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# RELIABILITY OF SHEAR STRENGTH MODELS FOR FIBRE REINFORCED CONCRETE MEMBERS WITHOUT SHEAR REINFORCEMENT

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**ABSTRACT**

The scope and amount of fibre reinforced concrete (FRC) structural applications have seen significant increases. This means that safe and reliable ultimate limit state (ULS) models are necessary for FRC structural members. Among these, shear strength of FRC members without shear reinforcement is highly important due to the brittleness of shear failure. Because of this, the *fib* Model Code 2010 introduced two shear strength models: an empirical model based on Eurocode 2 and a physical model based on the Modified Compression Field Theory. However, a comprehensive reliability assessment of these models has been lacking. Therefore, in this study, the safety format of these models is assessed and the partial safety factors for FRC in shear, $\gamma_c$ and $\gamma_F$ are updated. As a first step, a large database of experimental results on FRC beams is used to determine model uncertainties. Following this, a comprehensive parametric probabilistic analysis is performed using the First Order Reliability Method to determine the adequate values of $\gamma_c$ for different target reliability indices $\beta$. The results of this study show that in order to reach typical reliability indices used in ULS design, $\gamma_c$ and $\gamma_F$ values need to be increased for FRC members without shear reinforcement for both models proposed by the *fib* Model Code 2010.

**KEYWORDS:** fiber reinforced concrete, safety, partial safety factor, beams, database, Model Code.

## 1. INTRODUCTION

Fibre reinforced concrete (FRC) is already recognized as a feasible and more sustainable, alternative to traditional reinforced concrete (RC) solutions in many structural applications [1–4]. One attractive and advantageous use of FRC is in members exposed to shear where its use can permit the elimination or reduction of shear reinforcement. Such a use of FRC can bring significant savings in material costs and time of construction. However, to design such members, reliable ultimate limit state (ULS) models for FRC are necessary. For the case of flexural strength of FRC, comprehensive reliability-based assessments of flexural design models have been performed [5]. At the same time, such work for shear strength, and in particular, for shear strength of members without shear reinforcement, has been lacking.

Shear itself has been a topic that has attracted interest of practitioners and researchers for decades [6] with numerous empirical, semi-empirical and theoretical models having been proposed. Still, the behaviour of members without shear reinforcement has remained a challenging modelling task [7], even for RC members, e.g. the current version of Eurocode 2 [8] contains an empirical model for the shear strength of RC members without shear reinforcement. As for the *fib* Model Code 2010 [9] it presents two models for the shear strength of FRC members without shear reinforcement. The first model—herein referred to as MC2010-A—is the officially adopted model that arose from the work of Minelli et al. [10–12], based on the Eurocode 2 empirical shear resistance model. Although fibres provide several contributions to shear strength—toughness, aggregate interlock, improved bending strength of struts, increased dowel action of the longitudinal reinforcement—the MC2010-A model considers only the contribution of the fibres through the pull-out mechanism and as a type of "distributed reinforcement" [13]. The second model—herein referred to as MC2010-B—is an alternative model presented in the *fib* Model Code 2010, based on the Modified Compression Field Theory [14] and is more consistent with the *fib* Model Code 2010 shear strength model for RC members [9].

So far, research has mostly focused on assessing the "model error", $\delta$, of these models, i.e. the ratio between actual behaviour/shear strength measured in experiments and shear strength predicted by models. Even for RC members, the scatter of model errors for shear strength of members without shear reinforcement is quite high with coefficients of variation (CoV) routinely exceeding 20% [15,16]. For FRC, Marí et al. [17] calculated the model error on a database of experimental results on steel fibre reinforced concrete (SFRC) beams for models MC2010-A and MC2010-B and found average values of 1.04 and 0.99, respectively, and CoV values of 23% and 24%. Even though these results are commensurable with model uncertainties for RC members without shear reinforcement, they are inconclusive about the reliability of FRC shear design since the probability of failure remains unquantified. Both models MC2010-A and MC2010-B are based on an FRC partial safety factor $\gamma_c$ = 1.50, accepted in order to maintain continuity with RC but, apparently, without developing a full probabilistic analysis (at least it is not reported in the *fib* Model Code 2010 background documentation).

However, considering the high scatter associated with FRC tensile/flexural residual strength, the reliability index $\beta$ achieved by these models is not clear in advance. Therefore, the goal of this study is to perform a probabilistic analysis of both *fib* Model Code 2010 models for the shear strength of FRC members without shear reinforcement and calibrate the partial safety factor $\gamma_c$ required for achieving code-prescribed failure probabilities $P_f$ according to different consequence classes. To achieved this, first, the model error was determined on a database of experimental results. Then, a parametric study was performed using the First Order Reliability Method (FORM) considering different probability distributions and parameters of input variables. Finally, based on the results, the FRC partial safety factor was calibrated based on the target reliability index and failure probability. The analysed models are based on SFRC; nonetheless, the models have been reported to be compatible with polymeric fibre reinforced concrete as well [18,19]. The results and conclusions presented herein have the potential to become a reference for future revisions of the national and international design codes for FRC members.

## 2. DESCRIPTION OF THE MODELS AND ASSESSMENT OF MODEL ERROR

### 2.1. Model Code 2010 models for shear strength of FRC members without shear reinforcement

2.1.1. Model MC2010-A

According to the MC2010-A model, shear strength of FRC members "with conventional longitudinal reinforcement and without shear reinforcement" is given by the following expression:

$$V_{\mathrm{Rd,F}} = \left\{ \frac{0.18}{\gamma_\mathrm{c}} \cdot k \cdot \left[ 100 \cdot \rho_\mathrm{l} \cdot \left( 1 + 7.5 \cdot \frac{f_\mathrm{Ftuk}}{f_\mathrm{ctk}} \right) \cdot f_\mathrm{ck} \right]^{1/3} + 0.15 \cdot \sigma_\mathrm{cp} \right\} \cdot b_\mathrm{w} \cdot d \tag{1}$$

where:

$V_{Rd,F}$ is the shear strength in [N];
$\gamma_c$ is the partial safety factor for concrete;
$k$ is a factor considering the size effect, determined as

$$k = 1 + \sqrt{\frac{200}{d}} \leq 2.0 \tag{2}$$

$d$ is the effective depth in [mm];
$\rho_l$ is the longitudinal reinforcement ratio defined as

$$\rho_l = \frac{A_{sl}}{b_w \cdot d} \tag{3}$$

$b_w$ is the smallest width of the cross-section in the tensile zone in [mm];
$f_{Ftuk}$ is the characteristic value of the ultimate residual tensile strength of FRC, considering a crack width $w_u$ = 1.5 mm, according to Eq. (5.6-6) of the *fib* Model Code 2010:

$$f_{Ftu} = f_{Fts} - \frac{w_u}{CMOD_3} \cdot (f_{Fts} - 0.5 \cdot f_{R3} + 0.2 \cdot f_{R1}) \geq 0 \tag{4}$$

CMOD3 is the crack mouth opening displacement (CMOD) of 2.5 mm per EN 14651 [20];
$f_{Ftu}$ is the mean value of the ultimate residual tensile strength of FRC,
$f_{Fts}$ is the mean FRC serviceability residual strength equal to $0.45 \cdot f_{R1k}$;
$f_{R1}$ is the mean FRC residual strength corresponding to CMOD = 0.5 mm;
$f_{R3}$ is the mean FRC residual strength corresponding to CMOD = 2.5 mm;
$f_{ctk}$ is the characteristic value of the tensile strength of concrete in [MPa];
$f_{ck}$ is the characteristic value of the compressive strength of concrete in [MPa] defined as $f_{cm} - 8$ MPa, where $f_{cm}$ is the mean compressive strength;
$\sigma_{cp}$ is the average stress acting on the concrete cross-section $A_c$ [mm2] due to an axial force $N_{Ed}$ [N] caused by loading or prestress ($N_{Ed}$ is positive for compression), i.e. $\sigma_{cp} = N_{Ed}/A_c < 0.2 \cdot f_{cd}$, where $f_{cd}$ is the design compressive strength.
As in Eurocode 2, the shear resistance $V_{Rd,F}$ cannot be smaller than a minimum value $V_{Rd,F,min}$:

$$V_{Rd,F,min} = \left(0.035 \cdot k^{3/2} \cdot f_{ck}^{1/2} + 0.15 \cdot \sigma_{cp}\right) \cdot b_w \cdot d \tag{5}$$

It should be noted that Eq. (1) uses the partial safety factor $\gamma_c$ for "concrete without fibres" [9].

2.1.2. Model MC2010–B

The MC2010-B model defines $V_{Rd,F}$ as an expansion of $V_{Rd,c}$ defined for RC members:

$$V_{Rd,F} = \frac{1}{\gamma_F}\left(k_v \cdot \sqrt{f_{ck}} + k_v \cdot f_{Ftuk} \cdot \cot\theta\right) z \cdot b_w \tag{6}$$

where:

$k_f$ =0.8
$f_{Ftuk}$ is the characteristic value of the FRC ultimate residual tensile strength of FRC at $w_u$
$k_v$ for members without shear reinforcement:

$$k_v = \frac{0.4}{1 + 1500 \cdot \varepsilon_x} \cdot \frac{1300}{1000 + k_{dg} \cdot z} \tag{7}$$

$k_{dg}$ is the aggregate size parameter defined as $32/(16 + dg) \geq 0.75$ where $d_g$ is the maximum aggregate size (if smaller than 16 mm, $k_{dg}$ can be taken as 1.0)

$\varepsilon_x$ is the longitudinal strain at the mid-depth of the effective shear depth $z$

$$\varepsilon_x = \frac{1}{2 \cdot E_s \cdot A_{sl}} \left( \frac{M_{Ed}}{z} + V_{Ed} + N_{Ed} \cdot \left( \frac{1}{2} \mp \frac{\Delta e}{z} \right) \right) \tag{8}$$

$M_{Ed}$,$V_{Ed}$, $N_{Ed}$ are the design bending moment, shear force, and axial force at the shear control section, respectively and $\Delta e$ is the eccentricity of the design axial force.
The limits of the compressive stress field angle $\theta$ are between a minimum value $\theta_{min}$ and 45° with $\theta_{min}$ defined as

$$\theta_{min} = 29° + 7000 \cdot \varepsilon_x \tag{9}$$

Finally, the crack width at the ultimate limit state $w_u$ is defined as

$$w_u = 0.2 + 1000 \cdot \varepsilon_x \geq 0.125 \text{ mm} \tag{10}$$

Importantly, unlike the MC2010-A model, the MC2010-B model in Eq. (6) prescribes the use of the $\gamma_F$ partial safety factor for FRC. Nonetheless, the partial safety factor is also defined as 1.50 [9]

### 2.2. Database of experimental results for model error assessment

In order to adequately assess the error model $\delta$ a comprehensive database of experimental results is needed. In this case, the database of experiments on SFRC beams without shear reinforcement, compiled by Lantsoght and made available online [21,22] was used. The database is described in detail by Lantsoght [21]. The database consisted of 488 results on SFRC beams with longitudinal reinforcement and without shear reinforcement, collected from 65 individual studies. The parameter range of the original database is shown in Table 1 under the "Original database" column. All results reported in the database were from simply supported beams tested in either three- or four-point bending; the majority of had rectangular cross-sections. Significantly, residual strength was not a parameter reported in the database, because not all studies reported these values, for a variety of reasons. However, a large number of fibre properties is reported, as well as a large variety of steel fibre types [21].

Since the range of parameters was very wide, even outside of the scope of the models in some cases, three filtering criteria were imposed:

1. Concrete classes between C12 and C120 were considered (mean compressive strengths between 20 and 128 MPa);
2. Only beams with a longitudinal reinforcement ratio smaller than 4% were considered;
3. Only beams with a clear shear span-to-effective depth ratio larger than 2.0 were considered.

The first criterion was applied as these classes are the lower and upper compressive strength class, respectively, as defined by the *fib* Model Code 2010. This reduced the number of results to 477.

The second criterion was applied as some of the beams had very large reinforcement ratios unrepresentative of practical applications. This further reduced the number of results to 443.

Finally, the third criterion was applied because a different resisting mechanism is activated in these cases, consisting in the direct transfer of load to the support for values of the shear span-to-effective depth ratio smaller than 2.0. This reduced the number of results to 332. The ranges of parameter values for the filtered database are provide in Table 1 under the "Filtered database" heading.

Around 90% of the results were on beams with $f_{cm}$ < 70 MPa, effective depth between 100 and 500 mm and with a longitudinal reinforcement ratio between 1.0% and 3.5%. The fibre volume fraction of 329 out of the 332 beams was below 2.0% (160 kg/m$^3$) and for 296 beams it was below 1.5% (120 kg/m$^3$).

**Table 1.** Example of construction of a table.

| | Original database [22] $n$ = 488 | | Filtered database $n$ = 332 | |
|---|---|---|---|---|
| Parameter | Min | Max | Min | Max |
| $b_w$ (mm) | 50 | 610 | 50 | 610 |
| $h$ (mm) | 100 | 1,220 | 100 | 1,220 |
| $d$ (mm) | 85 | 1,118 | 85 | 1,118 |
| $l_{span}$ (mm) | 204 | 7,823 | 459 | 7,823 |
| $a/d$ (–) | 0.46 | 6.00 | 2.22 | 6.00 |
| $a_v/d$ (–) | 0.20 | 5.95 | 2.00 | 5.95 |
| $\rho_l$ (%) | 0.37% | 5.72% | 0.37% | 3.70% |
| $f_y$ (MPa) | 276 | 900 | 276 | 610 |
| $f_{cm}$ (MPa) | 9.8 | 215.0 | 20.2 | 111.5 |
| $V_f$ (%) | 0.2% | 4.5% | 0.2% | 4.5% |
| $\lambda$ (–) | 25 | 191 | 25 | 191 |
| $f_{uf}$ (MPa) | 260 | 4,913 | 260 | 4,913 |

$l_{span}$ – clear span of the beam; $a/d$ – shear span-to-effective depth ratio measured from left side of loading plate to left side of support;
$a_v/d$ – clear shear span-to-effective depth ratio measured from face of loading plate to face of support; $f_y$ – yield strength of steel reinforcement;
$V_f$ – fibre volume fraction; $\lambda$ – fibre aspect ratio (ratio of fibre length to diameter);
$f_{uf}$ – tensile strength of fibres

### 2.3. Calculation of model errors

As reported above, residual tensile strengths for SFRC in the database of experimental results were not reported. Hence, the $f_{Ri}$ values had to be estimated. This was done using the regressions presented in Figs. 2a (for $f_{R3}$) and 2b (for $f_{R1}$), derived from a statistical analysis of experimental results using the EN 14651 standard [20] on notched 150 × 150 × 600 mm beams, as reported by Venkateshwaran et al. [23], Tiberti et al. [24], Galeote et al. [25], as well as other experimental programs conducted at the Structures and Materials Technology Laboratory (LATEM) of the Polytechnic University of Catalonia (UPC). The database included a large variety of concrete mixes, with the compressive strength range of 15–117 MPa, volume fraction of fibres 0.33–2.52%, fibre aspect ratios 35–110, fibre tensile strength 1000 to 3000 MPa and fibre modulus of elasticity ($E_f$) 190000–210000 MPa. Figure 2 demonstrates a good fit between the proposed linear regressions and the observed data, with the values of the coefficients of determination ($R^2$) of 0.90 and 0.75 for $f_{R1}$ and $f_{R3}$ predictions, respectively.

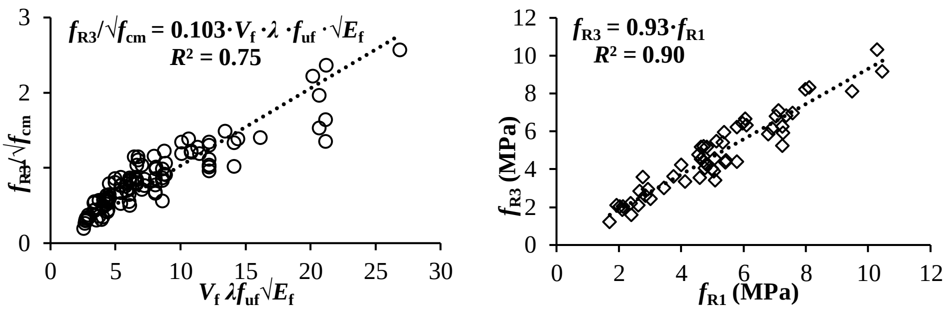


**Figure 1.** Correlations used to assess $f_{R3}$ (left) and $f_{R1}$ (bottom) of the SFRC.

Then, the model error $\delta$ could be determined as $\delta = V_{experiment}/V_{model}$. For this purpose, the partial safety factor $\gamma_c$ and $\gamma_F$ were eliminated from Eqs. (1) and (6), respectively, and mean values of material properties were used. Values of mean compressive strength were used, $f_{R1}$ and $f_{R3}$ were predicted based on the above-presented regression and the tensile strength $f_{ct}$ was calculated based on *fib* Model Code 2010 expressions [9].

For MC2010-A, the upper limit of 2% was applied to the longitudinal reinforcement ratio. For MC2010-B the compression field angle was adopted as $\theta = 45°$ as it provides optimal results. The descriptive statistics of the model errors on the basis of 332 results are reported for models MC2010-A and MC2010-B in Table 2. A box-and-whiskers plot was used to eliminate outliers, i.e., values of $\delta$ smaller than $Q_1 - 1.5 \cdot IQR$ and greater than $Q_3 + 1.5 \cdot IQR$ were excluded (where $Q_1$ and $Q_3$ are the first and third quartile, respectively, and $IQR$ is the "interquartile range", i.e. $Q_3 - Q_1$). The obtained results can be seen to be comparable to previous analyses of the same models as well as analyses of RC model errors.

**Table 2.** Summary statistics of the model error $\delta$ for the MC2010-A and MC2010-B models.

| | **MC2010-A** <br> ***n* = 327** | **MC2010-B** <br> ***n* = 328** |
|---|---|---|
| Mean, $\mu$ | 1.075 | 0.912 |
| Standard deviation, $\sigma$ | 0.245 | 0.266 |
| CoV | 22.8% | 29.1% |

## 3. PARTIAL SAFETY FACTOR CALIBRATION

### 3.1. Design set and probability analysis

To assess the reliability of the studied models, a set of design cases was defined, as presented in Table 3. A typical range of thicknesses of building and bridge-deck slabs, beams, footings, and mat foundations was selected as corresponding to the range of 200–1000 mm. The cross-section width was considered as constant and equal to 300 mm since shear strength depends linearly on it. The effective depth was determined as $d = h - d_s = h - 50$ mm. The variables in Table 3 produce 140 combinations of geometry, longitudinal reinforcement and concrete class.

The process is initiated by computing the reference minimum ($V_{Rd,min}$) and maximum ($V_{Rd,max}$) shear capacities for every combination of the design variables of Table 3. For this purpose, current code value of the resistance safety factors $\gamma_c = \gamma_F = 1.50$ were retained. FRC residual flexural capacities were limited to $f_{R3k,min}$ and $f_{R3k,max}$ of 3 and 10 MPa, respectively. The range was further divided in quarters, so that five design loads were obtained for each case. Considering the 5 design loads, a total of 700 design cases (140 × 5) were generated.

**Table 3.** Range of variables in the design set.

| Parameter | Values of parameters in the design set | | | | | |
|---|---|---|---|---|---|---|
| $b$ (mm) | 300 | | | | | |
| $h$ (mm) | 200 | 400 | 600 | 800 | 1000 | | |
| $\rho_l$ (–) | 0.002 | 0.005 | 0.010 | 0.015 | 0.020 | 0.025 | 0.030 |
| $f_{ck}$ (MPa) | 30 | 50 | 70 | 90 | | | |

The reliability of each design case was assessed by the reliability index $\beta$, related to the probability of failure ($P_f$) by $\beta = -\Phi^{-1}(P_f)$, where $\Phi$ is the cumulative standard normal distribution. FORM [26,27] was used to estimate the reliability index.

A design failure is identified when a negative value is found in the limit state function $G = V_R - V_S = \delta \cdot V_{R,model} - V_S$, where $V_{R,model}$ is the shear resistance predicted by the model and $V_S$ is the shear load. Considering $G$ a function of random variables, the probability of failure is computed as the probability of obtaining a negative value of $G$, i.e.

$$P_f = P(G < 0) = P(\delta \cdot V_R - V_S < 0) \tag{11}$$

To calculate $V_{R,model}$, models MC2010-A and MC2010-B were as described in section 2.1, without the safety factor and using the observed values of the materials and geometry variables. However, it must be noted that in Eq. (8), the value of $V$ has to be multiplied by $\gamma_F$. The set of random variables and the corresponding distribution functions used are summarized in Table 4. The model error was selected as lognormally distributed according to the recommendations of the Joint Committee on Structural Safety (JCSS) [28].

**Table 4.** Definition and distribution of random variables.

| Variable | Description | Statistical model | Mean value ($\mu$) | CoV |
|---|---|---|---|---|
| $\delta$ | Model error | Lognormal | 1.075 | 0.228 |
| $f_c$ | Compression strength | Lognormal | $f_{ck} + 8\ MPa$ | 0.050–0.128 |
| $f_{ct}$ | Tensile strength | Lognormal | $f_{ck} \le 50MPa \rightarrow 0.3 f_{ck}^{\frac{2}{3}}$ <br> $f_{ck} > 50MPa \rightarrow$ <br> $2.12 \cdot \ln\left(1 + \frac{f_{cm}}{10}\right)$ | 0.182 |
| $f_{Ftu}$ | Residual strength at $w_u$ | Lognormal | $1.412 \cdot f_{Ftuk}$ | 0.2 |
| $\Delta b$ | Geometrical error in section width | Normal | $0.003 \cdot b \le 3$ mm | $\frac{4 + 0.006 \cdot b \le 10\text{ mm}}{\mu_b}$ |
| $\Delta d$ | Geometrical error in effective depth | Normal | 10 mm | 1 |

### 3.2. Calibration of the partial safety factor $\gamma_c$ in model MC2010-A and $\gamma_F$ in mode MC2010-B

To establish the relationship between $\gamma_c$ and $\gamma_F$ to $\beta$, the required $f_{Ftu}$ has to be designed for each element belonging to the design set of section 3.1, for different values of $\gamma_c$ and $\gamma_F$. For this purpose, $V_{Rd} = V_{Sd}$ is imposed and the reliability index is computed for each case. The design shear load $V_{Sd}$ was assumed to be deterministic; therefore, the computed reliability index refers to the probability of reaching a shear strength ($V_R$) smaller than the design resistance ($V_{Rd}$): $P(V \le V_{Rd}) = \Phi(\beta_R)$ where $\beta_R$ is the resistance reliability index.

The reliability indices associated to the models MC2010-A and MC2010-B for estimating the shear strength capacity of FRC members without shear reinforcement (Eqs. (1) and (6)) were assessed for a range of safety factors $\gamma_c$ and $\gamma_F$ varying between 1.50 and 2.50, Fig. 2.

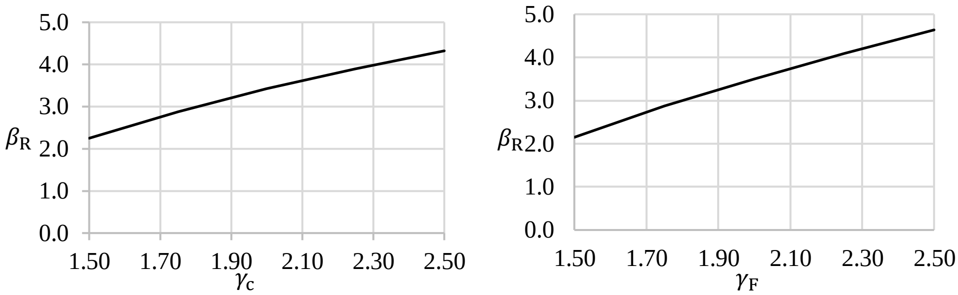


**Figure 2.** Variation of the resistance reliability index with respect to $\gamma_c$ for model MC2010-A (left) and $\gamma_F$ for model MC2010-B (right).

As can be seen, the obtained values of $\beta_R$ are 2.15 for both $\gamma_c$ = 1.50 in model MC2010-A and for $\gamma_F$ = 1.50 in model MC2010-B, with values going up to 4.32 and 4.64 for $\gamma_c$ and $\gamma_F$ equal to 2.50, respectively.

Generally, the target reliability index can be established as a result derived from an analysis coupling economic costs and failure consequences for human beings. As reference, a target reliability index for ULS verifications for a period of 50 years and medium consequences of failure of $\beta_{target}$ = 3.8 is suggested in the *fib* Model Code 2010. Additionally, $\beta_{target}$ of 3.1 and 4.3 are suggested in the same code for low and high consequences of failure, respectively, for a period of 50 years [9]. It should be noted that $\beta_{target}$ accounts for uncertainties associated with both resistance and loads, as random variables, whereas $\beta_{R,target}$ includes only those associated with the resistance. However, it may be assumed that the $\beta_{target}$ and $\beta_{R,target}$ are linearly related through the resistance sensitivity coefficient ($\alpha_R$) as $\beta_{R,target}$ = $\alpha_R \cdot \beta_{target}$, where $\alpha_R$ can be considered as 0.8 [29].

Table 5 gathers the $\beta_{target}$ values for the three reference consequences of failure defined in the *fib* Model Code 2010, together with the computed $\gamma_c$ and $\gamma_F$ required to guarantee these $\beta_{target}$ values in the design set.

**Table 5.** Safety factors $\gamma_c$ and $\gamma_F$ for the target reliability of different consequences of failure.

| Consequence of failure | $\beta_{target}$ For 50 years | $\alpha_R \cdot \beta_{target}$ (taking $\alpha_R$ = 0.8) | $\gamma_c$ | $\gamma_F$ |
|---|---|---|---|---|
| Low | 3.1 | 2.48 | 1.59 | 1.61 |
| Moderate | 3.8 | 3.04 | 1.82 | 1.81 |
| High | 4.3 | 3.44 | 2.01 | 1.97 |

As can be seen, for the reference consequence of failure ($\beta_{target}$ = 3.8), the required $\gamma_c$ is 1.82 for model MC2010-A, which is 21% larger than that currently proposed in the code ($\gamma_c$ = 1.50). A similar value of was found for $\gamma_F$ (1.81) for model MC2010-B.

## 4. CONCLUSIONS

This paper presents reliability-based calibration of partial safety factors $\gamma_c$ and $\gamma_F$ for the shear design of FRC members without shear reinforcement according to two *fib* Model Code 2010 models. For this purpose, a database of experimental results was used for assessing model errors, after which FORM analyses were performed to calibrate the partial safety factors. Based on the obtained results, the following conclusions can be drawn:

- The model errors of the MC2010-A and MC2010-B models were determined to be lognormally distributed with mean of 1.075 and 0.912, respectively, and CoV values of 22.8% and 29.1%, respectively, based on 332 experimental results.
- Based on a probability analysis and a parametric study of 700 individual cases, the relationship between the target failure probability $\beta_R$ and partial safety factors $\gamma_c$ and $\gamma_F$ was determined. It was found that $\beta_R$ values associated with the value of $\gamma_c = \gamma_F$ =1.50—currently adopted in the *fib* Model Code 2010—are 2.25 and 1.88, for models MC2010-A and MC2010-B, respectively, i.e. below the target values of $\alpha_R \cdot 3.1$ = 2.48; $\alpha_R \cdot 3.8$ = 3.04 and $\alpha_R \cdot 4.3$ = 3.44 associated with low, moderate and high consequences of failure, respectively, and a service life of 50 years, considering the resistance sensitivity coefficient as $\alpha_R$ = 0.8.

The results of this study allow confirming that target failure probabilities could not be achieved by the MC2010-A and MC2010-B models for the shear resistance of FRC members without shear reinforcement with $\gamma_c = \gamma_F$ =1.50. Therefore, calibration of the partial safety factor is required for future code revisions. Although the results of the study are dependent on the range of parameters considered in the experimental database used for assessing the model error and on the choice of parameter ranges in the probability analysis, they can still be considered as robust enough for drawing general conclusions.

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