. Likewise, as a round robin exercise on the application of TS [2], experts of National Standartization Bodies (NSB) of eight CEN member states have solved the example. The authors were responsible for the definition of the task including the input data. After the national experts had delivered their results, a sample solution has been provided by applying the operational rules developed in [3, 4] which are compatible with the basic principles specified in [2]. The problem statement and the input data for the case study are presented in section 3, while in section 4 the verifications, carried out by applying the aforementioned approaches, are described and the outccomes discussed. In section 5, a full probabilistic analysis is conducted for the beam cross-section considered and the results are compared to those obtained by applying the partial factor approaches. Conclusions from the application of the different methods are summarized in section 6.

2. PFM for reliability verification

2.1 Design

In most codes, the design values x_{id} for variables X_i are not considered directly [7]. Rather, random variables with representative or characteristic values $x_{k,i}$ are entered in the models used for structural reliability verifications and the respective design values are obtained by applying a set of partial factors. In approximate analytical methods, such as First or Second Order Reliability Methods (FORM/SORM) [7], partial factors depend on:

- the partial factor format adopted;
- the chosen reference period;
- the representative or characteristic values of the variables, x_{ki};
- the parameters (e. g. mean value, μ_{Xi}, standard deviation, σ_{Xi}) and the type of distribution assumed for the variables, X_i;
- the target reliability index, β_t, for the limit state and the design situation at issue;
- the α_i factors describing the sensitivity to variations in X_i with regard to attaining the considered limit state.

2.2 Assessment

2.2.1. Tools developed

When assessing the reliability of existing structures, updating information on a variable by gathering site-specific data to reduce the associated uncertainties affects both the characteristic value of the variable considered and the respective partial factor. For this reason, and in view of all the foregoing, the following issues have been addressed in a prior study to develop tools for the assessment of the reliability of existing sound structures, based on the partial factor method [4]:

- identification of representative failure modes and the respective limit state functions (LSF);
- definition of an appropriate reference period;
- definition of target reliability levels (item 2.2.2);
- adoption of a partial factor format for the assessment of structural reliability (item 2.2.3);
- for relevant variables X_i; according to the LSF considered, inference of default probabilistic models (e.g., parameters μ_{Xi}, σ_{Xi} and type of distribution) which may be used as uncertain prior information [3];
- methods for updating of probabilistic distributions of basic variables (including updating of characteristic values, $x_{ki,\mathrm{a}}$, and partial factors, $\gamma_{Xi,\mathrm{a}}$) by combining site-specific data with prior information, represented by the aforementioned default probabilistic models, for instance.

2.2.2. Target reliability

Structural acceptability should be determined by comparing the outcome of the assessment of an existing structure to established safety requirements as regards acceptable risks. The easiest and perhaps most logical approach is to establish the acceptable risks, and the corresponding reliability indices β_t, as the inherent risks associated with general state-of-the-art practice set out in existing structural standards [8,9,10], which are regarded to be acceptable by definition. The assessment tools developed in [3,4] are based on that criterion and have been applied in the present study for the assessment of the existing RC beams (sections 3 and 4). The partial factor format adopted for the assessment is described below.

2.2.3. Partial factor format for assessment

A cross-section, structural member or connection, is deemed to be structurally safe for its remaining service life if the following criterion holds in all significant hazard scenarios:

$$E_\mathrm{a} \leq R_\mathrm{a} \tag{1}$$

For persistent and transient situations, the assessment value for the action effects, E_a, can be determined as follows:

$$E_\mathrm{a} = \gamma_{S\mathrm{a}} \cdot E\left\{ \sum_{j\geq 1} \gamma_{g\mathrm{a},j} \cdot G_{k\mathrm{a},j};\ \gamma_{p\mathrm{a}} \cdot P_{k\mathrm{a}};\ \gamma_{q\mathrm{a},1} \cdot Q_{k\mathrm{a},1};\ \sum_{i>1} \gamma_{q\mathrm{a},i} \cdot \psi_{0,i} \cdot Q_{k\mathrm{a},i} \right\} \tag{2}$$

where: $G_{ka,j}$, P_{ka}, $Q_{ka,1}$, $Q_{ka,i}$, respectively represent the updated characteristic values for the permanent, prestressing, leading and accompanying variable actions; and γ_{ga}, γ_{pa}, γ_{qa}, represent the updated partial factors through which possible unfavourable deviations in the action values from the updated characteristic values are accounted for in the model. With updated partial factor γ_{Sa}, allowance is made for the uncertainties associated with the action effect models and the simplified representation of actions. Since model uncertainties vary depending on the action effects considered, in lieu of applying a single partial factor, γ_{Sa}, as many factors are introduced as different action effects are to be calculated and are denoted as $\gamma_{Sa,M}$, $\gamma_{Sa,V}$, $\gamma_{Sa,N}$ for, respectively, bending moments, shear forces and axial forces. Consequently, the partial factor format adopted to establish assessment values for action effects differs from standard code rules for structural design such as laid down in the Eurocodes [6]. However, this format is more accurate for assessment purposes.

The assessment value for ultimate resistance, R_a, is determined by using updated characteristic values for material or product properties, $X_{ka,i}$, and updated assessment values for geometrical data, a_a, equivalent to those defined in the Eurocodes [6]:

$$R_{\mathrm{a}} = \frac{1}{\gamma_{R\mathrm{a}}} \cdot R\left\{\eta_{\mathrm{a}i} \cdot \frac{X_{k\mathrm{a},i}}{\gamma_{m\mathrm{a},i}}; a_{\mathrm{a}}\right\} \quad i \geq 1 \qquad (3)$$

The partial factor for resistance is divided into the updated partial factors for material or product properties, $\gamma_{ma,i}$, and an updated partial factor associated with resistance model uncertainties, γ_{Ra}. The latter also vary depending on the failure mechanism considered, which is why different γ_{Ra} factors are applied. In RC structures, for instance, different updated partial factors associated with resistance model uncertainties are defined for bending moments, $\gamma_{Ra,M}$, tensile forces in the web, $\gamma_{Ra,Vs}$, diagonal compression forces in the web, $\gamma_{Ra,Vc}$, and axial compression forces, $\gamma_{Ra,N}$. The criteria applicable to action effects also apply to the respective resistances: the partial factor format adopted to calculate the assessment value for ultimate resistance is a more accurate approach to assessment than the usual code formats used for design purposes [6]. Default values are used for most conversion factors for the effects of volume, scale, moisture or temperature on the material or product properties. Such factors may be updated ($\eta_{a,i}$), however, if the necessary information is available.

2.3 Reliability verificaction by PFM in fib Bulletin 80

2.3.1. General

Both partial factor formats provided in [5], the DVM and the APFM, enable the incorporation of specific reliability-related aspects for existing structures. The DVM proposes a fundamental basis for evaluating partial factors, whereas the APFM provides adjustment factors to be applied to the partial factors for new structures in [6]. The two alternative methods are briefly explained in the following.

2.3.2. Design Value Method

The DVM consists in the application of the partial factor method as defined in the Eurocode for new structures [6], but allowing to derive the partial factors γ_X from the actual distribution of the variable X under consideration (based on prior information, results of tests or the combination of both). Furthermore, the partial factors can be established taking into account case-specific target reliability levels and the remaining working life for existing structures.

2.3.3. Adjusted Partial Factor Method

According to [5] the basic philosophy of the APFM consists in calculating adjusted partial factors γ_X for the variables X related with an existing structure, considering alternative values for, respectively, the reference period t_{ref}, the target reliability index β_t and the coefficient of variation V_X of the variable under consideration. For a given variable X, this adjusted partial factor is obtained by multiplying the partial factor $\gamma_{X,new}$ provided in the Eurocodes for new structures [6] by an adjustment factor ω_γ:

$$\gamma_X = \omega_\gamma(t_{ref}, \beta_t, V_X) \cdot \gamma_{X,new} \tag{4}$$

2.4 Reliability verification by PFM in prCEN-TS

According to [2], the assessment of an existing structure should initially be carried out using PFM. After this method has been utilized, the assessment value method, probabilistic methods and the risk assessment method can be used subsequently for:

- overcoming the conservatism of partial factor methods;
- cases of structural failures with serious consequences;
- cases of insufficient robustness;
- evaluating the efficiency of monitoring and maintenance strategies;
- making fundamental decisions concerning a whole group of structures.

When using the PFM, the TS [2] indicates that the inequality given by Formula (1) shall by verified. The assessment value of the effect of actions E_a should be determined in the same way as for the design value E_d according to [6], but substituting the values of design parameters (including basic variables, partial factors, and factors ψ), by the corresponding values for assessment, if available. Likewise, the assessment value of resistance R_a should be determined in the same way as for the design value R_d in [6], but substituting, as far as possible, the values of design parameters (including basic variables, partial factors for materials, and conversion factors), by the corresponding values for assessment. The partial factor format described in section 2.2.3 is compatible with the general principles proposed in [2] for determining E_a and R_a, respectively.

3. Problem statement

The transformation of an industrial building from the 1940's into a cultural centre requires the reliability verification of the existing reinforced concrete structure for the future use conditions. The building is located in the northwest of Spain and is exposed to a severe marine environment. The existing structure has been abandoned for 20 years and shows significant corrosion damage, in particular with regard to the secondary structural elements as well as the non-structural elements. Figure 1 shows the condition of the existing building before the transformation.

Figure 1. Condition of the industrial building before transformation (Courtesy of Diaz & Diaz Architects S.L.).

Due to architectural requirements, non-structural and secondary structural elements were demolished. The aim was to integrate most of the existing main structural elements, repaired if deteriorated, into the load bearing system of the transformed building. Reliability verification was therefore required for these existing structural members, taking into account the properties of the transformed building and its future use. The planned transformation of the building is represented in Figure 2.

In the present application example, the structural safety of the existing floor beams should be verified. For sake of simplicity, the only limit state considered in this contribution is that associated with bending failure at mid-span.

The beams are simply supported, with 6 m span length, and their centre to centre distance is 5 m. After the demolition of the existing slabs, new lightweight floors will be installed, supported by the existing main beams. No shear connection will be provided between these beams and the new floor.

After the transformation of the building, the characteristic value of the permanent load due to the self-weight of the new floor, partition walls, etc. will reach g_k = 2 kN/m^2. The floor system, including the beams to be assessed, belongs to an area with fixed seats. The remaining working life of the transformed structure is 50 years. A staged assessment procedure based on the PFM should be carried out. The existing beams should be assessed taking into account their actual conditions after repair, i.e. no deterioration needs to be considered, and the intended use of the corresponding building area.

Figure 2. Final condition of the building (Rendering courtesy of Diaz & Diaz Architects S.L.).

4. Verification based on PFM

4.1. First stage

4.1.1. Input data

For the first stage, site information is obtained by investigating a limited number of the demolished secondary members of the building, with material characteristics, which are considered to be representative for the assessment of the main structural elements. Site specific characteristics of concrete and reinforcing steel are established based on the information gathered. The obtained sample parameters (mean value, f_{cm}, standard deviation, σ_{fc}, and coefficient of variation V_{fc}) of the concrete compressive strength, f_c, are summarized in Table 1. Table 2 contains the mean, f_{ym}, and the coefficient of variation, V_{fy}, of the reinforcing steel yield strength, f_y. For both, f_c and f_y, the coefficients of variation associated with structures designed and built according to former code rules [11] are available. These coefficients of variation, are V_{fc} = 0.25 and V_{fy} = 0.10, respectively, and can be adopted as prior information.

Table 1. First stage sample parameters for concrete compressive strength.

Source	Number of tests	f_{cm} [N/mm²]	σ_{fc} [N/mm²]	V_{fc}
Secondary elements	6	27.3	9.7	0.36

Table 2. First stage sample parameters for reinforcing steel yield strength.

Source	Number of tests	Diameter [mm]	f_{ym} [N/mm²]	V_{fy}
Secondary elements	3	20	219	0.05

Geometry of the demolished secondary structural members is also established. As a result, it can be assumed that cross-sectional dimensions correspond to nominal values, with deviations which are of the same order of magnitude as modern construction tolerances. Cross-section geometry of the studied main beam is defined in Figure 3.

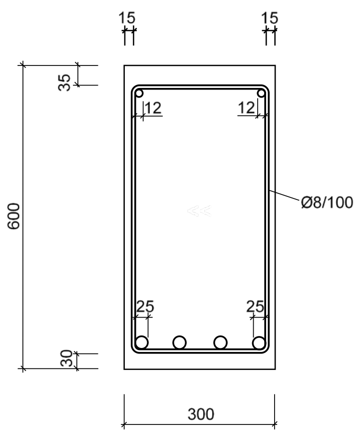

Figure 3. Beam cross-section.

Table 3 shows the parameters probabilistic models for the different load and resistance variables corresponding to the state of information at the first stage of the assessment. The parameters for the concrete compressive strength, f_c, and yield strength of reinforcement, f_y, have been updated according to the procedure included in [2, 7], by combining the information from the former code [11] with the test results (Tables 1 and 2). For the other basic variables involved in the assessment, default probabilistic models were adopted in accordance with the models in [3]. For solving the example according to both partial factor approaches provided by *fib* Bulletin 80 [5], DVM and APFM, the parameters for the different variables have been established following the recommendations given in the same document. The updated models for f_c and f_y, listed in Table 3, were applied in the case of the DVM. Meanwhile, for the APFM, values for the coefficients of variation related to the existing structure, V_c'' and V_s'', for concrete compressive strength and reinforcing steel yield strength, respectively, as well as for their characteristic values f_{ck} and f_{yk}, have been obtained directly from the test results (Tables 1 and 2). The values for new structures, suggested in [5], for the coefficients of variation, $V_c' = 0.15$ and $V_s' = 0.05$, respectively, have been adopted. In this way, the limiting conditions for the application of the method, regarding the coefficients of variation, $(V_s''/ V_s')_{\min} = 0.8$ and $(V_c''/ V_c')_{\min} = 0.5$, are fulfilled. The probabilistic models for the action variables have been assumed in line with the indications of [5] for the DVM and APFM. The characteristic values and partial factors applied by the experts of the eight CEN member states, that have solved the example, were adopted taking into account the site-specific data, and established according to relevant Eurocodes and Nationally Determined Parameters (NDPs).

The assumed target reliablity indices, β_t, and characteristic values of the imposed load, q_k, are indicated in Table 5. The values for β_t have been adopted following the indications of [2], Eurocode [6], National Standards of the CEN member states, as well as other considerations (e. g. based on risk analysis). Regarding q_k, the recommendations of the Eurocode [12] together with NDPs have been considered.

4.1.2. Results

The partial factors resulting from the application of the procedures included in [4] and [5], together with the probabilistic models previously discussed (Table 3), are indicated in Table 4.

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Table 3. First stage probabilistic models for basic variables adopted for the assessment according to [4].

Variable	Distribution	X	X_k	V_X	μ_X/X_k
Self-weight of concrete, kN/m³	N	g_c	25	0.04	1.00
Other permanent loads, kN/m²	N	g_p	2.0	0.10	1.00
Variable load (imposed), kN/m²	Gum	q_{imp}	4.0	0.26	0.68
Model uncertainty for load effect (bending moments)	LN	$\xi_{E,M}$	1.0	0.10	1.00
Concrete compressive strength, N/mm² *	LN	f_c	16.5	0.25	1.61
Yield strength of reinforcement, N/mm² *	LN	f_y	181	0.10	1.21
Effective depth of mid span cross-section, mm	N	d	549.5	0.04	1.0
Cross-section width, mm	N	b	300	0.03	1.0
Cross sectional area of reinforcement, mm²	N	A_s	1963	0.02	1.0
Uncertainty of bending resistance model	LN	$\xi_{R,M}$	1.0	0.064	1.0

* Updated according to [2, 7]

From Table 5 can be observed that structural safety is neither verified for any of these procedures, nor do the results obtained by all of the NSBs (based on TS [2]), satisfy the safety requirements. Therefore, a second stage with updated information would be necessary. It can be also observed, from Table 5, that the results obtained for the assesment values of the effects of actions (bending moments), $M_{E\mathrm{a},1}$, show a relatively slight variation, while the assessment values of resistance, $M_{R\mathrm{a},1}$, show more significant variations. The small variation in the values of $M_{E\mathrm{a},1}$ can be explained taking into account that both the characteristic values of actions and their probabilistic models, that were used for the deduction of partial factors, have not been derived from site-specific data but were adopted according to the recommendations of [2,3,5,6,12] or NDPs of CEN member states.

Table 4. Partial factors for the first stage of the assessment.

Symbol	DVM [5]	APFM [5]		[4] procedure
	Partial factor	Adjustment factor	Partial factor	Partial factor
$\gamma_{gc,1}$	1.13	–	–	–
$\gamma_{gp,1}$	1.27	–	–	–
$\gamma_{q,1}$	1.37	–	–	–
$\gamma_{Sa,M,1}$	1.11	–	–	–
$\omega_{\gamma,G,1}$	–	0.96	–	–
$\omega_{\gamma,Q,1}$	–	0.89	–	–
$\gamma_{Gc,1}$	1.25	–	–	–
$\gamma_{Gp,1}$	1.41	–	–	–
$\gamma_{G,1}$	–	–	1.30	1.35
$\gamma_{Q,1}$	1.52	–	1.34	1.5
$\gamma_{s,1}$	1.15	–	–	1.13
$\gamma_{Ra,s,1}$	1.02	–	–	1.08
$\gamma_{c,1}$	1.42	–	–	1.0
$\gamma_{Ra,c,1}$	1.1	–	–	1.08
$\omega_{\gamma,S,1}$	–	0.97	–	
$\omega_{\gamma,C,1}$	–	1.23	–	
$\gamma_{S,1}$	1.17	–	1.12	1.22
$\gamma_{C,1}$	1.56	–	1.84	1.08

Regarding $M_{R\mathrm{a},1}$, large differences appear between the results obtained by the different CEN member states based on TS [2]; whereas the values resulting from the application of the two approaches proposed by [5] and the tools developed in [3,4] are quite similar. The large differences between the results for $M_{R\mathrm{a},1}$ obtained by the experts of CEN member states are mainly due to the different assumptions for the prior information, the procedures that have been used for updating the models for f_y and f_c, as well as the characteristic values adopted for these material properties. Particularly, in the case of NSB3 a comparatively small value for $M_{R\mathrm{a},1}$ has been obtained. This is due to the fact that the characteristic values for f_c and f_y have been determined from the sample parameters (Tables 1 and 2), according to [7], without taking into account the prior information, available from the former code [11]. Moreover, in this case the corresponding partial factors have not been updated and the values for new structures, according to [13], have been applied instead.

Table 5. First stage target reliability indices, characteristic values for imposed loads and assessment values for bending moment and resistance.

Approaches	β_t	q_k*	$M_{E\mathrm{a},1}$**	$M_{R\mathrm{a},1}$**	$M_{E\mathrm{a},1} \leq M_{R\mathrm{a},1}$
fib Bullettin 80 [5]					
DVM	3.8	4.0	225.6	152.3	No
APFM	3.8	4.0	205.4	151.3	No
NSB of CEN Member states [2]					
NSB1	3.8	5.0	230.0	155.0	No
NSB2	3.3	5.0	256.0	159.0	No
NSB3	3.5	4.0	223.2	131.0	No
NSB4	3.8	4.0	223.0	156.0	No
NSB5	3.8	4.0	217.0	183.0	No
NSB6	3.8	4.0	212.0	197.0	No
NSB7	3.3	5.0	229.5	191.9	No
NSB8	4.0	4.0	214.0	179.0	No
[4] proced.	3.8	4.0	223.1	150.7	No

* [kN/m²] ** [kNm]

4.2. *Second stage*

4.2.1. *Additional information*

Since in the first stage structural safety was not verified by applying any of the partial factor approaches considered, a second assessment stage is performed. For this purpose, additional information about the material properties was obtained directly from the investigated main beams. Tensile tests were conducted on four specimens. However, all of them were fabricated from samples extracted from one 25 mm diameter reinforcing bar, in order to minimize destructive material testing. The following values are measured for the reinforcing steel yield strength: f_y = {312; 301; 308; 306} N/mm^2. The sample parameters obtained from the tensile test results are shown in Table 6. Likewise, tests were conducted on four cores drilled in the main beams to determine the concrete compressive strength. The results are summarized in Table 7.

Table 6. Sample parameters for reinforcing steel yield strength.

Source	Number of tests	f_{ym} [N/mm²]	V_{fy}	f_{yk} [N/mm²]
Main beams	4	307	0.01	299

Table 7. Sample parameters for concrete compressive strength.

Source	Number of tests	f_{cm} [N/mm²]	V_{fc}	f_{ck} [N/mm²]
Main beams	4	25.9	0.09	20.1

Using the additional information, the parameters for f_c and f_y have been updated by combination, following the procedure included in [2, 7], with the results from the first stage (Table 3). The updated probabilistic models for these two variables are indicated in Table 8. They were used for the reliability verification applying the procedure included in [4] and the DVM [5]. For the APFM, values for the coefficient of variation V_c'', as well as for the characteristic values f_{ck} and f_{yk}, were obtained directly from test results (Tables 6 and 7). As in the first stage, the prior values, suggested in [5], for the coefficients of variation related to new structures, V_c' and V_s', respectively, were adopted. However, as established in [5], V_s'' = 0.04 was assumed, in order to fulfill the condition (V_s''/ V_s')$_{\min}$ = 0.8, in such a way that the uncertainty of model resistance could not become the dominant variable. For all other variables, the same probabilistic models as in the first stage were used when applying the procedure developed in [4] and the two approaches proposed in *fib* bulletin 80 [5], respectively.

Table 8. Second stage updated probabilistic models.

Variable	Distribution	X_k	V_X	μ_X/X_k
f_y [N/mm²]	LN	260.4	0.08	1.18
f_c [N/mm²]	LN	16.8	0.21	1.57

4.2.2. *Results*

The updated partial factors for the resistance variables used in the second stage of the assessment procedures according to [4] and [5] are indicated in Table 9. They were derived by applying the models described in the previous section. Since the probabilistic models for the actions have not been updated, the same partial factors as in the first stage were used (Table 4).

Table 9. Partial factors for resistance variables used in the second stage of the assessment

Symbol	DVM [5]	APFM [5]		[4] procedure
	Partial factor	Adjustment factor	Partial factor	Partial factor
$\gamma_{s,2}$	1.12	–	–	1.09
$\gamma_{Ra,s,2}$	1.02	–	–	1.08
$\gamma_{c,2}$	1.34	–	–	1.0
$\gamma_{Ra,c,2}$	1.1	–	–	1.08
$\omega_{y,S,2}$	–	0.96	–	
$\omega_{y,C,2}$	–	0.84	–	
$\gamma_{S,2}$	1.14	–	1.10	1.18
$\gamma_{C,2}$	1.47	–	1.26	1.08

The outcomes for the assessment values of the effect of actions and resistance are shown in Table 10. As can be seen, the beam reliability is verified by the experts of five of the CEN member states and also by applying the APFM [5]. On the contrary, reliability can neither be verified based on the DVM [5], nor on the application of the tools and procedure proposed in [3,4]. The results obtained by applying both procedures are indeed very similar. Consequently, for this application example, the results obtained with the DVM and the APFM do not agree with indications in [5], according to which the latter is often more conservative than the former. The reason for this is that when applying the APFM, the coefficients of variation and characteristic values for material properties were obtained directly from test results, conducted on specimens fabricated from samples extracted from the main beams, which showed a significantly small dispersion and led to a higher assessment value for bending resistance, $M_{Ra,2}$. For the other cases, although the beam reliability is not verified, the assessment value for bending moment $M_{Ea,2}$ is only about 2 % greater than the assessment value of resistance. The significant differences between the results for $M_{Ra,2}$ obtained by the experts of CEN members states are mainly due to the same reasons as previously outlined for the first stage.

From Table 10 can be observed that the assessment value of the bending resistance obtained by the NSB4 is significantly smaller than the other results. It has slightly rised from 156.0 in the first stage to 163.0 kNm in the second. This is explained by the fact that the expert has not taken into account the data for the reinforcing steel yield strength, obtained from the tensile tests and available for the second stage of the assessment, by considering that they are not representative. However, since the material properties have been calculated based on measurements, the expert considers to be justified the use of reduced values of partial factors for concrete and reinforcement according to [13].

Table 10. Second stage assessment values for bending moments and resistance.

Approaches		$M_{Ea,2}$*	$M_{Ra,2}$*	$M_{Ea,2} \leq M_{Ra,2}$
fib Bullettin 80 [5]	DVM	225.2	217.0	No
	APFM	205.4	265.3	Yes
NSB of CEN Member states [2]	NSB1.	230.0	216.5	No
	NSB2	Not carried out		
	NSB3	223.2	265.0	Yes
	NSB4	223.0	163.0	No
	NSB5	217.0	250.0	Yes
	NSB6	212.0	230.0	Yes
	NSB7	229.5	288.7	Yes
	NSB8	207.0	214.0	Yes
[4] procedure		223.1	218.4	No

* [kNm]

5. Probabilistic analysis

The structural reliability of the existing reinforced concrete beam cross-section (Figure 3) exposed to the permanent and variable loads specified in section 3 has been analysed by using the FORM method. When using probabilistic methods, the reliability of a structure shall be verified in terms of the failure probability according to the following condition [2,7]:

$$P_f = P\{g(x_i) < 0\} < P_{ft} \tag{5}$$

where $g\ (x_i\ )$ is the limit state function of the basic variables x_i and P_{ft} is the target probability of failure for a given reference period, here t_{ref} = 50 years. In the example of the reinforced concrete beam, the limit state function for the bending resistance in the mid-span cross-section is given as:

$$g(x_i) = \xi_{R,M}\left[f_y \cdot A_s \cdot d - 0.5 \cdot \frac{(f_y \cdot A_s)^2}{b \cdot \eta \cdot f_c}\right] - \xi_{E,M}\left[M_{gc} + M_{gp} + M_q\right] \tag{6}$$

where M_{gc}, M_{gp} and M_q are the bending moments due to the self-weight of the beam, the other permanent loads and the variable (imposed) load, respectively. Other variables are explained in previous sections. In a first analysis, the probabilistic models given in Table 3 are used. The resulting design values and sensitivity factors for the basic variables are indicated in Table 11. A probability of failure of $p_f = 2.86 \times 10^{-3}$ has been obtained, which corresponds to a reliablity index $\beta = 2.76$. Considering the assumed target reliability index of $\beta_t = 3.8$ (Table 5), the reliability of the beam is insufficient. This confirms the results obtained in first stage of the safety verification by applying the different partial factor approaches (Table 5).

Table 11. Assessment values and sensitivity factors obtained in the probabilistic analysis.

Variable	Unit	1st analysis		2nd analysis*	
		X_a	α_X	X_a	α_X
$\xi_{R,M}$	–	0.97	0.21	0.96	0.2
A_s	mm²	1954	0.08	1951	0.074
f_y	N/mm²	195.2	0.397	278.5	0.292
d	mm	538.6	0.18	533.5	0.176
b	mm	299.8	0.006	299.7	0.009
f_c	N/mm²	24.84	0.052	24.43	0.062
η	–	1.0	–	1.0	–
$\xi_{E,M}$	–	1.116	-0.42	1.173	-0.4
M_{gc}	kN·m	20.29	-0.02	20.3	-0.015
M_{gp}	kN·m	46.38	-0.11	46.52	-0.08
M_q	kN·m	103.4	-0.76	153.6	-0.82

* With updated values

To improve the estimate of β, the updated values for the reinforcing steel yield strength, f_y, and for the concrete compressive strength, f_c, indicated in Table 8 are adopted. The probabilitistic analysis of the RC beam results in a probability of failure $p_f = 1.6 \times 10^{-5}$ and the corresponding reliablity index increases to $\beta = 4.16$, which is higher than the required reliability level expressed by $\beta_t = 3.8$. Therefore, for the remaining working life the beam can be judged sufficiently reliable for bending resistance. The obtained design values, X_d, and sensitivity factors, α_X, for the basic variables are also indicated in Table 11. The comparatively low value obtained for the sensitivity factor of f_c shows that the concrete compressive strength is not among the most relevant variables with regard to bending failure and more important variables, as f_y, should be preferably updated. Most effective would be updating the parameters of the variable load q, in case corresponding information would be available.

6. Conclusions

In this paper, the assessment of one of the main RC beams of an industrial building, that will be transformed into a cultural centre, has been carried out by applying both partial factor approaches, DVM and APFM, proposed in [5]. Furthermore, the example was solved in a round robin exercise, on the application of TS [2], as well as according to the operational rules developed in [3,4], which are in line with the recommendations of [2]. All the partial factors approaches applied are suitable for the assessment of existing reinforced concrete structures and allow for taking into account the remaining working life adopted, the target reliability indices assumed in each case and the updated information with site-specific data obtained from in situ inspections, measurements and tests.

The assessment based on PFM has been performed in two stages, successively raising the quality of the information available, and was limited to the mid-span cross-section for sake of simplicity. The results of the first stage showed that for none of the applied approaches, the safety for bending resistance was verified. The outcome for the assessment values of the effects of actions in this stage were quite similar in all cases, whereas for bending resistance significant variation can be observed. The latter could be explained by different assumptions for the prior information as well as the procedures adopted for updating the models for material properties, resulting in different characteristic values for these properties.

In the second stage, the application of the procedure in [4] and the DVM [5] revealed that the studied beam does not meet the structural reliability requirements for the remaining service life. On the other hand, the reliability of the beam was verified by applying the APFM, which means that the results obtained with both *fib* Bulletin approaches [5] do not confirm that the APFM often may lead to more conservative results than the DVM. Furthermore, the safety of the beam was verified by five of the experts of the NSBs of CEN member states who applied PFM following the indications of the TS [2]. However, due to the same reasons exposed before, the results obtained for the assessment values of resistance again showed a significant variation.

In practice, the approach proposed in the TS [2] and the operational rules included in [3,4] seem to be more straightforward for practical applications than the *fib* Bulletin 80 methods [5]. Comparing the two approaches, DVM and APFM, it is apparent that the former enables incorporating the updated information more accurately than the latter. In the DVM, the partial factors are updated by means of the representative values (mean or characteristic) and the respective coefficients of variation, while in the APFM updated information is incorporated only through the coefficients of variation of the variables. Moreover, the APFM establishes limiting values for the coefficients of variation of resistance and load effects, in such a way that the model uncertainties could not become dominant variables, considering that for practical purposes this is not advised. The latter affects the incorporation of the updated information and, therefore, may distort application results.

All the considered approaches are based on the use of fixed sensitivity factors, α_X, according to [6] and [7], for determining partial factors. For determining the partial factors for concrete and reinforcing steel resistance, as well as for the actions, permanent or variable, they assume that each of the material properties or load effects are dominating. In contrast, normally only one action and one resistance variables are considered dominant whereas the remaining variables are non dominant [6,7]. This assumption could also distort the results and explain the observed differences.

Finally, the reliability of the beam for bending resistance has been verified by means of a full probabilistic analysis, carried out by using updated values for the concrete compressive strength and the reinforcing steel yield strength. The values obtained for the sensitivity factors show for which variables further updating efforts would be most effective, if needed.

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