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PROBABILISTIC MODELS FOR RESISTANCE VARIABLES IN *fib* MODEL CODE 2020 FOR DESIGN AND ASSESSMENT

Miroslav Sýkora[a,*], Wouter Botte[b], Robby Caspeele[b], Dimitris Diamantidis[c], Aurelio Muttoni[d], Raphaël D. J. M. Steenbergen[b,e]

[a] *Czech Technical University in Prague, Klokner Institute, Šolínova 7, 166 08 Prague, Czech Republic*

[b] *Ghent University, Faculty of Engineering and Architecture, Department of Structural Engineering and Building Materials, Technologiepark-Zwijnaarde 60, 9052 Zwijnaarde, Belgium*

[c] *OTH Regensburg, Faculty of Civil Engineering, Galgenbergstrasse 30, 93053 Regensburg, Germany*

[d] *École Polytechnique Fédérale de Lausanne, School of Architecture, Civil and Environmental Engineering, Station 18, CH-1015 Lausanne, Switzerland*

[e] *TNO Delft, Department of Structural Reliability, P.O. Box 155, NL-2600 AD Delft, Netherlands*

[*] corresponding author: miroslav.sykora@cvut.cz

Abstract.

The *fib* Model Code offers pre-normative guidance based on the synthesis of international research, industry and engineering expertise. Its new edition (draft MC 2020) will bring together coherent knowledge and experience for both the design of new concrete structures and the assessment of existing concrete structures. This contribution presents an overview of the main developments related to the partial factors for materials. In the draft MC2020, the partial factors are presented in tables for clusters of cases depending on consequence classes and variability of basic variables. Furthermore, formulas and background information are provided to facilitate updating of the partial factors. This contribution discusses the different assumptions adopted in MC 2020 for design and assessment. Main changes with respect to the previous version are related to description of the difference between in-situ concrete strength and the material strength measured on control specimens, and to modelling of geometrical variables. The presented comparison of the requirements imposed by Eurocodes and MC 2020 for design reveals insignificant differences. The assessment requirements may be decreased by about 25% when the conditions specified in MC 2020 are satisfied. Hence, the revised MC 2020 will provide designers and code makers with wider possibilities to utilise actual data and long-term experience in assessments of existing structures.

Keywords: Assessment of existing concrete structures, Eurocodes, *fib* Model Code 2020, partial factors, probabilistic models, reliability, updating.

1. Introduction

fib (International Federation for Structural Concrete) has been systematically revising its flagship document - *fib* Model Code (MC). The MC offers pre-normative guidance, synthesis of international research with industry and engineering expertise, and advanced tools for international code writers as well as industry practitioners. The main aspiration of its new edition (draft MC 2020) is to bring together coherent knowledge and experience for both design and assessment and to provide a single code for both new and existing concrete structures [1]. The *fib* MC serves as a pre-standard, intended to provide the basis for development of future Eurocodes and other international standards.

Under *fib* Commission 3 Existing Concrete Structures, Task Group 3.1 is drafting the MC 2020 sections on:

- Reliability requirements (target reliability levels for various limit states in design and assessment and different consequence classes) and
- The partial factor method for design and assessment.

This contribution presents an overview of the main developments related to the latter topic, focusing on resistances. It is demonstrated that the revised MC 2020 should provide designers and code makers with wider possibilities to utilise actual data and long-term experience in the assessment of existing concrete structures.

2. Probabilistic models for resistances

2.1. General

The draft MC 2020 section on the partial factor method is focused on the analysis of basic structural elements such as beams, slabs and columns through simplified (analytical) models. The safety formats

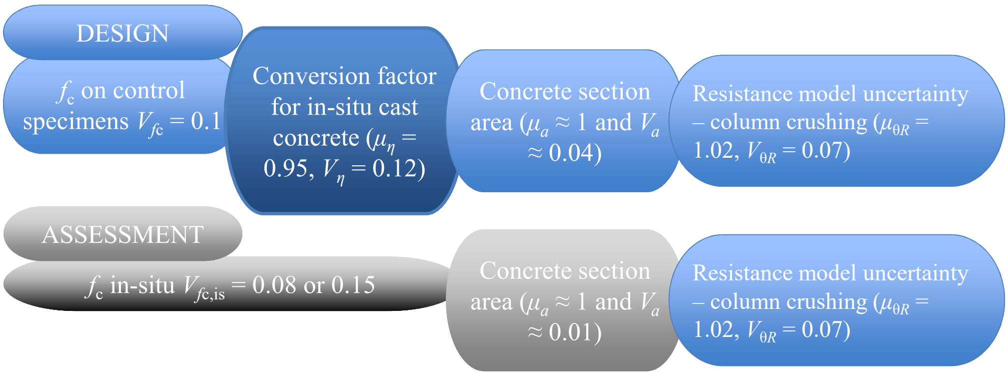

Figure 1. Basic variables considered in resistance model for short columns in design and assessment situations.

to be applied in conjunction with advance numerical models are also addressed by MC 2020 but they remain beyond the scope of this contribution.

MC 2020 emphasises the importance of updating of the models applied in reliability verifications considering all available information about the structure. However, to assist routine applications, the Model Code also provides the conventional probabilistic models for parameters of resistances and load effects considered to be applicable for common reinforced concrete (RC) structures with a reasonable level of approximation. These conventional models are mostly based on the probabilistic models assumed in the background document to the revised prEN 1992-1-1:2021 for design and assessment of concrete structures [3] and they are consistent with those provided by the JCSS Probabilistic Model Code [4], and with the recommendations of *fib* bulletin 80 [5].

Focusing on non-deteriorated RC structures, the flexural resistance of members and the resistance of short columns under compression, R, are obtained as the product of a resistance model uncertainty, θ_R, geometrical property, a, and of material strength, f:

$$R \approx \text{constant} \times \theta_R \times a \times f \qquad (1)$$

where the strength f can cover the factor, η, accounting mainly for the difference between the compressive concrete strength measured on control specimens (f_c) and the in-situ strength ($f_{c,is}$), and then $f_{c,is} \approx \eta \times f_c$. The resistance R is assumed to be lognormally distributed.

It is assumed that reinforcement properties are governing bending failures whereas concrete compressive strength dominates in compressive crushing of columns without significant second order effects. Figure 1 shows the basic variables considered in the resistance model for short columns in design and assessment situations. The flexural resistance, dominated by reinforcement yielding, is described in a similar way.

Table 1 provides the probabilistic models for resistance parameters considered in MC 2020. μ denotes the bias of a variable - systematic deviation of random values of the variable from its characteristic (nominal) value, considered as the ratio of mean to characteristic (nominal) value; V is a coefficient of variation.

2.2. Material properties

The models for concrete strengths, f_c and $f_{c,is}$, and the conversion factor, η, provided in Table 1 are related to ordinary strength concretes (cylinder concrete strengths approximately up to 100 MPa) and standard quality related to in-situ cast RC structures. For high-strength or precast concretes or other than standard execution quality, other models may apply [6]. The coefficient of variation (CoV) of concrete strength commonly decreases with increasing mean strength; see also [7]. For instance, Torrenti and Dehn [8] proposed for in-situ cast concrete:

$$V_{fc} \approx 0.1 \times (f_c/40\,\text{MPa})^{3/2} \qquad (2)$$

The value of μ_η in Table 1 represents concretes in tops of columns; for other governing regions the bias may be increased to 1.0 or up to 1.05 for bottoms of columns. Similarly, for smaller members ($h < 450$ mm) lower biases $\mu_\eta = 0.95$ may be considered while

Basic variable X	Situation	Dist.	Bias μ_X	Coeff. of variation V_X	Note
Concrete compressive strength measured on control specimens (in-situ cast concrete), f_c	Design	LN	$\exp(1.645 V_{fc}) = 1.18$	0.1	$\mu_{fc} = 1.10$ and $V_{fc} = 0.06$ could be considered for precast concrete.
Concrete compressive in-situ strength, $f_{c,\,is}$	Assessment	LN	$\exp(1.645 V_{fc,is}) = 1.14 - 1.28$	0.08–0.15	To be estimated from core tests.
Conversion factor for in-situ cast concrete, η	Design	LN	0.95	0.12	Based on [2]. For precast concrete, lower coefficient of variation can apply.
Yield strength of steel reinforcement, f_y	Design and assessment	LN	$[\exp(1.645 V_{fy})] = 1.08$	0.045	No conversion factor is commonly applied ($\mu_\eta = 1$ and $V_\theta = 0$).
Concrete section area, A_c	Design	N	1	0.04	The case of normal uncertainty in geometry considered as the default case, width of the column $\sim$ 300 mm. MC 2020 provides guidance for other cases.
	Assessment	N	1	0.015	Should be based on *measurements in the existing structure. Normal uncertainty* in geometry as adopted for design may be considered in the absence of measurements. See Section 3 for details.
Effective depth, d	Design	N	1	0.04	Normal uncertainty in geometry as the default case, effective depth $\sim$ 200 mm. MC 2020 provides guidance for other cases.
	Assessment	N	1	0.01	To be based on *measurements in the existing structure. Normal uncertainty* may be considered in the absence of measurements.
Flexural resistance model uncertainty, θ_R	Design and assessment	LN	1.09	0.045	Considered to derive γ_s in both design and assessment situations.
Resistance model uncertainty for crushing of columns without significant 2$^{\text{nd}}$ order effects, θ_R		LN	1.02	0.07	Considered to derive γ_c in both design and assessment situations.

Table 1. Probabilistic models for resistance parameters considered in draft MC 2020.

for larger members ($h \geq 450$ mm), more favourable values may be applied (e.g. $\mu_\eta = 1.03$). MC 2020 indicates that the conversion factor can be influenced by the effects of non-standard quality of execution, various types of binding materials, seasonal aspects, size of the structure etc. More details including an extensive literature review and the analysis of available test results can be found in the background document to prEN 1992-1-1 [3]. Note that the conversion factor η is implicitly covered by test results for in-situ compressive strength, $f_{c,is}$, and $\mu_\eta = 1$ and $V_\eta = 0$ are considered when the model for $f_{c,is}$ is based on core tests.

2.3. Geometrical variables

Regarding geometrical variables, it is assumed that the section area, A_c, relates to the failure modes governed by concrete crushing (typically columns under compression) while the effective depth, d, should be taken into account when reinforcement yielding is dominating (typically bending of beams or slabs). The case of *normal uncertainty* in geometry is considered as the default case for design. For the section area of a square column with fully correlated dimensions $b \times b$, no bias is assumed and the CoV of A_c can be obtained as:

$$V_{Ac} = 2V_b = 0.04 \times (300\,\mathrm{mm}/b)^{2/3} \tag{3}$$

When the two widths are statistically independent, the factor of 2 should be replaced by $\sqrt{2}$ and thus:

$$V_{Ac} = 0.0283 \times (300\,\mathrm{mm}/b)^{2/3} \tag{4}$$

In the case of doubts about a level of correlation, use of Equation 3 is recommended.

When considering *normal uncertainty in geometry*, the statistical characteristics of effective depth can be considered as a function of its nominal value, d_{nom}:

$$\begin{aligned} \mu_d &= 1 - 0.05\,(200\,\mathrm{mm}/d_{nom})^{2/3} \\ V_d &= 0.05\,(200\,\mathrm{mm}/d_{nom})^{2/3} \end{aligned} \tag{5}$$

It is further considered that uncertainty in geometry may be reduced when a decisive geometrical property is *measured in the existing (finished) structure*. This is considered as the default case for the assessment of existing structures. $\mu_{Ac} = 1$ and $V_{Ac} = 0.015$ may then be considered for the section area, and $\mu_d = 1$ and $V_d = 0.01$ for the effective depth.

Further, uncertainty in geometry can also be reduced when an increased *execution quality and quality control* are required. For instance, when higher execution quality is reached (ensuring that the geometrical deviations of Tolerance Class 2 according to EN 13670 on execution of concrete structures are fulfilled), $\mu_{Ac} = 1$ and $V_{Ac} = 0.02$ may be considered for the section area. For the effective depth, the following characteristics may be considered:

$$\begin{aligned} \mu_d &= 1 - 0.025\,(200\,\mathrm{mm}/d)^{2/3} \\ V_d &= 0.025\,(200\,\mathrm{mm}/d)^{2/3} \end{aligned} \tag{6}$$

It can be shown that the partial factor for concrete, γ_C, is insignificantly affected by the variability of the section area in common cases. In contrast, CoVs of the effective depth, yield strength and model uncertainty for bending are of a similar magnitude and the variability of the effective depth should be adequately considered when determining the partial factor for reinforcement, γ_S. Considering a reference period of 50-years, a recommended value of the sensitivity factor for resistance is $\alpha_R = 0.8$ and the target reliability index for design and Consequence Class 2 (medium failure consequences) is $\beta = 3.8$ in MC 2020. Assuming that the three resistance variables (f_y, d, θ_R) have a similar effect on the design or assessment value of the flexural resistance, the sensitivity factor for the effective depth is estimated as $\alpha_d = 0.8/\sqrt{3} = 0.46$ and its design value should approximately correspond to a 4% fractile, $\Phi(0.46 \times 3.8) = 4\,\%$ (with Φ denoting the cumulative distribution function of the standardised normal distribution). For assessment situations, target reliabilities lower than those for design are considered in MC 2020 and a slightly higher fractile would characterise the assessment value of the effective depth. To provide indications for both situations, Figure 2 displays 5% fractiles of the effective depth as a function of its nominal value, d_{nom} (the fractiles are normalised with respect to d_{nom}). The models for effective depth described in this section are applied.

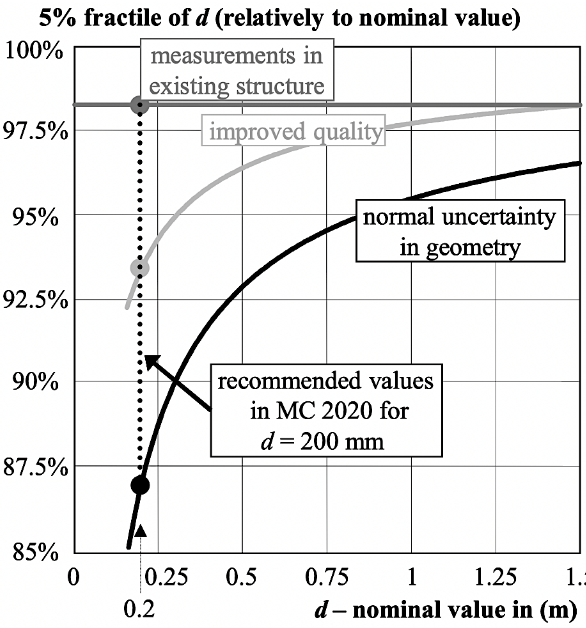

Figure 2. Effective depth - 5% fractiles as a function of nominal value.

Figure 2 shows that the probabilistic models recommended for the effective depth in MC 2020 (for $d = 200$ mm) yield rather lower estimates for the cases of improved quality and particularly for normal uncertainty that are primarily relevant for design situations. In the latter case for $d_{nom} = 1000$ mm, the design value of effective depth increases by about 8% in comparison to the reference case based on $d_{nom} = 200$ mm. In contrast, *measurements in the existing structure* are associated with the smallest uncertainty and the highest design or assessment value of effective depth is obtained.

Regarding considerations of the variability of effective depth in design, MC 2020 provides an additional simplification. The variability of d may be ignored ($\mu_d = 1$ and $V_d = 0$) if the design resistance is based on the design value of effective depth obtained as $d_d = d_{nom} - \Delta d$ where:

- d_{nom} is determined on the basis of the nominal cover.
- Δd is the deviation of the effective depth with:
  - $\Delta d = 15$ mm for reinforcing and post-tensioning steel,
  - $\Delta d = 5$ mm for pre-tensioning steel

2.4. Model uncertainties

In the background document to prEN 1992-1-1 [3], the statistical characteristics for resistance model uncertainties adopted in MC 2020 (Table 1) are based on the comparison of test and model results. The model uncertainty is estimated as the ratio of test resistance to a model estimate [9]. For short columns with negligible second order effects, Moccia et al. [6] investigated cases without and with eccentricity for cylinder concrete strengths up to 100 MPa. They found a bias of 1.02 and CoV of 0.087. The latter is affected by the test variability and the uncertainties in the test parameters. Considering typical variabilities for these effects, the $V_{\theta R}$-value reduced to 0.07.

The statistical characteristics for flexural resistance, $\mu_\theta R = 1.09$ and $V_{\theta R} = 0.045$, were derived in the background document [2] in a similar way. Bending tests were considered for various reinforcement steel classes and calculated strains around 1-1.5%. Both bias and CoV tend to increase with increasing calculated strain.

In the draft MC 2020, the resistance model uncertainties are assumed to have a lognormal distribution [4]; for further information see also Annex A of *fib* bulletin 80 [5] or the numerical study by Sykora et al. [10]. Note that slightly less favourable resistance model uncertainty characteristics (bias around unity and higher CoV) are indicated in MC 2020 complex situations analysed by advanced numerical models [11].

3. Updating of probabilistic models

When specific information about the structure under investigation is available, the probabilistic models for materials and actions and subsequently related partial factors can be updated. Particularly in the assessment of an existing structure, uncertainties in resistances and load effects can often be reduced on the basis of inspections, measurements, and tests. The experience from practical applications suggests that it is often highly beneficial when in-situ concrete strength is investigated and hence the important uncertainty in the conversion factor is eliminated. However, it is also possible that uncertainties to be considered in the reliability verification of the existing structure exceed those considered for a relevant design situation and the partial factors adopted for design might be insufficient.

The prospective Eurocode on the reliability assessment of existing structures, prEN 1990-2:2021, makes distinction between the preliminary and detailed assessment; the latter being in the main focus of this contribution. Regarding the updating of basic variables, the preliminary assessment is typically conducted assuming the default characteristics of basic variables similar to those for new structures while these characteristics are updated for the detailed assessment or in the case of doubts about an appropriate model for the basic variable.

If justified to be relevant for the structure under investigation, prior information can be combined with new information obtained e.g. by measurements and/or tests through Bayesian updating. MC 2020 provides no information about the strength of the prior information provided by the presented conventional models. Following the background document to the reliability basis in Eurocodes [12], it might be assumed that the MC 2020 resistance models are based on prior information associated with the equivalent sample sizes $n' = 1$ to 5 for the mean values (biases) and $v' = 3$ to 10 for standard deviations. Typically, lower values apply for concrete and higher for steel reinforcement. More details can be found in the JCSS Probabilistic Model Code [4] and the JCSS monograph on assessment of existing structures [13].

4. Recommended values of partial factors in draft MC 2020

The draft MC 2020 elaborates on a partial factor method for the reliability assessment of new or existing structures conforming to ISO 2394:2015 for structural reliability principles and EN 1990:2004 for basis of structural design.

4.1. Partial factors for design

For design with respect to Ultimate Limit States (ULSs), the partial factor for materials is obtained according to MC 2020 as:

$$\gamma_M = R_k/R_d \approx [\exp(-1.645\, V_f)]/ \\ \left[\mu_{\theta R}\, \mu_a\, \mu_\eta \exp(-\alpha_R\, \beta \sqrt{V_{\theta R}^2 + V_a^2 + V_\eta^2 + V_f^2}\,\right] \quad (7)$$

CC	γ_C	γ_S	γ_G - self-weight	γ_G - other permanent loads	γ_Q (imposed)	γ_Q (wind)
CC1	1.4	1.1	1.2	1.3	1.3	1.6
<u>CC2</u>	<u>1.5</u>	<u>1.15</u>	<u>1.25</u>	<u>1.35</u>	<u>1.5</u>	<u>1.85</u>
CC3	1.6	1.175	1.25	1.4	1.7	2.1

TABLE 2. Recommended values of partial factors for design according to MC 2020 and various CCs.

CC	γ_C	γ_S	γ_G - self-weight	γ_G - other permanent loads	γ_Q (imposed)	γ_Q (wind)
CC1	1.1	0.975	1.125-1.2	1.25-1.3	1.0	1.05
<u>CC2</u>	<u>1.15</u>	<u>1.0</u>	<u>1.125-1.225</u>	<u>1.275-1.325</u>	<u>1.075</u>	<u>1.15</u>
CC3	1.15	1.0	1.15-1.25	1.3-1.375	1.25	1.3

TABLE 3. Recommended values of partial factors for assessment of existing structures according to MC 2020.

where a denotes a decisive geometrical property and the subscripts "k" and "d" refer to characteristic and design value, respectively.

Equation 7 assumes that:

- The resistance is obtained as the linear product according to Equation 1.
- The characteristic value of the material property (commonly strength) corresponds to a 5% fractile of the respective lognormal distribution.
- Unity characteristic values are considered for resistance model uncertainty and conversion factor, respectively.
- Statistical uncertainty is reflected in the estimate of characteristic resistance.

The same assumptions apply also for relationship (8) in Section 4.2.

Table 2 provides the recommended values of the partial factors for design according to MC 2020 for low, medium and high consequence classes (CC1 to CC3 respectively); γ_G denotes the partial factor for permanent load effects and γ_Q for variable load effects. A detailed discussion on the partial factors for load effects is beyond the scope of this contribution but selected values are provided in Table 2 and in Table 3 in Section 4.2 to allow for comparisons with the design requirements of Eurocodes in Section 5. For the sake of brevity, the following discussion is focused on CC2 only.

The partial factors for materials in Table 2 are derived using relationship (7) and considering:

- The probabilistic models provided in Section 2.
- A 50-year reference period along with the target reliability index $\beta = 3.8$ (CC2) and sensitivity factors $\alpha_R = 0.8$ for resistance and $\alpha_E = -0.7$ for load effects.

The partial factors in Table 2 are in broad agreement with the values provided by Eurocodes. Important for many concrete structures is that γ_G for self-weight can be reduced to 1.25. In contrast, the γ_Q-value for wind action effects is increased. Note that EN 1990:2002 and prEN 1990:2021 keep partial factors for materials fixed across CCs and differentiates by adjusting partial factors for unfavourable load effects only.

The background documents for MC 2020 further demonstrate that similar values of the partial factors are obtained when the informative annual target reliability indices β given in MC 2020 are taken into account along with the sensitivity factors adjusted for a 1-year reference period - $\alpha_R = 0.7$ and $\alpha_E = 0.8$.

Note that MC 2020 provides partial factors for a number of other design situations such as the cases with higher execution quality, effective depth considerably exceeding 200 mm and effective depth measured in the finished structure.

The draft MC 2020 emphasises that the recommended values of the partial factors for design or assessment may only be applied when the conditions of the structure under consideration comply with the assumptions adopted therein. Examples of the cases when the γ-values should be updated include:

- Changes of the target reliability level and/or of the sensitivity factors
- Decreased or increased variability of a basic variable such as a material property, geometry, load effect or model uncertainty
- Basic variables having other probabilistic distributions than those assumed when deriving the γ-values

In particular for existing structures, the partial factors often need to be updated considering structure-specific information.

Higher-level methods should be applied when structure-specific conditions deviate from those commonly accepted. The more advanced approaches include the design (assessment) value method (using e.g. Equation 7), a full-probabilistic approach (with updated models for basic variables and without making assumptions on the values of the sensitivity factors), or risk analysis (by which the target level can be optimised for the particular structure).

4.2. Partial factors for assessment

In order to identify the need for a safety measure to achieve a required reliability level at ULS for the existing structure, a relationship similar to Equation 7 is considered for the partial factor for concrete:

$$\gamma_M = [\exp(-1.645\, V_{fis})] / \left[\mu_{\theta R}\, \mu_{Ac}\, \exp(-\alpha_R\, \beta \sqrt{V_{\theta R}^2 + V_{Ac}^2 + V_{fis}^2}\right] \quad (8)$$

where the in-situ concrete compressive strength evaluated from core tests, $f_{c,\,is}$, is considered and the conversion factor is not included.

Table 3 provides the recommended values of the partial factors for assessment to verify the need of safety measure(s) (termed as "assessment of the existing structure" hereafter for brevity) under the assumptions provided below.

The partial factors for materials in Table 3 are derived using the Equation 8 and considering:

- The probabilistic models provided in Section 2 along with $V_{fc,is} = 0.15$ and $V_{fy} = 0.045$, and various levels of the load effect model uncertainty (on which the γ_G-values are dependent while the γ_Q-values are affected insignificantly)
- An annual reference period along with the target reliability index $\beta = 3.3$ (CC2) and modified sensitivity factors $\alpha_R = 0.7$ and $\alpha_E = -0.8$

Regarding decisive geometrical parameters, related uncertainties are described in MC 2020 for two cases: A) measurements in the finished structure and B) normal uncertainty. Following Section 2.2, the partial factors in Table 3 are based on $\mu_a/a_{nom} = 1$, $V_{ac} = 0.015$, and $V_d = 0.01$ in case A). In case B), it is assumed that reliability verification adopts information on geometry from documentation that is checked by in-situ measurements, but the geometry of the decisive section is not directly verified. $\mu_{Ac}/A_{c,nom} = 1$, $\mu_d/d_{nom} = 0.95$, $V_{ac} = 0.04$, and $V_d = 0.05$ are then considered.

The draft MC 2020 specifies that the partial factors in Table 3 should only be applied when:

- Concrete compressive strength is verified by adequate destructive tests (following standards on the evaluation of concrete in-situ strength such as EN 13791:2019).
- The type and amount of reinforcement is verified.
- Degradation does not affect structural behaviour significantly.

Additionally to the conditions described in Section 4.1, updating of the recommended values may be needed e.g. due to structural damage including deterioration, differences in material properties, detailing provisions or execution tolerances.

Note that the partial factors for resistance may need to be increased to cover epistemic uncertainties related to the lack of knowledge about important details that may have a significant influence on structural behaviour such as actual geometry or amount of reinforcement in joints of frames. In these cases, a sensitivity analysis can help to explore the effect of such uncertainties on structural behaviour. As an example, assumptions for representative favourable and unfavourable situations might be made and structural analysis can then reveal the effect of this uncertainty on structural resistance.

The draft MC 2020 further emphasises that human safety may require reliability levels higher than those provided in the MC for the assessment (such as annual $\beta = 3.3$ for CC2) if structural failure is expected to result in human losses. In particular, for special cases e.g. with many persons at risk or with large failure consequences, higher target levels might be more appropriate and the partial factors given in Table 3 should be increased.

It is seen from Table 3 that all MC 2020 partial factors for the assessment are considerably lower than those provided in the MC and in Eurocodes for design. As an example of the important change, γ_S reduces from 1.15 to 1.0. This decrease can be associated with two differences between design and assessment situations of similar importance:

1. The probability of the fractile associated with the design (assessment) value of resistance increases by an order of magnitude - from $\Phi(\alpha_R \times \beta) = \Phi(0.8 \times 3.8) = 1.18$ ‰ for design to $\Phi(0.7 \times 3.3) = 1.04\%$ for the assessment.
2. The effective depth is assumed to be measured in the existing structure and the bias and CoV becomes more favourable than in design (cf. Figure 2 indicating the difference of about 10% for normal uncertainty and measurements in the finished structure).

In addition, MC 2020 again provides partial factors for other assessment situations (concrete strengths with low variability when $V_{fc,is} = 0.08$ or reliability assessments based on information about geometry from verified documentation).

5. Comparison of design and assessment requirements

In this section, the requirements imposed by the recommended values of the partial factors according to Eurocodes ("EC", primarily intended for design), draft MC 2020 for design (MC design), and for assessment (MC assessment) are compared. Table 4 provides the overview of the values considered in this section, focusing on CC2 only.

The comparison is made on the basis of a fundamental reliability condition, $R_d = E_d$, for compression of the short column with a dominating concrete contribution according to Equation 9 and for bending of the beam with a dominating reinforcement contribution according to Equation 10:

Reference	γ_C	γ_S	γ_G - permanent loads	γ_Q (imposed)	γ_Q (wind)
EC*	1.5	1.15	1.35	1.5	1.5
MC design			1.3**		1.85
MC assessment	1.15	1.0	1.25**,***	1.075	1.15

*Considered here to be primarily intended for design.
**Considering self-weight and other permanent actions equally important.
***Averaged over low and normal levels of load effect model uncertainty, rounded.

TABLE 4. Overview of partial factors considered in comparison of design and assessment requirements (CC2).

$$A_c\, f_{ck}/\gamma_C = \gamma_G\, G_k + \gamma_Q\, Q_k = \gamma_G(1-\varkappa) + \gamma_Q\, \varkappa = E_d(\varkappa) \tag{9}$$

$$A_s\, f_{yk}\, d_{nom}/\gamma_S = \gamma_G\, G_k + \gamma_Q\, Q_k = E_d(\varkappa) \tag{10}$$

where $\varkappa = Q_k/(G_k + Q_k)$is the load ratio based on the characteristic values of load effects; and $E_d =$ design (assessment) values of the load effect.

The ratio of section or reinforcement area required by *MC assessment* to the requirement by Eurocodes becomes:

$$\rho_{c,ex} = \frac{A_{c,ex}}{A_{c,EC}} = \frac{\dfrac{\gamma_{C,ex}\, E_{d,ex}(\varkappa)}{f_{ck,is}}}{\dfrac{\gamma_{C,EC}\, E_{d,EC}(\varkappa)}{f_{ck}}} \approx 1.15 \frac{\gamma_{C,ex}\, E_{d,ex}(\varkappa)}{\gamma_{C,EC}\, E_{d,EC}(\varkappa)} \tag{11}$$

$$\rho_{c,ex} = \frac{A_{c,ex}}{A_{c,EC}} = \frac{\dfrac{\gamma_{S,ex}\, E_{d,ex}(\varkappa)}{f_{yk}\, d_m}}{\dfrac{\gamma_{S,EC}\, E_{d,EC}(\varkappa)}{f_{yk}\, d_nom}} \approx 1.05 \frac{\gamma_{S,ex}\, E_{d,ex}(\varkappa)}{\gamma_{S,EC}\, E_{d,EC}(\varkappa)} \tag{12}$$

In Equation 11, *EC* is based on the characteristic value of concrete strength from control specimens, $f_{ck} = \mu_{fc} \exp(1.645\, V_{fc}) \approx 0.85\, \mu_{fc}$ considering $V_{fc} = 0.1$. MC assessment applies the characteristic value of in-situ strength affected by the bias of the conversion factor. When the effect of concrete aging is-mostly conservatively-ignored, the characteristic value might be estimated as $f_{ck,is} \approx \mu_\eta\, \mu_{fc} \exp(1.645\, V_{fc,is})$ which leads to $f_{ck,is} \approx 0.74\, \mu_{fc}$ for $\mu_\eta = 0.95$ and $V_{fc,is} = 0.15$, and to $f_{ck}/f_{ck,is} \approx 1.15$. For MC design based on f_{ck}, this effect plays no role and Equation 11 reduces to $\rho_{c,des} = \gamma_{C,des}\, E_{d,des}(\varkappa)/[\gamma_{C,EC}\, E_{d,EC}(\varkappa)]$.

In Equation 12, all three approaches are based on the same characteristic value of yield strength, f_{yk}, and there is no numerical effect on ρ_s-values. *EC* is based on the nominal value of effective depth, dnom, while MC assessment assumes that in-situ measurements provide its mean, dm. According to Section 2.3, it is then considered $d_{nom}/d_m \approx 1.05$. For MC design based on d_{nom}, Equation 12 reduces to $\rho_{s,des} = \gamma_{S,des}\, E_{d,des}(\varkappa)/[\gamma_{S,EC}\, E_{d,EC}(\varkappa)]$.

When evaluating ρ-values, the load ratio is a study parameter considered in the range from 0.1 to 0.7 characteristic for RC structures [14] and the values of partial factors are considered according to Table 4. Figure 3 displays the variation of ratio ρ for *MC design*. When $\rho < 1$, *MC design* leads to lower requirements than EC. In this case, there is no difference between flexural resistance and columns under compression as the same material factors are provided by *EC* and *MC design* (cf. Table 4). In the case with imposed loads, *MC design* leads to nearly same levels as *EC* (the former being lower by 1-3%). For wind pressure (and similarly for snow loads), *MC design* (50 y.) requires higher resistances (5-15%) as the γ_W-value is significantly increased in comparison to *EC*.

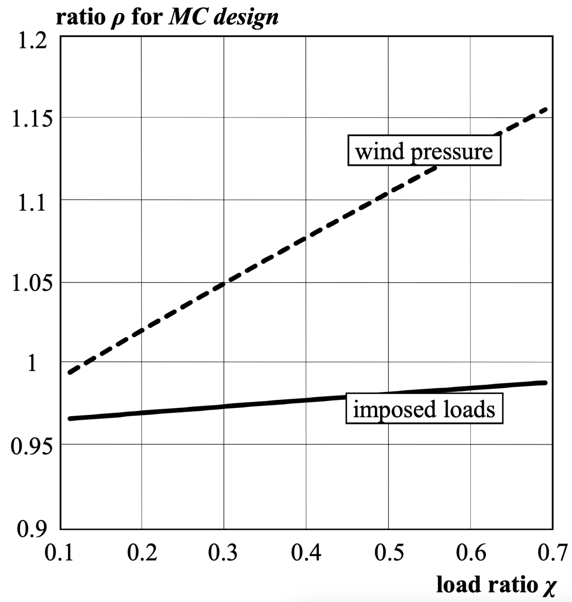

FIGURE 3. Variation of ratio ρ for MC design.

Figure 4 displays the variation of ratio ρ for *MC assessment*. In this case, MC assessment is below *EC* for all situations. In comparison to flexural resistance,

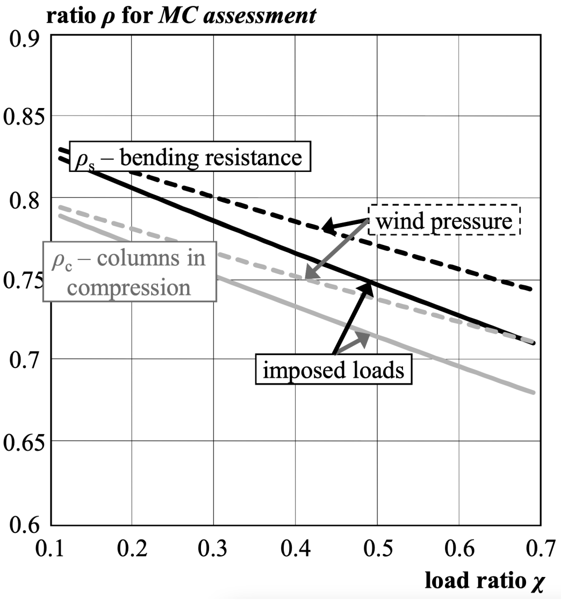

Figure 4. Variation of ratio ρ for *MC assessment*.

systematically lower levels are obtained for columns in compression. For flexural resistance, 75-80% of the reference *EC* level is mostly reached for variable action-dominated or permanent action-dominated structures, respectively. For columns, the level drops to 70-75%. For both failure modes, slightly lower levels are obtained for imposed loads.

The ρ-ratios shown in Figure 4 can be considered low and consequently the reliability levels associated with *MC assessment* might be questioned. Such reduced requirements are attributed mainly to a lower target reliability adopted in the MC for assessment. As an example, for $\varkappa = 0.5$ and wind pressure, the γ_c-value increases from 0.74 (obtained for $\beta = 3.3$) to 0.83 ($\beta = 3.7$). A smaller effect is related to reduced uncertainty as the measurements in the existing structure are assumed.

6. Discussion

Besides the partial factors for design and assessment discussed in this contribution, the draft MC 2020 provides guidance for a number of other design and assessment situations. While new parts of an upgraded structure are to be designed in a way proposed for new structures, a specific approach for the reliability assessment of existing parts of the upgraded structure is provided. This approach is based on requirements adopted also for assessment in Section 4.2, addressing the need of concrete core tests, the verification of the actual reinforcement, and a check of the degradation. However, in contrast to Section 4.2, the target reliability levels for design are considered and dimensions are assumed to be measured in-situ in all cases (as is common practice before structural interventions).

The draft MC 2020 provides detailed guidance for the specification of the target reliability levels considering annual or 50-year reference periods, somehow recommending the use of the former one. While both reference periods can be considered in design situations, in assessment situations MC 2020 focuses only on annual reference period and recommends annual target levels (requiring that the annual target reliability should be fulfilled in every year of a service life of the structure). For the two reference periods, different values of the sensitivity factors are recommended.

7. Summary and concluding remarks

The partial factors for materials, permanent load effects, and variable load effects are presented in draft *fib* MC 2020 in tables for the clusters of cases depending on consequence classes and variability of basic variables. Formulas and background information facilitate updating of the recommended values, i.e. adjusting the partial factors for structure-specific conditions. The key inputs for deriving the partial factors include the probabilistic models of basic variables. Different assumptions may then need to be adopted in design and assessment situations; the main differences are related to the considerations of the concrete strength measured in control specimens (design) and in-situ compressive concrete strength (assessment) as well as to the models for geometrical properties.

The presented comparison of the requirements imposed by Eurocodes and MC 2020 for design reveals on average insignificant differences (with somewhat increased partial factors for snow and wind loads and reduced partial factor for permanent actions in the Model Code). As a key new feature in the draft MC 2020, a lower target reliability is recommended for the assessment and the respective assessment requirements may decrease by about 25%. The conditions specified in MC 2020 should then be satisfied, including in-situ verification of concrete strength, reinforcement, and effects of degradation.

An important principle when developing MC 2020 proposals was that the Model Code should be consistent with Eurocodes, primarily with prEN 1990 for basis of design and assessment and prEN 1992-1-1 for design and assessment of concrete structures including bridges. Regarding specific issues for concrete structures, MC 2020 often offers more detailed guidance than Eurocodes. In particular, the revised MC 2020 will provide designers and code makers with wider possibilities to utilise actual data and long-term experience in assessments of existing structures.

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