Assessment of the Partial Safety Factor Related to Resistance Model Uncertainties in 2D NLFEAs of R.C. Systems

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ABSTRACT: This work estimates the partial safety factor corresponding to the resistance model uncertainties in non-linear finite element method analyses (NLFEAs) of reinforced concrete structures considering various structural typologies with different behaviours and failure modes (i.e., walls, deep beams, panels). The comparison between the two-dimensional NLFE structural model results and the experimental outcomes is carried out considering the possible solution strategies available to describe the mechanical behaviour of reinforced concrete members in different software codes. Several NLFE structural models are defined for each experimental test in order to investigate the resistance model uncertainty. Then, a consistent treatment of the resistance model uncertainties is proposed following a Bayesian approach identifying the mean value and the coefficient of variation of the resistance model uncertainties. Finally, in agreement with the safety formats for NLFEAs of reinforced concrete structures, the partial safety factor is calibrated.

KEYWORDS: 2D NLFEA; resistance model uncertainties; partial safety factor; reinforced concrete structure; Bayes approach; global safety format

1 INTRODUCTION

The non-linear finite element method analyses (NLFEAs) are one of the most common and practical instruments able to model the actual mechanical response of reinforced concrete members and systems in conditions of both serviceability limit states (SLS) and ultimate limit states (ULS).

In general, the type of uncertainties affecting the structural analyses can be classified in two families: aleatory and epistemic. The aleatory uncertainties are related to the randomness of the variables that governs a specific phenomenon or a resisting mechanism, whereas the epistemic uncertainties are mainly related to the "lack of knowledge" in the characterization of the structural model (Kiureghian & Ditlevsen 1982, Mancini et al. 2017 and Gino et al. 2017). The reliability assessment of a structural system by means of a NLFE analysis has to account for both such families of uncertainty.

Guidelines for NLFE analyses have been recommended by fib Bulletin 45 (2008) and Belletti et al. (2011) in order to perform an accurate calibration of the structural finite element model. Contextually, different safety formats for NLFEAs have been proposed in the literature by several authors (Allaix et al. 2013, Shlune et al. 2012) and international codes (CEN EN 1992-2 2004, *fib* Model Code 2010). The different safety formats allow the calculation of the design structural resistance R_d, by means of Eq.(1):

$$R_d = \frac{R_{rep}}{\gamma_R \cdot \gamma_{Rd}} \quad (1)$$

where R_{rep} is the representative value of the structural resistance (intended as the global strength of the structural member or system) estimated by means of NLFEAs according to the mentioned above safety formats, γ_R is the partial safety factor accounting for the statistical variability of material properties (i.e., aleatory uncertainties), γ_{Rd} represents the partial safety factor accounting for the modelling uncertainties (i.e., epistemic uncertainties).

The present work calibrates the value of the partial safety factor related to the model uncertainties γ_{Rd} for two-dimensional (2D) non-linear finite element method analyses (NLFEAs) of reinforced concrete (r.c.) structures. In detail, twenty-one (21) experimental tests, related to various typologies of structures with different behaviours and failure modes (i.e., walls, deep beams, panels), have been reproduced by means of appropriate plane stress NLFE structural models and compared to the experimental results. Several NLFE structural models are defined for each experimental test in order to study the influence of the model uncertainties on the plane stress NLFE analysis of r.c. systems. Specifically, nine (9) plausible structural models (i.e., solution strategies) are defined adopting three (3) types of software and three (3) assumptions about the mechanical behaviours in tension for concrete. Successively, a consistent statistical treatment of the resistance model uncertainties is proposed according to Bayesian approach (Gelman et a. 2014). Firstly, the prior distributions of the resistance model uncertainties for the nine (9) different solution strategies are evaluated and, then, updated based on the average data coming from the other eight (8) NLFE models in order to assess the posterior distributions. Then, averaging the statistical parameters of the posterior distributions related to the nine (9) different structural models, the mean value and the coefficient of variation for the resistance model uncertainties have been characterised. Finally, in compliance with the safety formats for NLFEAs of r.c. structures (CEN EN 1992-2 2004, *fib* Model Code 2010), the partial safety factor related to the resistance model uncertainties is evaluated and proposed as a function of the pre-assigned reliability level for r.c. structures of new construction.

2 CHARACTERIZATION OF THE MODEL UNCERTAINTIES

As introduced in section 1, the uncertainties related to the definition of the resistance model should be assessed by means of the comparison between the experimental outcomes and the results from numerical simulations. According to Holický et al. (2016), the following aspects have to be considered in order to identify the resistance model uncertainties for NLFEAs:

- the set of the experimental data should contain the parameters necessary for the reproduction of the tests and for the definition of NLFE models;
- the experimental results should be differentiated concerning the typologies of structures and failure modes;
- a consistent statistic and probabilistic analysis of the observed model uncertainties needs to be performed to identify the most likely probabilistic distribution with the associated parameters.

In agreement to JCSS (2001), the resistance model uncertainty, ϑ_i, represents the ratio between the i-th actual resistance (response) estimated from an experimental test $R_{Exp,i}(X,Y)$ and the i-th resistance (or response) estimated by a NLFEA $R_{FEA,i}(X)$ and, may be expressed as follows:

$$\vartheta_i = \frac{R_{Exp,i}(X,Y)}{R_{FEA,i}(X)} \tag{2}$$

where X is a vector of basic variables included into the resistance model (i.e., NLFE model), Y is a vector of variables that may affect the resisting mechanism but are disregarded within the model. Note that the unknown effects of Y variables, if present, are indirectly incorporated by the characterisation of ϑ_i.

In the following, after a brief description of the experimental tests set considered for the investigation, the model uncertainties ϑ_i will be assessed differentiating between nine (9) different solution strategies.

3 NLFE MODELLING AND CASE STUDIES

In this section, the experimental outcomes reported in the scientific literature (Filho 1995, Foster & Gilbert 1998, Lefas & Kotsovos 1990, Leonhardt & Walther 1966, Vecchio & Collins 1982) and related to 21 different typologies of structural members are considered and assumed as a benchmark test set. All these experimental tests, developed respectively on four shear panels, on five deep beams and on eleven walls, have been performed through a monotonic incremental loading process up to failure. The specimens have been realized in laboratory and supported by statically determined configurations. The experimental results, in terms of load vs displacement or shear stress vs angular distortion diagrams, are herein compared to the outcomes from the different NLFE simulations.

3.1 *Definition and differentiation of the structural numerical models*

Several solution methodologies are available in order to perform plane stress NLFEAs of r.c. structures. In this work, three common software codes, marked as Software A, Software B and Software C are adopted with the aim to reproduce the outcomes of the experimental tests set. Each software makes it possible to perform different choices about the hypotheses related to equilibrium, compatibility and constitutive laws. Specifically, for each software, four-node quadrilateral iso-parametric plane stress finite elements, based on linear polynomial interpolation and 2x2 Gauss point's integration scheme are used for the numerical simulations. The assessment of the size of the FE meshes has been performed after an appropriate sensitivity analysis. The non-linear system of equations is solved by means of the standard Newton-Raphson iterative procedure based on the hypothesis of linear approximation (*fib* Bulletin 45 2008).

For each software, the following main characteristics for the finite element models are also assumed: concerning the concrete, non-linear behaviour in compression is considered, including softening with a reduction of the compression strength and shear stiffness (shear retention factor equal to 0.2) after cracking (Bertagnoli et al. 2015). The smeared cracking with fixed crack direction model (Bertagnoli et al. 2015) has been adopted; the steel reinforcements has been modelled with a three linear $\sigma-\varepsilon$ curve (Bertagnoli et al. 2015); discrete and smeared models for the reinforcement assuming a perfect bond interaction (Bertagnoli et al. 2015) has been used; concerning Young's modulus and tensile strength of concrete, the material properties are derived as a function of the experimental compressive strength, according to CEN EN 1992-1-1 (2004). More details concerning the finite elements formulation, shape functions, constitutive models and convergence criteria related to Software A, B and C selected for the present work may be also acknowledged in (Bertagnoli et al. 2015).

In addition to the abovementioned differences concerning the three different software codes, another important differentiation between the NLFE models has been considered with respect to the concrete tensile mechanical behavior. In fact, the interaction between reinforcing bars and concrete between cracks gives rise to the "tension stiffening effect". Performing numerical simulations, this effect may be accounted for through a modification of the constitutive tensile behavior of concrete. This modification consists into the definition of a tension softening law in the post peak concrete tensile response.

In this paper, three different constitutive laws for concrete in tension are considered in order to cover different hypotheses accounting for the tension stiffening effect (Bertagnoli et al. 2015): elastic-brittle (i.e., Brittle), elastic-plastic (i.e., Plastic) and a linear tension softening (i.e., LTS). The constitutive law having a linear tension softening for the concrete tensile behavior has been calibrated in each software with the aim to best fit the experimental outcomes.

The nine different structural models can be defined combining the use of the three different software (i.e., Software A, B and C) with the three different concrete tensile behaviours (i.e., Brittle, Plastic and LTS). The scheme of the solution methodologies adopted in this work is depicted in Figure 1.

Finally, the resistance model uncertainties can be identified and computed, according to section 2, for the different experimental tests of the 21 r.c. members, leading to a total number of 189 NLFE simulations.

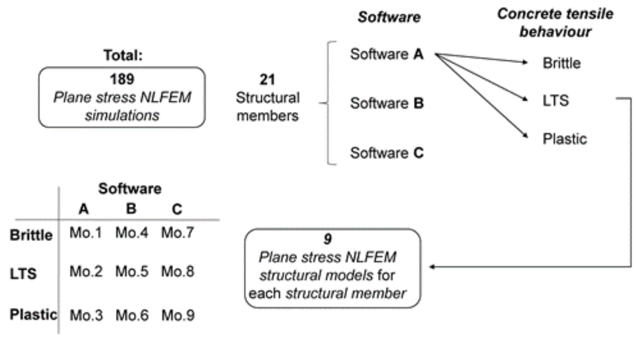


Figure 1. Differentiation between structural models and summary of the benchmark NLFE simulations.

3.2 *Experimental tests considered for the investigation and results*

In this section, the experimental results reported in the scientific literature (Filho 1995, Foster & Gilbert 1998, Lefas & Kotsovos 1990, Leonhardt & Walther 1966, Vecchio & Collins 1982) and related to 21 different typologies of structural members are considered and assumed as the benchmark test set. All these experimental tests, developed respectively on four shear panels, on five deep beams and on eleven walls, have been performed through a monotonic incremental loading process up to failure. The specimens have been realized in laboratory and supported by statically determined configurations. The experimental results, in terms of load vs displacement or shear stress vs angular distortion diagrams, are compared to the outcomes from the total of 189 plane stress NLFE performed, as previously discussed.

More details on the experimental tests set may be acknowledged in the original papers and (Bertagnoli et al. 2015).

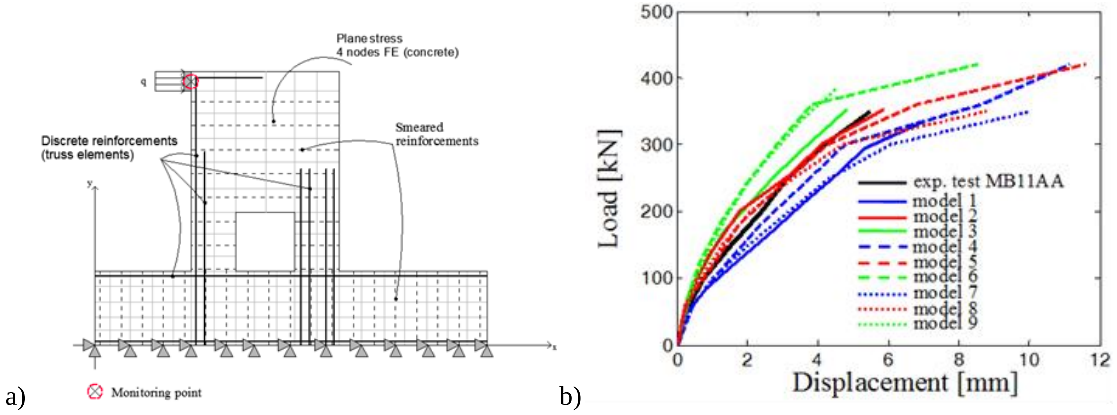


Figure 2. Numerical model schematization (a) and load vs displacement curves for the specimen MB11AA.

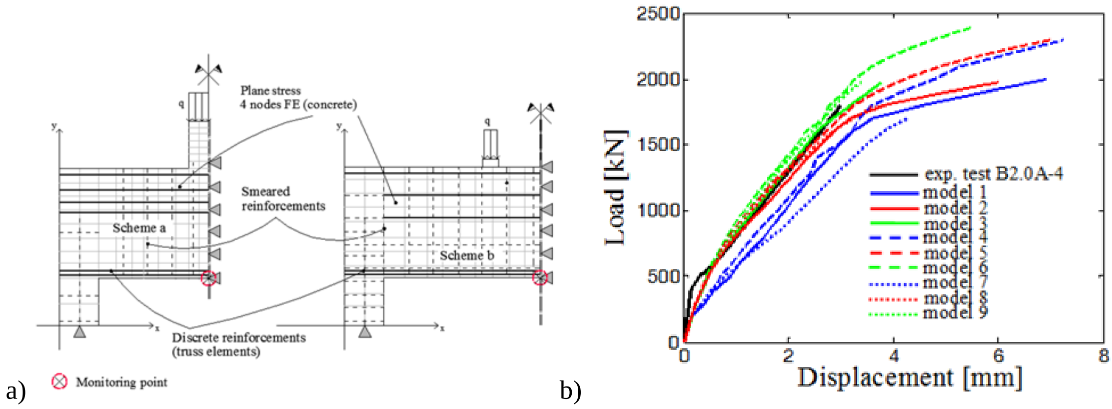


Figure 3. Numerical model schematization (a) and load vs displacement curves for the specimen B2.0A-4.

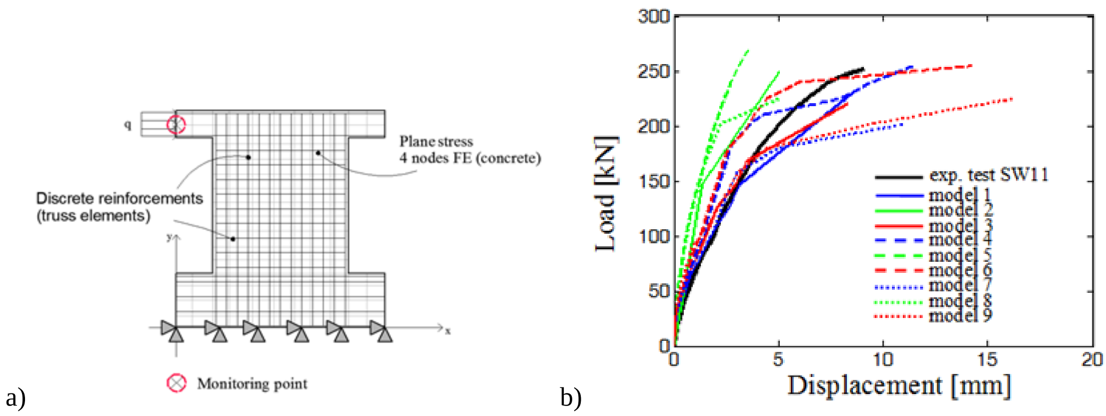
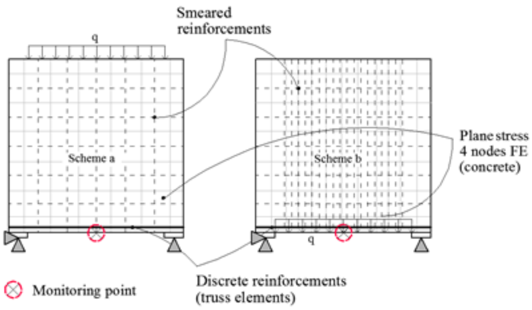


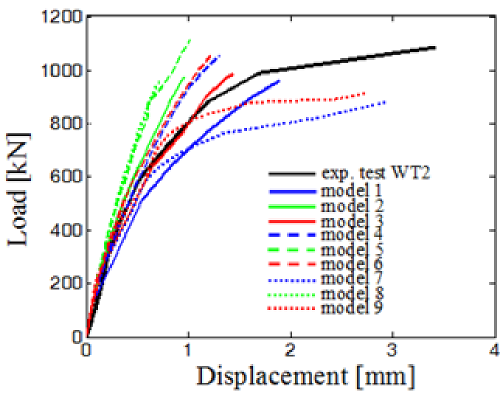
Figure 4. Numerical model schematization (a) and load vs displacement curves for the specimen SW11.

Table 1. Results of the investigation in terms of maximum load.

Ref.	Exp. test	Model uncertainties θ_i [-]								
		Mo. 1	*Mo. 2*	*Mo. 3*	*Mo. 4*	*Mo. 5*	*Mo. 6*	*Mo. 7*	*Mo. 8*	*Mo. 9*
Filho, 1995	MB1AA	1.04	0.99	0.99	0.83	0.83	0.83	1.00	1.00	0.91
	MB1AE	1.07	0.96	1.05	0.93	0.93	0.85	1.06	1.02	0.90
	MB1EE1	1.04	1.07	1.00	0.92	1.03	0.83	0.96	0.92	0.83
	MB1EE1	0.96	0.96	0.96	0.92	0.92	0.83	0.91	0.92	0.92
	MB4EE	1.04	1.08	0.98	0.88	0.95	0.88	1.00	1.00	0.89
Foster & Gilbert 1998	B2.0A-4	0.90	0.91	0.91	0.78	0.78	0.75	1.05	1.00	0.91
	B3.0A-4	0.96	0.96	1.00	0.78	0.83	0.78	1.10	1.10	0.96
	B2.0-1	1.03	1.06	1.09	0.99	0.94	0.84	1.06	1.16	1.16
	B3.0-1	0.93	0.97	0.91	0.83	0.83	0.77	1.05	1.00	0.91
	B2.0-3	0.90	0.90	0.89	0.88	0.82	0.74	1.00	0.93	0.93
Lefas & Kotsovos 1990	SW11	1.10	1.14	1.01	0.99	0.99	0.94	1.25	1.12	1.12
Leonhardt & Walther 1966	WT2	1.13	1.10	1.12	1.03	1.03	0.97	1.23	1.19	1.13
	WT3	0.97	0.97	0.97	0.89	0.89	0.84	1.03	0.97	0.97
	WT4	1.06	1.06	1.05	0.88	0.88	0.88	1.34	1.25	0.95
	WT6	0.81	0.79	0.79	0.85	0.85	0.79	0.99	0.99	0.83
	WT7	0.86	0.83	0.92	0.86	0.86	0.86	1.00	1.00	0.89
Vecchio & Collins 1982	PV10	0.95	0.99	0.90	1.04	0.99	0.93	1.06	1.06	0.99
	PV19	0.76	0.66	0.66	0.81	0.81	0.76	1.05	1.05	1.05
	PV21	0.73	0.73	0.73	0.82	0.82	0.81	1.09	1.09	1.06
	PV22	0.73	0.75	0.75	0.90	0.90	0.90	1.28	1.28	1.28



a)



b)

Figure 5. Numerical model schematization (a) and load vs displacement curves for the specimen WT2.

In Figures 2-6, the experimental and numerical tests results are reported, showing the numerical model schematization of each test with the application of the load “q” or of the shear stress τ. The complete results in terms of maximum load reached during the tests and within the simulation are reported in Table 1. In contrast with the simple geometry of the selected structural members, Figures 2-6 illustrate that it is very difficult to reproduce the actual behaviour. Furthermore, models with plastic and brittle concrete tensile behaviours, do not always represent the upper and the lower bound of the possible response, respectively, due also to the different failure modes. In fact, the resisting mechanism of the experimental test in Figure 2 is characterised by the progressive yielding of the tensile reinforcements and concrete crushing in the column on the right side of the square opening. The failure mode of the experimental test in Figure 3 is characterised by the progressive yielding of the tensile bottom reinforcements and concrete crushing at the top chord close to the column where the load is applied. The failure mode of the experimental test in Figure 4 is characterised by the progressive yielding of the external reinforcements with concrete crushing in the right corner of the wall at the connection with the stiff foundation.

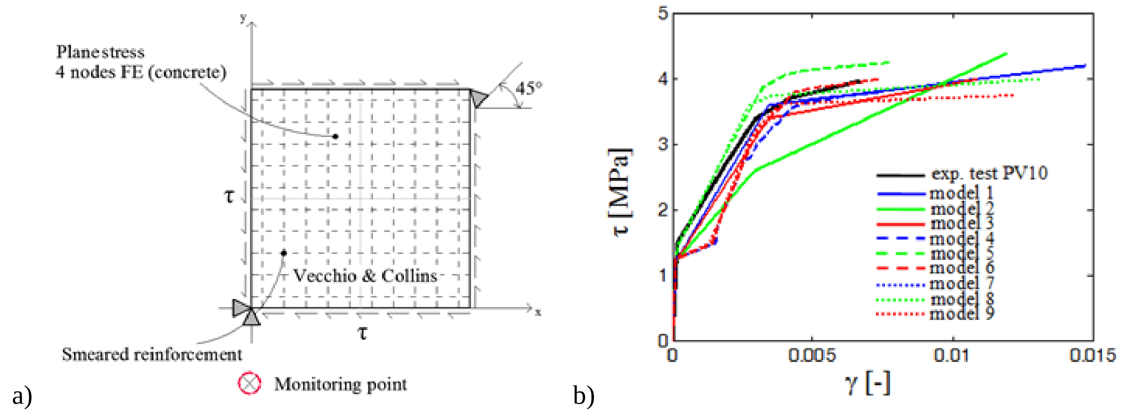


Figure 6. Numerical model schematization (a) and load vs displacement curves for the specimen PV10.

Table 2. Results of the investigation: ratios $\vartheta_i = R_{EXP,i}/R_{FEA,i}$.

Ref.	Exp. test	Model uncertainties θ_i [-]								
		Mo. 1	*Mo. 2*	*Mo. 3*	*Mo. 4*	*Mo. 5*	*Mo. 6*	*Mo. 7*	*Mo. 8*	*Mo. 9*
Filho, 1995	MB1AA	0.91	0.91	0.78	0.78	0.75	1.05	1.00	0.91	0.90
	MB1AE	0.96	1.00	0.78	0.83	0.78	1.10	1.10	0.96	0.96
	MB1EE1	1.06	1.09	0.99	0.94	0.84	1.06	1.16	1.16	1.03
	MB1EE1	0.97	0.91	0.83	0.83	0.77	1.05	1.00	0.91	0.93
	MB4EE	0.90	0.89	0.88	0.82	0.74	1.00	0.93	0.93	0.90
Foster & Gilbert 1998	B2.0A-4	0.99	0.99	0.83	0.83	0.83	1.00	1.00	0.91	1.04
	B3.0A-4	0.96	1.05	0.93	0.93	0.85	1.06	1.02	0.90	1.07
	B2.0-1	1.07	1.00	0.92	1.03	0.83	0.96	0.92	0.83	1.04
	B3.0-1	0.96	0.96	0.92	0.92	0.83	0.91	0.92	0.92	0.96
	B2.0-3	1.08	0.98	0.88	0.95	0.88	1.00	1.00	0.89	1.04
Lefas & Kotsovos 1990	SW11	1.14	1.01	0.99	0.99	0.94	1.25	1.12	1.12	1.10
Leonhardt & Walther 1966	WT2	1.10	1.12	1.03	1.03	0.97	1.23	1.19	1.13	1.13
	WT3	0.97	0.97	0.89	0.89	0.84	1.03	0.97	0.97	0.97
	WT4	1.06	1.05	0.88	0.88	0.88	1.34	1.25	0.95	1.06
	WT6	0.79	0.79	0.85	0.85	0.79	0.99	0.99	0.83	0.81
	WT7	0.83	0.92	0.86	0.86	0.86	1.00	1.00	0.89	0.86
Vecchio & Collins 1982	PV10	0.95	0.99	0.90	1.04	0.99	0.93	1.06	1.06	0.99
	PV19	0.76	0.66	0.66	0.81	0.81	0.76	1.05	1.05	1.05
	PV21	0.73	0.73	0.73	0.82	0.82	0.81	1.09	1.09	1.06
	PV22	0.73	0.75	0.75	0.90	0.90	0.90	1.28	1.28	1.28

The failure modes of the experimental tests in Figure 5 and 6 are characterised by the bottom reinforcement yielding and an arch-tie resisting mechanism with bar yielding, respectively. The results deriving from the mentioned above 189 NLFE simulations are useful to assess the resistance model uncertainties in plane stress NLFEAs of r.c. structures characterised by different failure modes. These results have demonstrated the need to calibrate appropriate values of the corresponding partial safety factor. In Table 2 the results in terms of the ratio $\vartheta_i = R_{EXP,i}/R_{FEA,i}$ are reported.

4 STATISTICAL PROCESSING FOR THE CALIBRATION OF THE PARTIAL SAFETY FACTOR

The present section focuses on the statistical processing of the model uncertainties represented in Table 2 and on the assessment of the partial safety factor for model uncertainties related to plane stress NLFEAs of r.c. structures. As discussed in section 3, the results differ significantly from one commercial software to the another highlighting that also the software selected in order to perform the NLFE simulations represents a source of uncertainty on the assessment of the structural resistance.

4.1 *Probabilistic model for model uncertainties*

First of all, the probabilistic model (Castaldo et al. 2018a,b,c,d,e, Castaldo et al. 2013) for the random variable ϑ (i.e., model uncertainty random variable) should be defined. The probability plot, reported in Figure 7, shows that the overall sample data of 189 results follow a unimodal lognormal distribution. Similar results are achieved for the sample of 21 results related to each model. This result is in compliance with JCSS (2001).

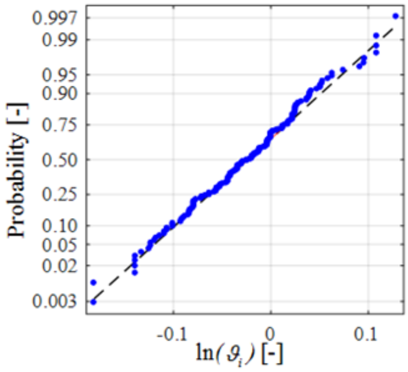


Figure 7. Probability plot of ln(ϑ_i) for all the models.

The Chi-square test has been performed for each structural model (Models 1 to 9) and for the overall sample confirming the assumption of unimodal lognormal distributions.

4.2 *Bayesian updating and assessment of the partial safety factor*

The probabilistic assessment of the resistance model uncertainty for NLFEAs is performed by means of a Bayesian approach (Gelman et a. 2014), which makes it possible to update prior data through new information. Precisely, the prior information is characterised by the model uncertainty values conditional to the use of a specific structural model, that can be represented by its lognormal cumulative distribution function $F(\vartheta \mid M_j)$ (i.e., CDF) (Table 3), whereas the new information, which makes it possible to update the prior results, consists of the numerical outcomes related to the other eight (8) models. Specifically, the prior information for each structural model is updated on the basis of the data obtained from the other eight (8) models in order to evaluate the posterior distributions. Then, the parameters (summarized in the vector z_j for each structural model) of nine (9) distributions are defined and denoted as $F_{Mj}(\vartheta \mid z_j)$ and represent the new information for updating the prior distributions. By means of a Bayesian procedure, the posterior distribution functions $F(\vartheta \mid M_j, z_j)$ for each structural model Mj can be assessed. Since the purpose of this study is the development of a comprehensive probabilistic model, the resistance model uncertainty, after the Bayesian updating, has to be averaged over all the structural models that can be used by engineers.

The posterior non conditioned distribution function $F(\vartheta \mid Z)$, having the distribution parameters (i.e., mean and standard deviation) summarized into the vector Z, can be evaluated averaging the statistical parameters of the nine posterior distributions conditioned to the different structural models $F(\vartheta \mid M_j, z_j)$. The parameters of posterior distributions are reported in Table 4. After the Bayesian updating, models 3, 6, 9 (plastic behaviour for concrete in tension) have shown the largest coefficient of variation. After that, averaging the statistical parameters of the posterior distributions related to the different structural models, the mean value and the coefficient of variation characterizing the resistance model uncertainties can be evaluated (Table 4).

Finally, in compliance with the hypothesis of log-normal distribution for the variable ϑ, the partial safety factor representative of the resistance model uncertainties γ_{Rd} can be determined, according to JCSS (2001), as follows:

$$\gamma_{Rd} = \frac{1}{\mu_\vartheta} \cdot \exp\left(\alpha_R \cdot \beta \cdot V_\vartheta\right) \quad (3)$$

where μ_ϑ is the mean value of the posterior averaged resistance model uncertainties distribution, V_ϑ is the coefficient of variation of the posterior averaged resistance model uncertainties distribution calculated as $\sigma_\vartheta/\mu_\vartheta$, α_R is the first-order-reliability-method (FORM) correction factor, assumed approximately equal to 0.8 and 0.32 as suggested by CEN EN 1992-2 (2004), *fib* Model Code (2010) and explained in the case of dominant and non-dominant resistance variable, respectively, β is the reliability index. The value of γ_{Rd}, in the hypothesis of non-dominant resistance variable according to CEN EN 1992-2 (2004) and *fib* Model Code (2010), with the FORM correction factor α_R equal to 0.32, for structures with an ordinary service life of 50 years and moderate consequences of structural failure (β=3.8), can be set equal to 1.15.

Table 3. Parameters of both prior distributions estimated by means of the minimum likelihood technique.

Structural model	Mean μ_ϑ	Standard deviation σ_ϑ	Coefficient of variation V_ϑ
	[-]	[-]	[-]
	Prior conditioned distributions $F(\vartheta \mid M_j)$		
1	0.94	0.13	0.14
2	0.93	0.15	0.16
3	0.93	0.14	0.15
4	0.89	0.08	0.10
5	0.89	0.08	0.09
6	0.84	0.08	0.09
7	1.07	0.10	0.09
8	1.05	0.09	0.09
9	0.97	0.12	0.12

Table 4. Parameters of both posterior distributions and averaged posterior distribution estimated by means of the minimum likelihood technique.

Structural model	Mean μ_ϑ	Standard deviation σ_ϑ	Coefficient of variation V_ϑ
	[-]	[-]	[-]
	Posterior conditioned distributions $F(\vartheta \mid M_j, z_j)$		
1	1.02	0.13	0.12
2	1.02	0.13	0.13
3	1.01	0.13	0.13
4	0.97	0.14	0.14
5	0.96	0.14	0.14
6	0.95	0.14	0.15
7	1.10	0.11	0.10
8	1.08	0.11	0.10
9	1.03	0.12	0.12
	Posterior distribution $F(\vartheta \mid z)$		
Average statistical parameters	1.01	0.13	0.12

5 CONCLUSIONS

The present investigation focuses on the evaluation the partial safety factor related to the model uncertainties for plane stress non-linear finite element method analyses of reinforced concrete systems. In particular, different experimental tests, concerning different typologies of structures with different behaviours and failure modes (i.e., walls, deep beams, panels), have been numerically simulated by means of appropriate two-dimensional FE structural models (i.e., plane stress configuration) and compared to the experimental outcomes. The resistance model uncertainties have been characterised by appropriate lognormal distributions and, then, a consistent treatment has been proposed following a Bayesian approach. Specifically, the mean value and the coefficient of variation of the resistance model uncertainties are, respectively, equal to 1.01 and 0.12. Finally, concerning ordinary structures of new construction, according to the hypotheses of non-dominant resistance variable for model uncertainties, moderate consequences of structural failure and 50 years service life, the partial safety factor for the resistance model uncertainties in 2D NLFE simulations of reinforced concrete structures equal to 1.15 is proposed.

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