Resistance Model Uncertainties in Plane Stress NLFEAs of Reinforced Concrete Systems subjected to Cyclic Loads

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ABSTRACT: This work describes the resistance model uncertainties in non-linear finite element analyses for reinforced concrete members under cyclic loads for seismic analyses. In detail, several plane stress finite elements analyses using different numerical codes and considering the possible modelling hypotheses are carried out to reproduce the seismic behaviour of various walls experimentally tested. The global resistances achieved from the numerical analyses are compared to the experimental outcomes to assess the influence of the model uncertainties.

KEYWORDS: NLFEAs; model uncertainties; reinforced concrete structures; cyclic loads.

1 INTRODUCTION

Non-linear finite element analysis (NLFEAs) have become one the most common and practical instruments able to describe the actual mechanical behaviour of reinforced concrete (RC) structural systems in different design situations (both ultimate and serviceability) to assess the safety and resilience of infrastructure systems (Troisi and Alfano 2019, 2020). In this context, several guidelines for NLFEAs have been recommended (*fib* Bulletin 45 2008, Belletti et al. 2008, Most 2011) in order to achieve an accurate calibration and definition of the structural FE model. However, the results from such NLFEAs need to be properly analysed in order to satisfy reliability requirements for design and assessment of civil structures (Di Trapani et al. 2018, Di Trapani et al. 2020a, Di Trapani et al. 2020b, La Mazza et al. 2017, Gino et al. 2016, Castaldo and Alfano 2020). To this purpose, several safety formats for NLFEAs have been defined in literature by several authors (Allaix et al. 2013, Shlune et al. 2012, Ftima, M.B. & Massicotte 2012) and international codes (EN1992-2, *fib* Model Code 2010) as well as their applications have been discussed by Castaldo et al. (2019), Blomfors et al. (2016), Val et al. (1997), Cervenka (2013). In mentioned above safety formats, uncertainties related to material properties (i.e., aleatory uncertainties) and the definition of the structural model (i.e., epistemic uncertainties) should be properly addressed in order to derive design values of the global structural resistances as discussed by Castaldo et al. (2018a), Castaldo et al. (2018b), Gino et al. (2019) and Kiureghian & Ditlevsen (2009). Differently to the materials uncertainty, the model uncertainty (i.e., uncertainty mainly related to the definition of the resistance model) associated with NLFEAs is not well known and assessed. In particular, different modelling hypotheses are available in order to define a non-linear FE structural model. In detail, the prediction of the structural response through NLFEAs is always affected by uncertainties because any numerical model derives from a set of hypotheses, mathematical instruments and simplification devoted to describe the real physical world. These model uncertainties can strongly influence the design and assessment of RC structures within the safety formats in case of cyclic loads. For this reason, in-depth characterization of the model uncertainties for NLFEAs of reinforced concrete structures is necessary. Several research studies highlight the need to assess the model uncertainties comparing the numerical and experimental results considering the different modelling hypotheses available to perform NLFEAs. In fact, different assumptions regarding the parameters that govern the equilibrium, kinematic compatibility and constitutive equations in cyclic loading conditions are present in scientific literature and implemented within the different numerical codes. These different choices associated to the mentioned above parameters may lead to discordant results (i.e., epistemic uncertainty (Kiureghian & Ditlevsen 2009)).

This study deals with the comparison between the results of different experimental tests (Oesterle (1979) and Dazio et al. (2009)) related to shear walls and the numerical outcomes derived from NLFEAs. In particular, eighteen plausible structural models based on the NLFE method are developed for each experimental test. The comparison between experimental and numerical outcomes is useful to characterize the model uncertainty random variable related to plane stress NLFEAs of reinforced concrete members under cyclic loads.

2 EPISTEMIC UNCERTAINTIES ACCORDING TO THE SAFETY FORMATS FOR NLFEAs

In general, both aleatory and epistemic uncertainties have to be considered in structural engineering (Castaldo et al. 2018a) to employ probabilistic models (Golzio and Troisi 2013, Garzillo and Troisi 2015) for a safe assessment of structural resistance. With reference to aleatory uncertainties, they are related to the inherent randomness of the structure or of the physical/material properties. Concerning to the epistemic ones, an increase of the knowledge about the specific problem can lead to a significant reduction of the level of uncertainty. In other words, epistemic uncertainties may be represented by the choices performed during the definition of a numerical model (i.e., modelling hypotheses) and, for example, by the assumptions regarding auxiliary non-physical parameters (Castaldo et al., 2018a).

With reference to the global safety formats for NLFEAs proposed by the codes (EN1992-2, fib Model Code 2010) and in literature (Blomfors et al. (2016), Val et al. (1997), Cervenka (2013)), the global design structural resistance R_d can be evaluated as follows:

$$R_d = \frac{R_{rep}}{\gamma_R \gamma_{Rd}} \tag{1}$$

where R_{rep} is the representative value of the global structural resistance (i.e., the peak response in cyclic analyses) evaluated using NLFEAs; γ_R denotes the global resistance partial safety factor accounting for the aleatory uncertainty on the material properties; γ_{Rd} denotes the resistance model uncertainty partial safety factor related to the definition of the non-linear numerical model.

In the particular case of NLFEAs on RC systems under a cyclic loads (e.g., seismic analysis), the influence of the modelling uncertainties may be larger if compared to the static case. In compliance with Holický et al. (2016) and Castaldo et al. (2018a) the resistance model uncertainty random variable, denoted as ϑ, can be computed as:

$$\vartheta = \frac{R_{Exp}}{R_{NLFEA}} \tag{2}$$

where R_{Exp} denotes the global structural resistance derived from the experimental investigation, whereas R_{NLFEA} is the global structural resistance obtained through a NLFEA. In this study, the global structural resistance, experimental or numerical, is herein intended as the peak load achieved during a cyclic analysis in line with the force-based approach of safety formats for NLFEAs (*fib* Model Code 2010).

3 NUMERICAL MODELLING HYPOTHESES AND RESULTS

3.1 *Modelling hypotheses for NLFEAs*

As introduced in the previous sections, different modelling hypotheses are assumed to develop NLFEAs for the RC structures under cyclic loads. Two numerical codes (ATENA, DIANA), denoted with the letters A and B in order to not advertise a specific software house, are adopted to reproduce the experimental tests outcomes adopting four-nodes iso-parametric quadrilateral plane stress finite elements based on linear interpolant shape functions and 2x2 Gauss integration points. The FE meshes are defined after a calibration process specific for each experimental test and numerical code. The standard Newton-Raphson method (*fib* Bulletin N°45) is adopted in order to solve the non-linear system of equations. The maximum number of iterations for each load step has been set equal to 200 and the displacement norm convergence criteria is adopted with a tolerance set equal to 2%. The concrete in compression has been modelled by means of a non-linear behaviour with compression softening and reduction of the compressive strength due to transversal cracking. In detail, the law of Saatcioglu & Razvi (1992) is used in order to model the mono-axial concrete behaviour both in unconfined and confined conditions in zones of the walls where closed stirrups are provided.

The shear behaviour of concrete has been reproduced by means of a constant value of the shear retention factor in order to take into account the influence of aggregate interlock in cyclic response of the shear walls (Belletti et al. 2017). The un-loading/re-loading process is characterized by a linear function secant to the origin both relating to compressive and tensile concrete response (ATENA, DIANA). The uniaxial model for concrete behaviour has been extended to bi-axial plane stress configuration according to Kupfer & Gerstle (1973). The smeared cracking with fixed crack direction model has been adopted in order reproduce cracked state concrete (Riggs & Powell 1986).

The reinforcement has been modelled by means of a tri-linear constitutive law with a cyclic damaging process modelled in compliance with the approach of Menegotto & Pinto (1973). Discrete models of the reinforcement with the hypothesis of perfect bond is performed.

The tensile concrete strength and Young's modulus are derived from the experimental compressive strength in compliance with EN 1992-1.

A further differentiation between the modelling hypotheses has been considered regarding both the concrete tensile mechanical behaviour and the shear stiffness in cracked concrete.

In fact, due to the interaction between reinforcement bars and concrete in cracks, the "tension stiffening effect" occurs. This phenomenon can be considered in the numerical simulations adopting a tension softening constitutive law for the tensile concrete. In detail, elastic-brittle, elastic-plastic and LTS (i.e., linear tension softening) are herein adopted as the different plausible modelling hypotheses Castaldo et al. (2018a). The first two hypotheses are representative of lower and upper bounds for tensile response of concrete. The LTS law has been calibrated through an iterative procedure to best fit each experimental test for each numerical code. Concerning the shear retention factor β, defined as the ratio between the shear stiffness of concrete after and before cracking, three models have been defined for each numerical code, tensile behaviour and experimental test. Specifically, a model with β equal to 0.1, a second model with β equal to 0.3, and a third model with the most appropriate value of β after an iterative research process conditional to the range 0.1-0.3 (Araújo et al. 2010, Fehiling et al. 2002).

In this way, 18 different sets (Figure A1 of the Annex) of modelling hypotheses (i.e., structural models M_j with $j=1,...,18$ representative of the epistemic uncertainties) derive from the combination of the three different concrete tensile behaviours with the three different values of shear retention factor and the two software codes.

3.2 *Experimental tests cases and numerical results*

The experimental investigation of Oesterle (1979) has analysed the response of several anti-seismic shear walls subjected to horizontal cyclical loading. This work examines the specimens denoted as B6, B7, B8 and F2. These walls are 4.57 m high, with a total width of 1.91 m and a web thickness of 0.102 m, and are constrained at the base through a rigid 0.61 m high and 1.22 m wide beam and at the top by a 0.203 m high and 1.22 m wide slab. The specimens B6, B7 and B8 have the same barbell shape (rectangular shape with boundary square elements of 305x305mm), and the same amount of vertical reinforcement. They differ in amount of shear reinforcement and in the axial load. F2 differs in the shape due to lateral I-shaped flanges (0.91x0.102m) and a greater amount of reinforcement for bending and shear. Concerning all cases, vertical load is applied at the top of the specimens and remains vertical throughout the horizontal load cycles. Hence, an increasing displacement is imposed to the top plate providing a series of increments, each of which consists of three complete cycles. In particular, three increments are applied up to the first yield, after that an additional displacement of 25 mm for each increment is imposed. The compressive strength of concrete varies between 21.8 and 49.3 MPa in the different tests, while the axial load varies in a range of 2.93-3.76 MPa. The numerical outcomes, in terms of peak global structural resistance, are listed in Tables A1 of the Annex for all the specimens. Figure A1 (a)-(f) of the Annex illustrates the experimental versus numerical results associated to the wall B6. In general, the peak horizontal load is overestimated, in particular when the elastic-plastic behavior of the concrete in tension is considered. In many simulations, this overestimation can be greater than 30%. However, all the models reproduce the experimental failure mechanism.

The experience of Dazio et al. (2009) refers to the investigation of the response of RC walls subjected to a quasi-static cyclic action. Specifically, four specimens (WSH2, WSH3, WSH4, WSH6) are examined in this study. The shear walls have the same geometry (i.e., 4.03 m high, 2 m wide and 0.15 m thick) and are restrained at the base through a beam integral with the supporting surface of section 0.6 m high and 0.7 wide and protruding 0.4 m from the two sides of the wall. At the top of the wall there is a 0.4 m thick and 0.92 m high beam with a taper to favourite the connection with the wall. The top beam is used to spread the axial load and apply the cyclic loading history by means of actuators located at a distance of 0.39 m from the upper edge. The walls are all reinforced with ϕ6/150 mm located horizontally along the entire width of the wall. The walls differ in the percentage of flexural reinforcement and in the percentage of shear reinforcement in the boundary element and for the value of the applied axial load. The concrete compressive strength and axial load are similar for WSH2, WSH3 and WSH4 with values of 40.5, 39.2 and 40.9 MPa and 691, 686 and 695 kN, respectively. While WSH6 has a higher compressive strength equal to 45.6 MPa and an axial load of 1476 kN. The loading history is in line with the standard protocol defined by Park (1988). The results of numerical simulations expressed as peak global structural resistance are presented in Table A2 of the Annex. Figure A3 (a)-(f) of the Annex illustrates the results for wall WHS3. The assumptions “brittle” or “LTS” for the tensile behaviour of the concrete reproduce efficiently the actual behaviour, while the models with concrete tensile behaviour perfectly plastic overestimate both the resistance and stiffness.

4 DISCUSSION

The results deriving from the abovementioned 144 non-linear FE simulations (18 modelling hypotheses x 8 experimental tests) are useful to estimate the resistance model uncertainties in plane stress NLFEAs for reinforced concrete structures under cyclic loading conditions.

These outcomes have also confirmed the several difficulties to run dynamic analyses in any software with the purpose to reproduce the actual failure behaviour of the structural members. The resistances reported in Table A3 of the Annex demonstrate that the hypotheses “elastic-brittle” and “elastic-plastic” for the tensile concrete do not necessarily represent the bound limits concerning the experimental peak load. Moreover, it can also be observed when the different assumptions are effective to reproduce the cyclic response of the structural members. In fact, in the case of the shear wall WSH2 the difference between the experimental and the numerical resistances is significantly high because the value of ϑ is equal to 0.64. It means that the finite element models are not always able to reproduce the experimental behaviour accurately but sometimes overestimate the peak resistances of the walls under specific hypotheses leading to unsafe estimations.

This aspect is a crucial issue when safety verifications are carried out in seismic zones. Concerning each numerical model M_j, Table A3 of the Annex also reports the mean value and coefficient of variation (CoV). The lowest unsafe bias is achieved for model M_9 as well as the highest dispersion is recognised for model M_{10}, M_{13} and M_{16}. More details may be found in (Castaldo et al. 2020).

5 CONCLUSIONS

This work quantifies the values of the model uncertainties (i.e., epistemic uncertainties) regarding the global structural resistance for plane non-linear finite element analyses of reinforced concrete systems under cyclic loads. Various experimental tests, concerning different walls subjected to cyclic shear actions, have been numerically simulated by means of 144 NLFEAs considering different modelling hypotheses: two different software codes, three different constitutive laws for the behaviour of concrete in tension and three different shear retention factors. In general, it can be observed that the assumption “perfectly plastic” for the tensile behaviour of the concrete always gives a greater overestimation of the structural resistance, and that the different values of the shear retention factor vary the amplitude of the cycle, and therefore the dissipated energy. However, in terms of resistance, a shear retention factor close to 0.1 is the one that best fits the experimental test. The mean bias between the model can be quantified as 0.9 with an average coefficient of variation of 0.07.

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ANNEX

8 structural members ⇨

Concrete tensile behavior	Numerical code A			Numerical code B		
	Shear retention factor β			*Shear retention factor* β		
	0.1	*variable: 0.1-0.3*	*0.3*	*0.1*	*variable: 0.1-0.3*	*0.3*
Brittle	Model 1: M_1	Model 4: M_4	Model 7: M_4	Model 10: M_{10}	Model 13: M_{13}	Model 16: M_{16}
LTS	Model 2: M_2	Model 5: M_5	Model 8: M_8	Model 11: M_{11}	Model 14: M_{14}	Model 17: M_{17}
Plastic	Model 3: M_3	Model 6: M_6	Model 9: M_9	Model 12: M_{12}	Model 15: M_{15}	Model 18: M_{18}

⇨ 144 NLFE simulations

Figure A1. Scheme of the 18 different structural models to investigate the resistance model uncertainties.

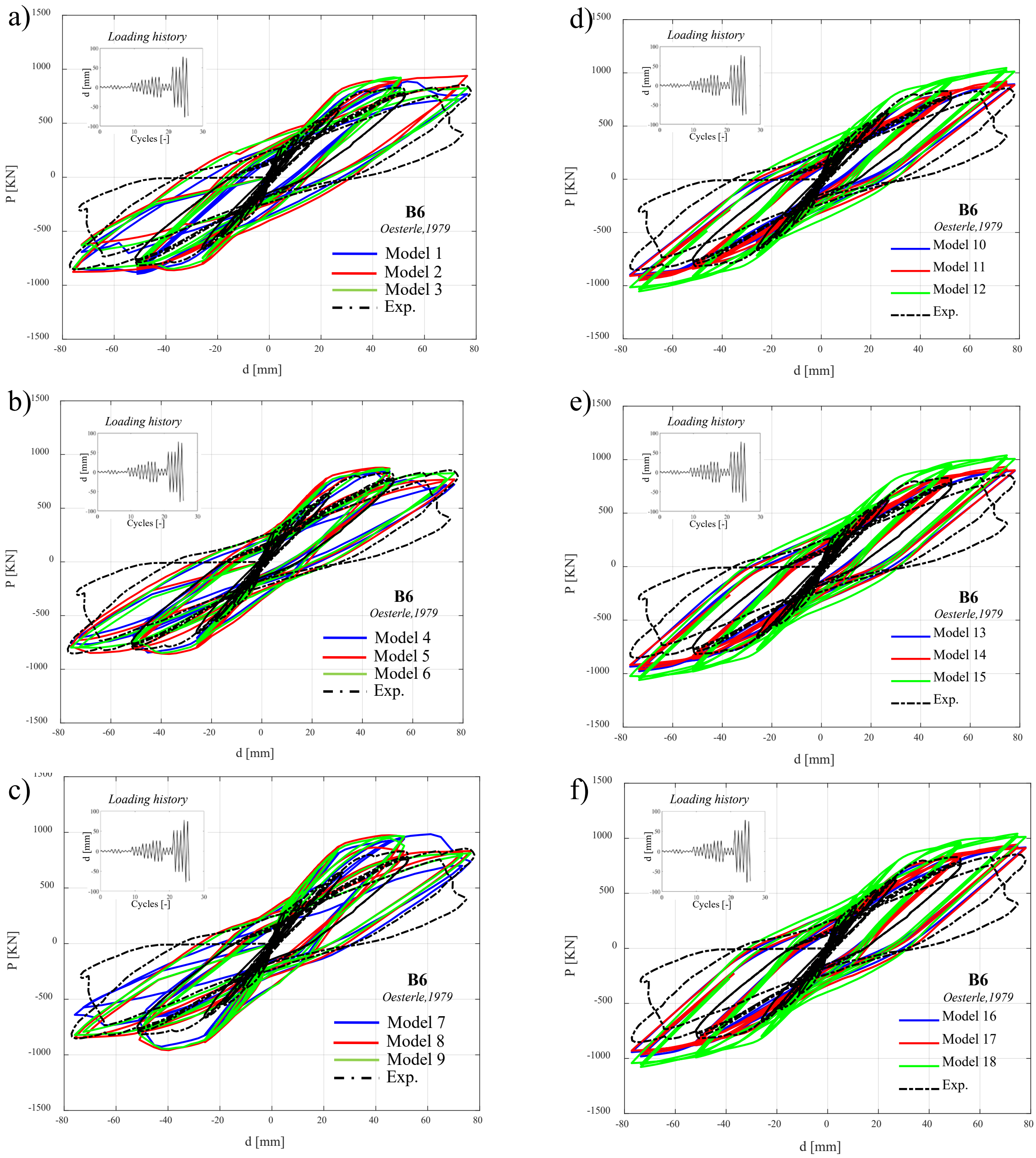

Figure A2. Results from NLFEAs and experimental tests expressed as load vs displacement curves - B6 of Oesterle 1979; (a-c) Numerical code A, (d-f) Numerical code B.

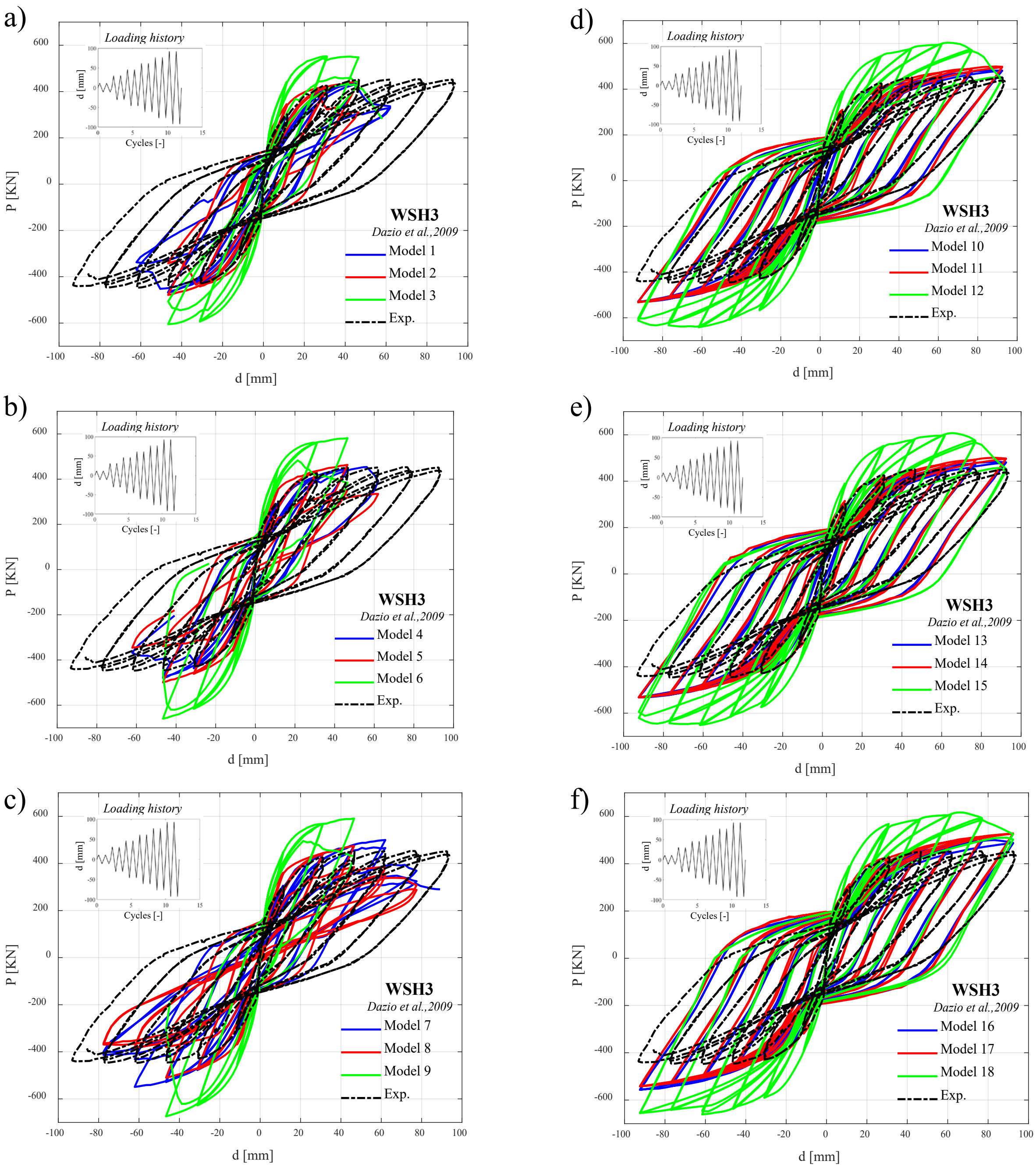

Figure A3. Results from NLFEAs and experimental tests expressed as load vs displacement curves – WSH3 of Dazio et al. 2009; (a-c) Numerical code A, (d-f) Numerical code B.

Table A1. Peak resistances from both the experimental tests Oesterle (1979) and NLFEAs for the different structural models $M_{1\text{-}18}$, Numerical code A and B.

Exp. Test	R_{Exp} MN	R_{M1} MN	R_{M2} MN	R_{M3} MN	R_{M4} MN	R_{M5} MN	R_{M6} MN	R_{M7} MN	R_{M8} MN	R_{M9} MN	R_{M10} MN	R_{M11} MN	R_{M12} MN	R_{M13} MN	R_{M14} MN	R_{M15} MN	R_{M16} MN	R_{M17} MN	R_{M18} MN
B6	0.85	0.89	0.84	0.98	0.93	0.85	0.97	0.94	0.88	0.97	0.91	0.92	1.05	0.91	0.93	1.04	0.94	0.94	1.04
B7	1.01	1.05	1.01	1.28	1.11	1.05	1.30	1.12	1.07	1.34	1.29	1.26	1.46	1.32	1.28	1.47	1.35	1.29	1.47
B8	1.06	1.01	1.09	1.25	1.12	1.17	1.31	1.24	1.17	1.34	1.34	1.30	1.47	1.39	1.30	1.46	1.39	1.30	1.46
F2	0.92	0.86	0.92	1.16	0.92	0.92	1.20	0.95	1.00	1.22	1.15	1.13	1.37	1.15	1.13	1.38	1.22	1.13	1.39

Table A2. Peak resistances from both the experimental tests Dazio (2009) and NLFEAs for the different structural models $M_{1\text{-}18}$, Numerical code A and B.

Exp. Test	R_{Exp} MN	R_{M1} MN	R_{M2} MN	R_{M3} MN	R_{M4} MN	R_{M5} MN	R_{M6} MN	R_{M7} MN	R_{M8} MN	R_{M9} MN	R_{M10} MN	R_{M11} MN	R_{M12} MN	R_{M13} MN	R_{M14} MN	R_{M15} MN	R_{M16} MN	R_{M17} MN	R_{M18} MN
WSH2	0.36	0.38	0.36	0.51	0.39	0.37	0.52	0.44	0.41	0.56	0.38	0.41	0.49	0.37	0.41	0.49	0.39	0.41	0.49
WSH3	0.45	0.44	0.44	0.61	0.45	0.45	0.66	0.55	0.47	0.67	0.48	0.53	0.60	0.50	0.54	0.62	0.48	0.53	0.61
WSH4	0.44	0.45	0.45	0.51	0.48	0.45	0.53	0.48	0.52	0.57	0.41	0.44	0.47	0.42	0.44	0.48	0.44	0.46	0.48
WSH6	0.60	0.63	0.62	0.73	0.67	0.66	0.74	0.69	0.68	0.80	0.62	0.66	0.74	0.62	0.66	0.76	0.64	0.67	0.77

Table A3. $\vartheta = R_{Exp}/R_{NLFEA}$ for all the sets of modelling hypotheses.

Exp. Test	M_1 -	M_2 -	M_3 -	M_4 -	M_5 -	M_6 -	M_7 -	M_8 -	M_9 -	M_{10} -	M_{11} -	M_{12} -	M_{13} -	M_{14} -	M_{15} -	M_{16} -	M_{17} -	M_{18} -
B6	0.96	1.01	0.87	0.92	1.01	0.88	0.91	0.97	0.88	0.94	0.93	0.82	0.94	0.92	0.82	0.91	0.91	0.82
B7	0.96	1.00	0.79	0.91	0.96	0.78	0.90	0.95	0.76	0.78	0.81	0.69	0.77	0.79	0.69	0.75	0.79	0.68
B8	0.95	0.97	0.85	0.89	0.91	0.81	0.86	0.91	0.79	0.80	0.82	0.72	0.77	0.82	0.73	0.76	0.82	0.73
F2	1.07	1.00	0.80	1.01	1.01	0.77	0.97	0.93	0.76	0.80	0.82	0.67	0.80	0.82	0.67	0.76	0.82	0.66
WSH2	0.95	0.99	0.70	0.93	0.97	0.69	0.82	0.88	0.64	0.95	0.88	0.73	0.98	0.87	0.73	0.92	0.87	0.73
WSH3	1.03	1.02	0.75	1.00	1.01	0.69	0.83	0.96	0.67	0.94	0.85	0.75	0.91	0.84	0.74	0.94	0.85	0.75
WSH4	0.98	0.99	0.87	0.95	0.98	0.84	0.91	0.85	0.78	1.09	1.00	0.93	1.05	1.00	0.92	1.02	0.97	0.92
WSH6	0.94	0.96	0.81	0.90	0.91	0.80	0.87	0.88	0.75	0.97	0.90	0.81	0.96	0.90	0.79	0.93	0.89	0.78
Mean	0.98	0.99	0.81	0.94	0.97	0.78	0.88	0.92	0.75	0.91	0.88	0.77	0.90	0.87	0.76	0.87	0.87	0.76
CoV	0.04	0.02	0.07	0.04	0.04	0.08	0.05	0.04	0.09	0.11	0.07	0.10	0.11	0.07	0.10	0.11	0.06	0.10

REFERENCES

Allaix, D.L., Carbone, V.I. & Mancini G. 2013. Global safety format for non-linear analysis of reinforced concrete structures. *Structural Concrete,*14(1): 29-42.

Araújo, D.L., Carmo, L.C., Nunes, F.G.T. & Filho, R.D.T. 2010. Computational modelling of steel fibre reinforced Concrete beams subjected to shear. *IBRACON Structures and Materials Journal*, vol. 3, n1.

ATENA 2D v5. 2014. *Cervenka Consulting s.r.o*. Prague. Czech Republic.

Belletti B., Scolari M. & Vecchi F. 2017. PARC_CL 2.0 crack model for NLFEA of reinforced concrete structures under cyclic loadings. *Computer and Structures,* 191: 165-179.

Belletti, B., Damoni, C. & Hendriks, M.A.N. 2011. Development of guidelines for nonlinear finite element analyses of existing reinforced and prestressed beams. *European Journal of Environmental and Civil Engineering*, 15(9): 1361-1384.

Blomfors, M., Engen, M. & Plos, M. 2016. Evaluation of safety formats for non-linear finite element analyses of statically indeterminate concrete structures subjected to different load paths. *Structural Concrete*, 17(1): 44-51.

Castaldo P., Alfano G. 2020. Seismic reliability-based design of hardening and softening structures isolated by double concave sliding devices, *Soil Dynamics and Earthquake Engineering*, 129,105930.

Castaldo, P., Gino, D. & Mancini, G. 2019. Safety formats for non-linear analysis of reinforced concrete structures: discussion, comparison and proposals. *Engineering Structures*, 193,136-153.

Castaldo, P., Gino, D., Bertagnoli, G. & Mancini, G. 2018a. Partial safety factor for resistance model uncertainties in 2D non-linear finite element analysis of reinforced concrete structures. *Engineering Structures,* 176:746-762.

Castaldo, P., Gino, D., Bertagnoli, G. & Mancini, G. 2020. Resistance model uncertainty in non-linear finite element analyses of cyclically loaded reinforced concrete systems, *Engineering Structures*, 211(2020), 110496, https://doi.org/10.1016/j.engstruct.2020.110496

Castaldo, P., Gino, D., Carbone, V.I. & Mancini, G. 2018b. Framework for definition of design formulations from empirical and semi-empirical resistance models, *Structural Concrete*, 19(4): 980-987.

CEN EN 1992-2 Eurocode 2 *Design of concrete structures, Part 2: concrete bridges*. CEN 2005. Brussels.

Cervenka, V. 2013. Reliability-based non-linear analysis according to fib Model Code 2010. *Structural Concrete*. 14(1): 19-28.

Dazio A., Beyer, K., & Bachmann, H. 2009. Quasi-static cyclic tests and plastic hinge analysis of RC structural walls. *Engineering Structures*, 31: 1556-1571.

Di Trapani, F., Bertagnoli, G., Ferrotto, M.F., Gino, D., 2018. Empirical equations for the direct definition of stress-strain laws for fiber-section based macro-modeling of infilled frames, J Eng Mech, 144(11): 04018101.

Di Trapani, F., Bolis, V., Basone, F., Preti, M., 2020b, Seismic reliability and loss assessment of RC frame structures with traditional and innovative masonry infills, Engineering Structures, 208, 110-306.

Di Trapani, F., Giordano, L. & Mancini, G. 2020a. Progressive Collapse Response of Reinforced Concrete Frame Structures with Masonry Infills, *J. Eng. Mech.*, 146(3): 04020002.

DIANA FEA BV 2018. Delftechpark 19a 2628 XJ Delft. The Netherlands.

EN 1992-1-1. *Eurocode 2 – Design of concrete structures. Part 1-1: general rules and rules for buildings*. CEN 2014. Brussels.

Fehiling, E. & Bullo, T. 2002. Ultimate load capacity of reinforcement steel fibre concrete deep beams subjected to shear. *Finite elements in Civil Engineering Applications*, Hendriks & Rots.

fib Bulletin N°45. 2008. *Practitioner's guide to finite element modelling of reinforced concrete structures – State of the art report*. Lausanne.

fib Model Code for Concrete Structures 2010. 2013. Lausanne.

Ftima, M.B. & Massicotte, B. 2012. Development of a reliability framework for the use of advanced nonlinear finite elements in the design of concrete structures. *Journal of Structural Engineering*, 138:1054–64.

Garzillo C., Troisi R. 2015. Le decisioni dell'EMA nel campo delle medicine umane. pp.85-133. In EMA e le relazioni con le Big Pharma - I profili organizzativi della filiera del farmaco - ISBN:9788892102279 - G. Giappichelli

Gino, D., Bertagnoli, G. & Mancini, G. 2016. Effect of endogenous deformations on composite bridges. *Recent Progress in Steel and Composite Structures - Proceedings of the 13th International Conference on Metal Structures*, ICMS 2016, *15-17 June 2016, Zielona Gora, Poland,* pp. 287-298.

Gino, D., Castaldo, P., Bertagnoli, G., Giordano, L. & Mancini, G. 2019. Partial factor methods for existing structures according to fib Bulletin 80: Assessment of an existing prestressed concrete bridge. *Structural Concrete*, 2019: 1–17. https://doi.org/10.1002/suco.201900231

Golzio L. E., Troisi R. 2013. 'The value of interdisciplinary research: a model of interdisciplinarity between legal research and research in organizations". pp.23-38. In *Journal For Development And Leadership*, vol. 2.

Holický, M., Retief, J.V. & Sikora, M. 2016. Assessment of model uncertainties for structural resistance. *Probabilistic Engineering Mechanics.* 45: 188-197.

Kiureghian, A.D. & Ditlevsen O. 2009. Aleatory or epistemic? Does it matter?. *Structural Safety,* 31: 105-112.

Kupfer, H.B. & Gerstle, H.K. 1973. Behavior of Concrete under Biaxial Stresses. *Journal Engineering Mechanics Division,* 99(4).

La Mazza, D., Giordano, L., Castaldo, P. & Gino, D. 2017. Assessment of the efficiency of seismic design for structural robustness of rc structures , *Ingegneria Sismica*, 34(3-4): 63-77.

Menegotto, M. & Pinto, E. 1973. Method of analysis for cyclically loaded reinforced concrete plane frames including changes in geometry and non-elastic behavior of elements under combined normal force and bending, *Proceedings, IABSE Symposium*. Lisbon, Portugal.

Most, T. 2011. Assessment of structural simulation models by estimating uncertainties due to model selection and model simplification. *Computers and Structures*, 89(17-18): 1664-1672.

Oesterle, R.G. 1979. Earthquake resistant structural walls - Tests of isolated walls - Phase II. *Portland cement association.*

Park, R. 1988. Ductility evaluation from laboratory and analytical testing. *In: Proceedings of the 9th world conference on earthquake engineering.* Vol. III.

Riggs, H.R. & Powell, G.H. 1986. Rough crack model for analysis of concrete. *J. Eng. Mech. Div. ASCE*, 112(5): 448-464.

Saatcioglu, M. & R Razvi, S.R 1992. Strength and ductility of confined concrete. *Journal of Structural engineering,* 118(6):1590–1607.

Shlune, H., Gylltoft, K. & Plos, M.. 2012. Safety format for non-linear analysis of concrete structures. *Magazine of Concrete Research,* 64(7): 563-574.

Troisi R., Alfano G. 2019. Towns as Safety Organizational Fields: An Institutional Framework in Times of Emergency. *Sustainability* 2019, 11, 7025, doi:10.3390/su11247025.

Troisi R., Alfano G. 2020. Firms'crimes and land use in Italy. An exploratory data analysis. *New Metropolitan Perspectives, International Symposium – 4th edition*, 27-30 May 2020, pp 10.

Val, D., Bljuger, F. & Yankelevsky, D. 1997, Reliability evaluation in nonlinear analysis of reinforced concrete structures. *Structural Safety*, 19(2): 203-217.