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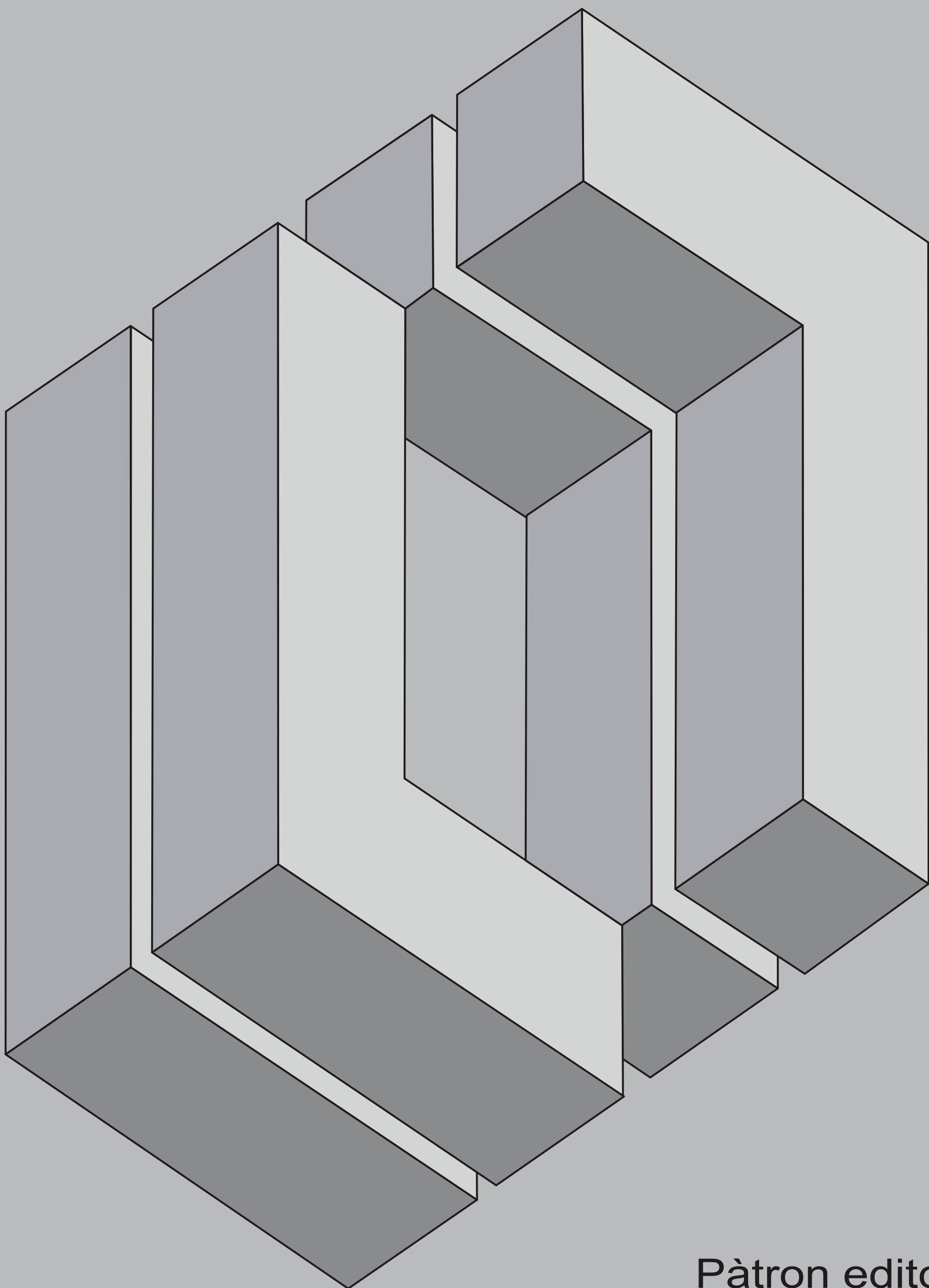

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Anno XXXIV – Special Issue – settembre 2017

Seismic Vulnerability of Structures and Infrastructures: Strategies for Assessment and Mitigation - PART I

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International Journal of Earthquake Engineering

Anno XXXIV – Special Issue – settembre 2017

Seismic Vulnerability of Structures and Infrastructures: Strategies for Assessment and Mitigation - PART II

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SPECIAL ISSUE

Seismic Vulnerability of Structures and Infrastructures: Strategies for Assessment and Mitigation

Editorial

Paolo Castaldo, Liborio Cavaleri, Fabio Di Trapani

Seismic engineering is constantly looking for new strategies and methods that provide designers with the opportunity to get more and more efficient solutions both from the performance and from the economic point of view. In this context, the scientific community is called not to miss its support and to face the new challenges coming from the observation of the damage caused by the recent earthquakes. This Special Issue of "Ingegneria Sismica" collects some works focusing the theme of the seismic vulnerability, some of them specifically refer to mitigation strategies while others address modeling strategies. Obviously, for the multiplicity of aspects involving the theme of "seismic vulnerability", only a few issues have been discussed. In particular, some works deal with the theme of base isolation from the point of view of design and seismic reliability, others are focused on the theme of structural robustness considered as a further goal of seismic design. Further papers are deepening the problems of interaction between frames and masonry infills, providing simplified modeling strategies, suggesting criteria to reduce interaction effects, but above all by providing methods for the analysis of combined in-plane / out-of-plane behavior. The themes related to modeling are also addressed with particular care. Other papers in fact face the issues of model uncertainty and of the reliability of constitutive laws for normal and high strength confined concrete, being the latter of fundamental importance for the assessment of the rotational capacity of reinforced concrete structural elements under seismic actions. Moreover, the problem of the reliability of seismic vulnerability assessment methods is also discussed, with particular reference to multimodal "pushover" analysis. This Special Issues is composed of 10 paper produced by 29 authors, to whom we would like to address a special thank for the effort made during the editorial phases.

Finally, the Guest Editors, Paolo Castaldo, Liborio Cavaleri e Fabio Di Trapani, wish to thank Prof. Gianmario Benzoni and Prof. Rosario Montuori for this opportunity and for their assistance.

SPECIAL ISSUE

Seismic Vulnerability of Structures and Infrastructures: Strategies for Assessment and Mitigation

Editoriale

Paolo Castaldo, Liborio Cavaleri, Fabio Di Trapani

L'ingegneria sismica è costantemente alla ricerca di nuove strategie e metodi che forniscano ai progettisti la possibilità di ottenere soluzioni sempre più efficienti sia dal punto di vista prestazionale che da quello economico. In questo contesto il mondo scientifico è chiamato a non far mancare il suo supporto e raccogliere le nuove sfide lanciate dalla osservazione dei danni causati dagli ultimi terremoti. Questo numero di Ingegneria Sismica raccoglie alcuni lavori che si inquadrano nel tema della vulnerabilità, alcuni di essi specifici nelle strategie di mitigazione mentre altri specifici nelle strategie di modellazione. Ovviamente, per la molteplicità degli aspetti che include il tema "vulnerabilità sismica", solo alcune questioni sono state trattate. In particolare, alcuni lavori affrontano il tema dell'isolamento sismico dal punto di vista della progettazione e da quello dell'affidabilità sismica, in altri si affronta il tema della robustezza strutturale considerata come ulteriore obiettivo della progettazione sismica, in altri ancora sono approfonditi i problemi di interazione tra telai e tamponamenti fornendo procedure semplificate di modellazione, suggerendo criteri destinati a ridurre l'interazione fra gli stessi, ma soprattutto proponendo metodi di analisi per il comportamento combinato nel piano e fuori piano. La tematiche correlate alla modellazione sono inoltre affrontate con particolare attenzione. È discusso il problema delle incertezze di modello e quello della affidabilità delle leggi costitutive per il calcestruzzo confinato a normale ed alta resistenza, di fondamentale importanza per la valutazione della capacità rotazionale degli elementi strutturali in c.a. in presenza di azioni sismiche. Viene anche affrontata la problematica dell'affidabilità dei metodi di valutazione della vulnerabilità sismica, con particolare riferimento alla analisi "pushover" multimodale. Complessivamente sono presenti 10 lavori prodotti da 29 autori a cui va uno speciale ringraziamento per lo sforzo profuso per la composizione di questo numero.

Infine, gli Editori, Paolo Castaldo, Liborio Cavaleri e Fabio Di Trapani, desiderano ringraziare il Prof. Gianmario Benzoni e il Prof. Rosario Montuori per l'opportunità ricevuta e per la loro assistenza per la realizzazione di questo numero speciale.

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Seismic Vulnerability of Structures and Infrastructures: Strategies for Assessment and Mitigation

A QUANTIFICATION OF MODEL UNCERTAINTIES IN NLFEA OF R.C. SHEAR WALLS SUBJECTED TO REPEATED LOADING

Diego Gino[1], *Gabriele Bertagnoli*[1], *Dario La Mazza*[1], *Giuseppe Mancini*[1]

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SUMMARY: *The non–linear finite element method allows to perform the prediction of the complex behaviour of reinforced concrete members from serviceability to ultimate conditions. The non-linear finite elements analysis can be successfully employed to solve a wide field of problems in structural engineering. In this context, in order to adopt NLFEM software for structural safety verification the level of uncertainties presented by such kind of analysis have to be deeply quantified.*
Appreciable results may be achieved simulating the response of structural components subjected to progressive incremental loading process. However, as in the case of seismic assessment of R.C. members, the same results may not be reached under repeated loading configuration.
In this paper two commercial non-linear finite element codes have been considered in order to assess the uncertainties in the simulation of non-linear concrete behaviour under repeated loading by adopting plane stress models. The resisting model uncertainties (intended as the ratio between experimental and NLFEA outcomes) in reproducing the experimental achievements related to six laboratory tests on shear walls with results known from the literature have been assessed. The overall capability of the two codes to predict the actual structural behaviour under cyclic loading is commented and mean value, variance and coefficient of variation of model uncertainties is quantified and discussed.

KEYWORDS: *Reinforced concrete, shear walls, modelling unceratinties, cyclic loading*

1 Introduction

The behaviour of reinforced concrete structures, from serviceability conditions to failure, is the result of the combination of different phenomena as cracking development, progressive steel bars yielding and second order effects in slender structural components.
The prediction of such complex behaviour is always a difficult task and requires to dispose of instruments able to describe the mechanical non-linear behaviour of the involved materials (i.e. steel of bar reinforcements and concrete matrix).
The non-linear finite element method (NLFEM) is more and more used in common practice in order to reproduce the actual response of reinforced concrete structures in all the relevant design loading conditions.
The following question arises: how accurately the NLFE analysis (NLFEA) is able to represent the actual structural behaviour? This topic is of great concern for engineering practice, because the safety verification depends on the assumptions underlying the NLFEM model developed by the engineer.
The NLFEA can be powerfully used in the solution of a wide field of problems in structural engineering. As for example, NLFEA can be used when seismic assessment have to be performed in new and existing structures accounting for local effects and infills-concrete frame interaction [Di Trapani *et al.*, 2017].

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The NLFEA can be employed in order to define reliability based criteria for seismic assessment of r.c. structures also in the case of presence of base isolation [Castaldo *et al.*, 2017, Castaldo *et al.*, 2017], deterioration [Castaldo *et al.*, 2017] and settlements due to external causes [Castaldo and De Iuliis, 2014].
In the scientific literature, several guidelines for calibration of NLFEM models have been proposed [Belletti *et al.*, 2015, *fib* Bulletin N°45, 2008]. However, the results from NLFEA should be processed in order to satisfy the expected reliability levels [ISO 2394, 1998]. The design resistance of a structure can be assessed, according to [CEB-FIP Model Code, 1990, EN 1992-1-1, 2004 and *fib* Model Code, 2010], by adopting a safety format. Each safety format allow to perform the safety verification at ultimate limit state (ULS) of the structure applying the following inequality:

$$F_d \leq R_d \tag{1}$$

Where F_d = design agent action; and R_d = design strength of the structure/structural member estimated by means of NLFEA after the application of a safety format.
In order to employ NLFEA for structural safety verification, particularly in presence of seismic excitation, the commercial NLFEM codes should be accurately tested in their capability to reproduce the actual behaviour of reinforced concrete structures from serviceability to ultimate conditions. Generally, commercial non-linear finite elements software provide better results when used to reproduce experimental tests with monotonically increasing loads, rather than with cyclic loading.
In the literature [Bertagnoli *et al.*, 2015] proposed the comparison between three commercial finite element software named A, B and C. In their investigation they tested the abovementioned software in reproducing the outcomes of laboratory tests on different structures under monotonically increased loads by adopting plane stress models (i.e. 2D NLFEA).
In this work, Software B and C have been selected in order to reproduce the experimental outcomes of the laboratory tests performed by [Pilakoutas and Elnashai, 1995, Lefas and Kotsovos, 1990] on six shear walls by means of NLFEM plane stress models. These shear walls have been subjected by the original Authors to repeated loading process and tested up to failure.
Successively, the model uncertainties ϑ expressed as the ratio between experimental and NLFEM simulation results have been estimated according to [JCSS, 2001]. The uncertainties related both to prediction of maximum load ϑ_L and maximum displacement ϑ_D reached during the laboratory tests have been evaluated and discussed.

2 Resisting model uncertainties

The behavior of reinforced concrete structures can be reproduced performing choices concerning physical - mechanical parameters of materials and geometrical configuration.
In general, these variable are completely known and assessed, but the definition of the structural resistance have to be determined by adopting a resisting model.
Each model (i.e. reality simplification) allows to perform prediction that can differ from the actual response of a structure of a certain amount. This is due to uncertainties related to

simplified assumptions and in disregard some parameters that can influence the actual resisting mechanism.
In the case of structural safety verification by adopting a specific resisting model, the identification and quantification of the related model uncertainties is a relevant task.
The resisting model uncertainties may be estimated by means of multiplicative relationship [JCSS, 2001]. Defining ϑ as the model uncertainty due to factors affecting test and model results, the following expression may be written:

$$R(X,Y) \approx \theta \cdot R_{\mathrm{mod}}(X) \tag{2}$$

where R(X,Y) is the actual resistance estimated from laboratory tests in this case; $R_{mod}(X)$ is the response (or the resistance) estimated by the resisting model (in this case NLFEM plane stress model); X is a vector of basic variables included into the resistance model; Y is a vector of variables that may affect the resisting mechanism, but are neglected in the model.
In the case of NLFEA, the quantification of resisting model uncertainties ϑ may be performed reproducing numerically the outcomes of experimental tests and defining the following ratio:

$$\theta_i = \frac{R_i(X,Y)}{R_{\mathrm{mod,i}}(X)} \tag{3}$$

where ϑ_i represents the i-th outcome of the model uncertainty estimated comparing experimental tests and numerical simulations.
In the following, the case study related to the shear walls tested under repeated loading process is reported.

3 Case studies

3.1 Lefas and Kotsovos shear walls

The experimental tests realized by [Lefas and Kotsovos, 1990] involved shear walls having 650 mm wide, 1300 mm high and 65 mm thick. The walls named SW31, SW32 and SW33 have been analysed in this work.
The specimens have been tested as isolated cantilever and monolithically connected to an upper and a lower beam. Figure 1(d) shows the nominal dimensions of the walls and the arrangement of the reinforcements.
In Tables 1 and Table 2 are summarized the main properties of the material used for each wall, where: f_{cm} is the concrete mean cylinder compressive strength, ε_{c1} is the strain at peak stress, ε_{cu1} is the ultimate strain in uniaxial loading, whereas f_y is steel yielding stress, ε_y is yielding strain, and f_u is steel failure stress and ε_u is ultimate strain.
Figures 1(a-c) shows the three different horizontal repeated loading histories adopted respectively for the specimens SW31, SW32 and SW33. After the cyclic phase all specimens have been incrementally loaded up to failure.

Table 1- Concrete properties for [Lefas and Kotsovos, 1990] shear walls

Specimen	f_{cm} (MPa)	ε_{c1} (‰)	ε_{cu1} (‰)
SW31	29.2	2.15	3.50
SW32	44.5	2.39	3.50
SW33	40.8	2.34	3.50

Table 2-Reinforcement properties for [Lefas and Kotsovos, 1990] shear walls

Diameter (mm)	f_y (MPa)	f_u (MPa)	ε_y (%)	ε_u (%)
4	420	490	0.21	7.50
6.25	520	610	0.26	7.50
8	470	565	0.24	7.50

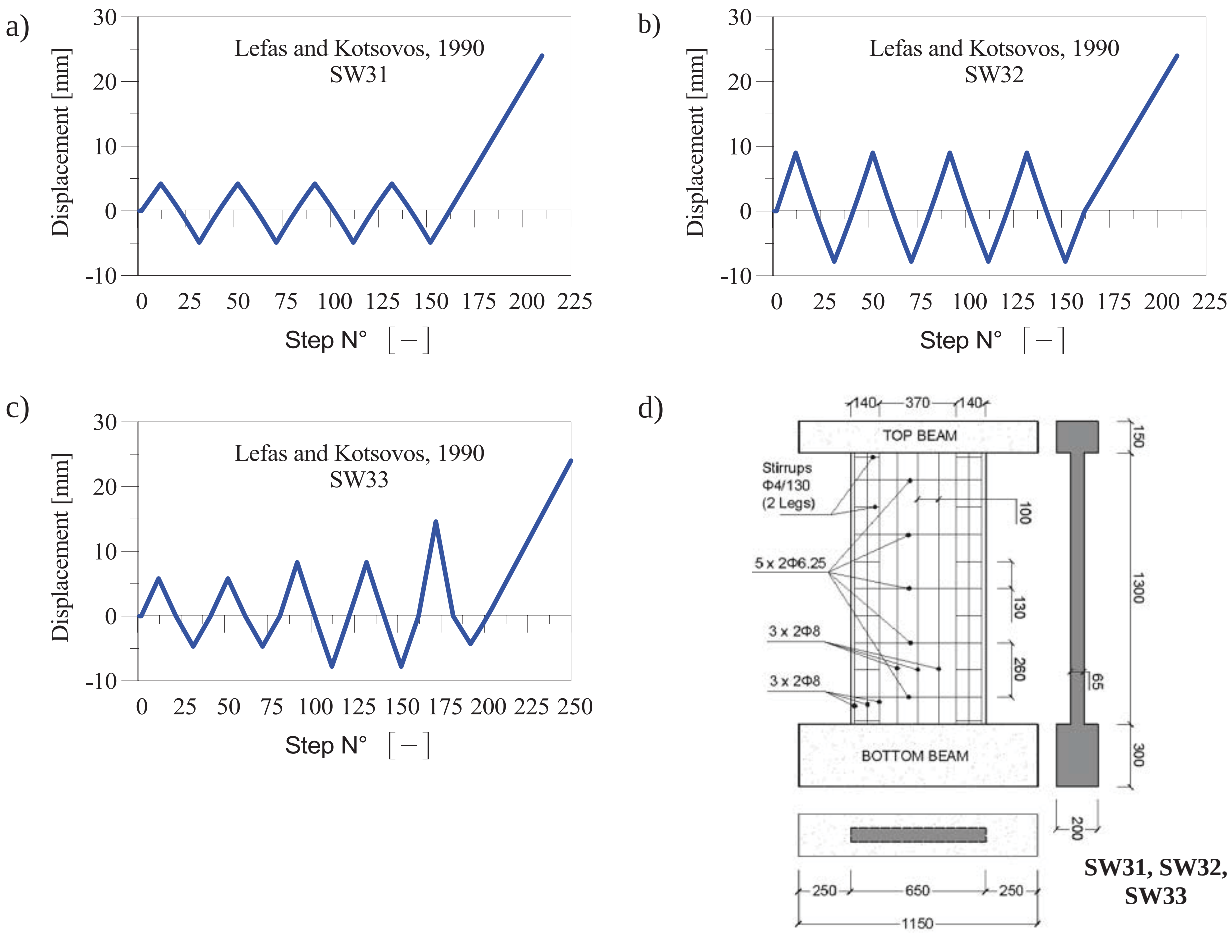

Figure 1-Loading history for SW31 (a), SW32 (b) and SW33 (c);geometry and reinforcement details of walls SW31, SW32 and SW33 (d) [Lefas and Kotsovos, 1990].

3.2 Pilakoutas and Elnashai shear walls

In the laboratory experiments performed by [Pilakoutas and Elnashai, 1995] the specimens was realized by means of 600 mm wide, 1200 mm high and 60 mm thick reinforced concrete walls. The walls named SW4, SW6 and SW8 have been considered in the present investigation.
The external horizontal load have been applied to the top beam using a jack. The force coming from the jack have been applied to the top beam, which was purposely designed in order diffuse the load uniformly into the wall.

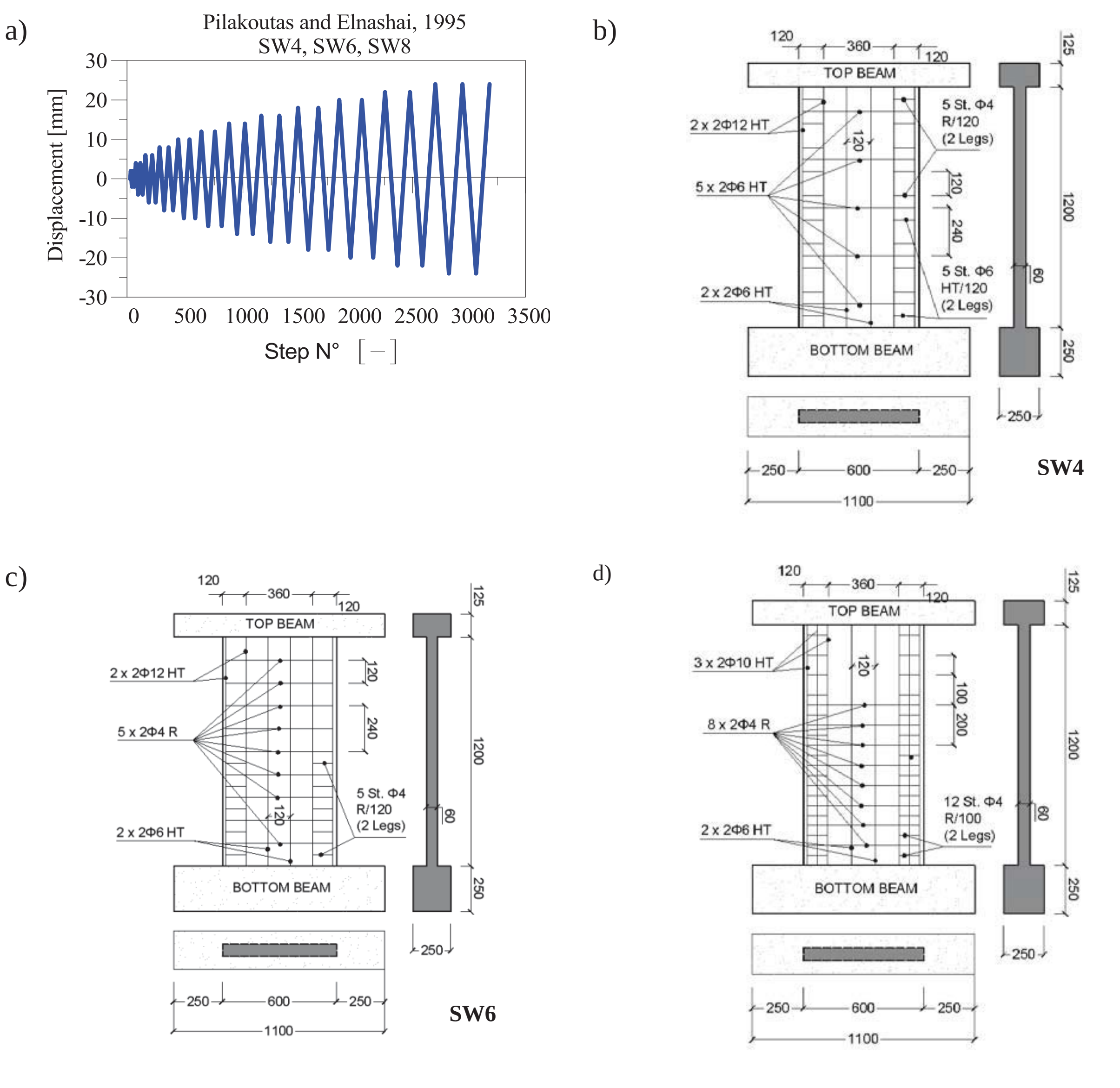

Figure 2-Loading history for SW4, SW6, SW8 (A);geometry and reinforcement details of walls SW4 (b), SW8 (c) and SW8 (d) [Pilakoutas and Elnashai, 1995].

Table 3-Concrete properties

Specimen	f_{cm} (MPa)	ε_{c1} (‰)	ε_{cu1} (‰)
SW4	36.9	2.28	3.50
SW6	38.6	2.30	3.50
SW8	45.8	2.41	3.50

Table 4-Reinforcement properties

Diameter (mm)	f_y (MPa)	f_u (MPa)	ε_y (%)	ε_u (%)
4	400	460	0.20	6.00
6	545	590	0.27	2.00
10	530	660	0.27	4.20
12	500	660	0.25	8.50

The shear walls was fully restrained at the base by anchoring the lower beam to the rigid floor of the laboratory. In Figure 2(b-d) are showed the nominal dimensions of the walls and the arrangement of the reinforcements.
In Table 3 and Table 4 are summarized the main properties of the material used for each wall as reported by the original paper, with: f_{cm} concrete mean cylinder compressive strength; ε_{c1} concrete strain at peak stress; ε_{cu1} concrete ultimate strain in uniaxial loading; f_y steel yielding stress; ε_y yielding strain; f_u steel failure stress; ε_u steel ultimate strain.
Figure 2(a) shows the loading history adopted for SW4, SW6 and SW8. A single load level is composed by two full cycles with the same maximum top displacement. When one load level is completed the top displacement was incremented of 2 mm in both directions. All specimens have been tested up to failure.

4 Non-linear FEM simulations

The outcomes of laboratory tests described in Section 3 have been reproduced by means of two internationally well-known commercial programs for non-linear analysis of structures. The comparison of the software in terms of accuracy is not within the scope of this investigation, therefore, they will be named as Software B and C according to [Bertagnoli *et al.*, 2015]. Firstly, the solution strategy selected for the definition of the NLFEM models is described in terms of choice of finite elements formulation and material models. Finally, the outcomes of the NLFEAs are reported and commented.

4.1 Finite elements formulation and material models

The 2D NLFEM models have been realized with quadrilateral iso-parametric plane stress elements in order to represent the concrete matrix. Linear interpolation as shown in Equation (3) and 2x2 Gauss integration scheme is used.

$$u(\xi,\eta) = a_0 + a_1\xi + a_2\eta + a_3\xi\eta \tag{4}$$

The mesh size have been defined after a calibration procedure and at the end the average element dimensions have been selected close to 10 cm. This allow to consider concrete as a homogeneous material. The non-linear system of equation is solved by means of the Newton-Raphson's iterative procedure [*fib* Bulletin N°45, 2008] both in the case of Software B and C. The mono-axial constitutive model of concrete in compression used in Software B is in according to [CEB-FIP Model Code, 1990], whereas the model reported by [EN 1992-1-1, 2004] have been selected in the case of Software C. Both software allows to account for the reduction of concrete compressive strength due to cracks development in the orthogonal direction.

The two-dimensional failure criteria defined by [Kupfer and Gerstle, 1973] is adopted both for Sotware B and Software C in order to model the plane-stress failure mode of concrete matrix.

All the material parameters have been defined in compliance with the laboratory experiments as reported in Table 1 and Table 3.

The smeared fixed crack direction model [De borst and Nauta, 1985, Riggs and Powell, 1986 and Rots, 1988] is adopted in all the software: the crack direction is defined at first cracking and does not change during the following load steps. Shear stresses can be present on the crack surface by means of reduction of shear stiffness after cracking (shear retention factor β=0.15).

In Software B and C, the concrete tensile behaviour have been modelled with an elastic with post-peak linear tension softening law. The latter presents the ultimate strain of the softening branch equal to $10 \cdot \varepsilon_l$. Where ε_l is the strain corresponding to the peak concrete tensile strength at the end of the elastic field.

Both in the case of Software B and C reinforcement are modelled by means of discrete truss elements. The steel mechanical behaviour is modelled as a Von Mises plastic material with a bi-linear stress-strain law defined with properties according to Table 2 and Table 4.

The progressive damaging of concrete matrix and bar reinforcements have been reproduced adopting the damaging models implemented by the Software B and C based on energy criteria.

The two-dimensional plane stress models of reinforced concrete members allow to take into account the effect of in-plane confinement due to reinforcement bars. However, in order to reproduce the actual behaviour of the structure under repeated loading, the out-of-plane confinement contribute may become relevant in the case of presence of closed stirrups. This is the case of the external chords of the shear walls analysed in the present paper and described in Section 3.

In order to account for the effect of out-of-plane confinement inside the 2D NLFEM models of the shear walls, the model proposed by [EN 1992-1-1, 2004] have been adopted.

The out-of-plane confinement can be modelled modifying the constitutive law of the concrete matrix. This modification consist into an increase of the peak strength (with the corresponding strain) and of the ultimate strain in compression. Such increase may be calculated proportionally to the amount of the out-of-plane shear reinforcement and to the effective transverse confinement stress. Example of application of this model have been proposed in the literature by [Bertagnoli *et al.*, 2011].

4.2 Results from NLFEA

The outcomes of the NLFEA assessed with Software B and Software C trying to reproduce the shear walls structural behaviour under cyclic loading are reported in Figures 3(a-i) and Figure 4(a-i). The load-displacement response under repeated loading obtained with Software B and C is compared with the experimental results for each one of the six shear walls considered in this work. The comparison between experimental and NLFEA results is done in terms of horizontal force (reaction) corresponding to the imposed displacement to the top beam. This is in according with the measurement performed during the laboratory tests, where the horizontal displacement of the top beam have been monitored in function of the applied force into the jack.

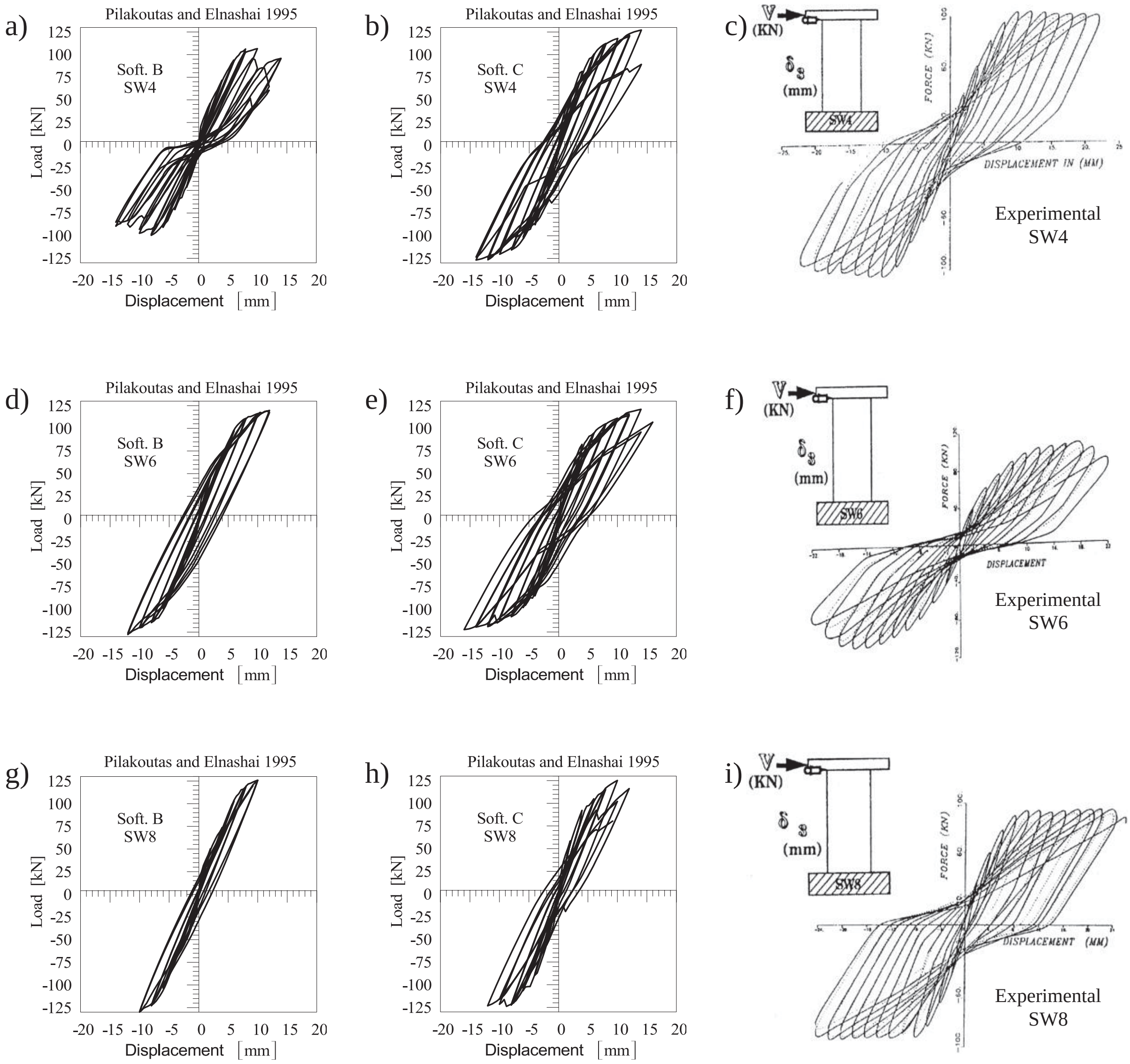

Figure 3-Comparison between Software B and C results and experimental outcomes [Pilakoutas and Elnashai, 1995].

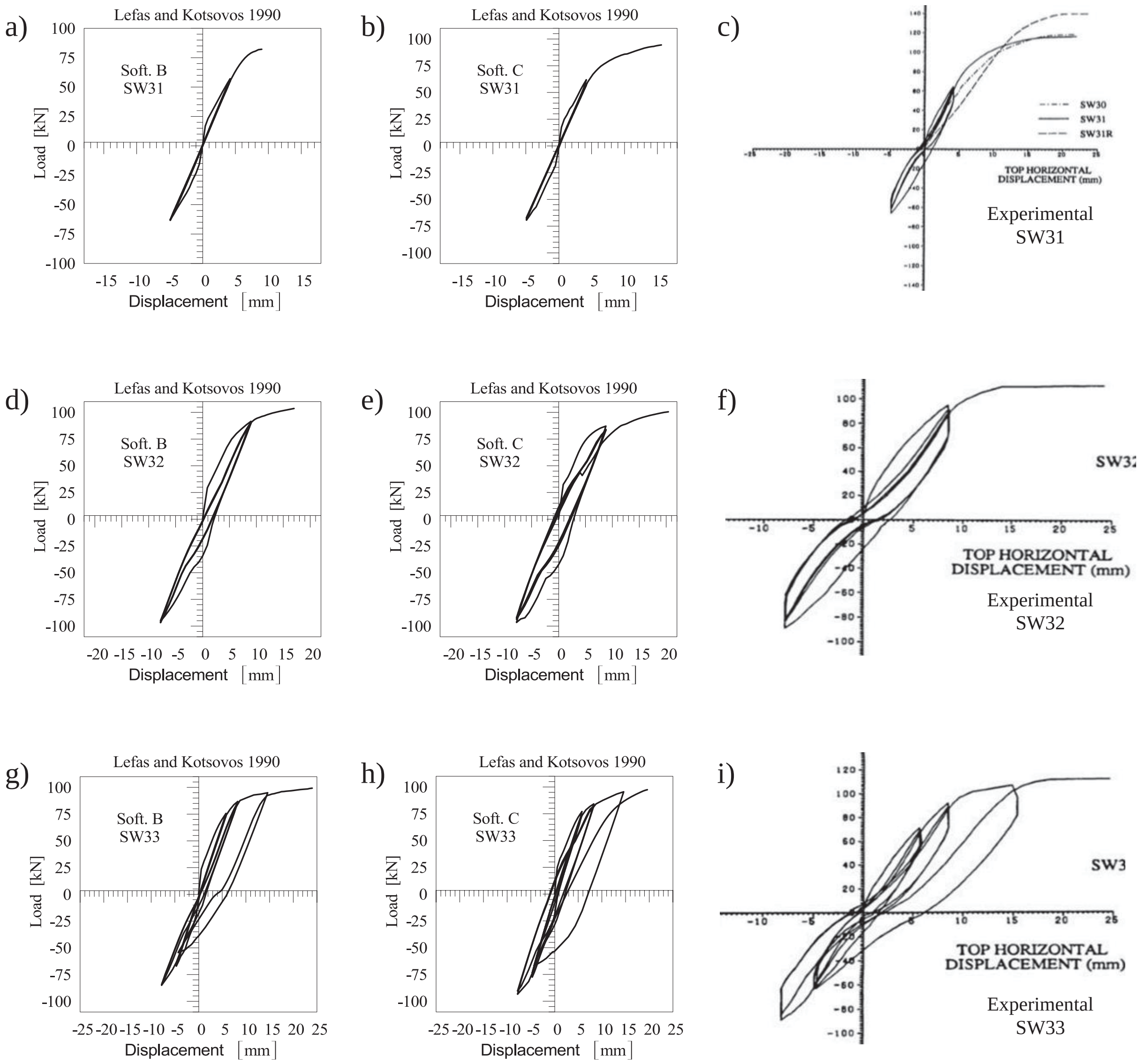

Figure 4-Comparison between Software B and C results and experimental outcomes [Lefas and Kotsovos, 1990].

The results obtained from the simulations of the experimental tests on walls SW31, SW32 and SW33 shows to be more accurate and faithful with the experimental outcome if compared with results obtained for walls SW4, SW8, SW10. In fact, the latter was not able to achieve the completion of the loading process proposed by experimental test due to lack of convergence of the solution. This difference in the goodness of the NLFEM simulation it is possibly due to the different loading process to which were subjected the two families of walls.

A relevant parameter to evaluate performance of the numerical analyses result to be the ratio between the top horizontal displacement and the height of the wall, which can be called "shear deformation".

In fact, it represents the global average level of angular distortion γ that occur into the wall during the test. Solutions proposed by Software B and C are satisfactory until the parameter γ reaches a value around $8 \cdot 10^{-3}$, which, being all the walls height almost equal, corresponds to a top horizontal displacement of about 10 mm.
Such value of γ implies a principal deformation ε_2 magnitude which is close to the deformation at failure of concrete (i.e. $\approx 3.5 \cdot 10^{-3}$).
When γ level of angular distortion is passed, both software are unable to reproduce accurately the actual structural behaviour.
For high displacement levels, the phenomena connected to confinement effect due to both reinforcements and two-dimensional behaviour become relevant and material non-linearity are particularly uncertain.

5 Quantification of the resisting model uncertainties

The uncertainties related to the realization of the numerical models of the shear walls can be quantified, according to Equation (3), through the definition of the ratio between experimental and NLFEM results.
In this paper, the resisting model uncertainties ϑ_L and ϑ_D have been estimated respectively in the case of prediction of the maximum top horizontal load F_{max} and of the maximum top horizontal displacement d_{max} reached during the cyclic loading process. Then, model uncertainties can be evaluated with the following expressions:

$$\theta_L = \frac{F_{\max,EXP}}{F_{\max,FEM}} \tag{5}$$

$$\theta_D = \frac{d_{\max,EXP}}{d_{\max,FEM}} \tag{6}$$

Where, $F/d_{max,EXP}$ and $F/d_{max,FEM}$ are the maximum horizontal load/displacement coming respectively from experimental tests and NLFEA.
The sample representing the outcome for the model uncertainties predicting the maximum load and the maximum displacement is reported in Table 5 both in the case of Software B and Software C. A graphical representation of the variation of the model uncertainties for the six shear walls considered in this work is reported in Figure 5.

Table 5-Model uncertainties estimated for Soft. B and Soft. C

Specimen	ϑ_L (-)			ϑ_D (-)		
	Soft. B	-	*Soft. C*	*Soft. B*	-	*Soft. C*
SW4	0.96	-	0.81	1.71	-	1.50
SW6	0.85	-	0.88	2.20	-	2.00
SW8	0.74	-	0.77	2.60	-	2.26
SW31	1.40	-	1.25	2.47	-	1.39
SW32	1.06	-	1.17	1.63	-	1.44
SW33	1.14	-	1.14	1.25	-	1.35

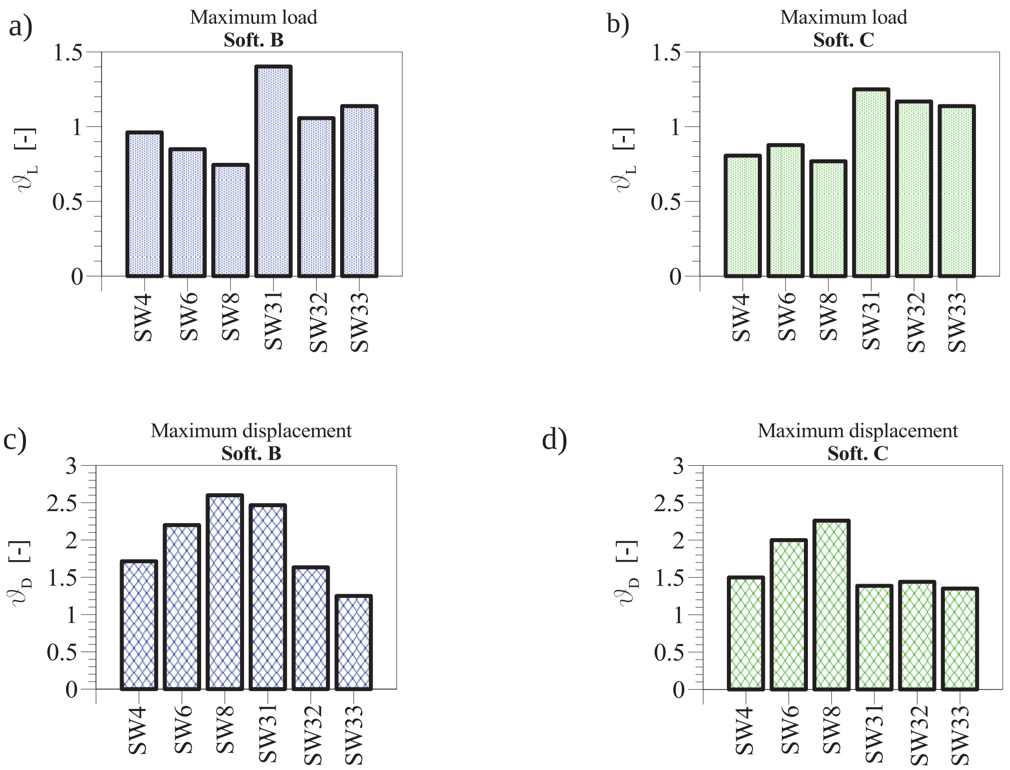

Figure 5-Model uncertainties predicting the maximum load reached during the loading process (a-b);model uncertainties predicting the maximum horizontal displacement reached during the loading process (c-d)

Software B and Software C shows a substantial equivalence in the simulation of the structural behaviour as they presents approximatively the same level of uncertainty both in prediction of the maximum load and maximum displacement during the cycles.

The mean value μ_ϑ, standard deviation σ_ϑ and coefficient of variation $V_\vartheta = \sigma_\vartheta / \mu_\vartheta$ for the resisting model uncertainties related to Software B, Software C and all the results have been evaluated and reported in Table 6.

Table 6-Mean value, standard deviation and coefficient of variation for model uncertainties ϑ_L and ϑ_D (Software B, Software C and all results)

Model uncertainty ϑ	Software B			Software C			All results		
	μ_ϑ (-)	σ_ϑ (-)	V_ϑ (-)	μ_ϑ (-)	σ_ϑ (-)	V_ϑ (-)	μ_ϑ (-)	σ_ϑ (-)	V_ϑ (-)
ϑ_L	1.04	0.23	0.22	1.00	0.19	0.19	1.01	0.20	0.20
ϑ_D	1.98	0.48	0.24	1.66	0.35	0.21	1.82	0.45	0.25

The results concerning ϑ_L presents a mean value close to 1 for both software (i.e. unbiased results). A slightly higher dispersion is shown by Software B with a COV of 0.22 rather than the value of 0.19 obtained in the case of Software C.
The model uncertainty ϑ_D presents a mean value equal to 1.98 for Software B and 1.82 for Software C. In the average the model predictions underestimates of about 80% the maximum displacement reached during the loading process in laboratory experiments (safe bias). The COV equal to 0.24 have been obtained for Software B and equal to 0.21 for Software C.
In general, Software B show slightly higher mean values (safe bias) and COV if compared with the results obtained with Software C.
The overall coefficients of variation of 0.20 and 0.25 respectively for ϑ_L and ϑ_D are in agreement with what is suggested by [JCSS, 2001] for model uncertainties identification.

6 Conclusions

In the present paper have been proposed the investigation related to model uncertainties in non-linear analysis of reinforced concrete shear walls subjected to horizontal cyclic loading. Two different commercial finite element software have been selected in order to simulate the structural behaviour of two families of shear walls tested by different authors with results reported in the scientific literature.
Appreciable accuracy and small scattering of the results have been achieved in the case of walls SW31, SW32 and SW33. On the contrary tests concerning SW4, SW6 and SW8 showed to be difficult to be reproduced numerically due to the high level of maximum displacements reached during the experiments. This results can be correlated to the level of shear distortion reached during the test. If fact, for shear deformation γ up to $8\cdot10^{-3}$ both software are able to reproduce the actual structural behaviour, whereas for higher distortion levels a loss of accuracy in the results is observed.
In general, the two different commercial software (i.e. Software B and Software C) are equivalent in their capability to predict the structural response under cyclic loading of the shear walls analysed in this work.
The model uncertainties ϑ_D and ϑ_L may be estimated respectively concerning the prediction of the maximum displacement and maximum load reached during the experimental test.
Considering the results related to both software, ϑ_L have mean value equal to 1.01 (i.e. unbiased) and coefficient of variation 0.20, whereas ϑ_D present mean value equal to 1.82 (i.e. safe bias) and coefficient of variation equal to 0.25.
In conclusion, a comprehensive quantification of the model uncertainties related to NLFEA of concrete structure under repeated loading requires a more detailed parametric analysis. In fact, with this aim it is necessary a further differentiation between the possible solution strategies (i.e. materials constitutive laws, finite element formulation, convergence criteria) available for the engineers in practice.
The research will be enhanced consider the influence of more FE model parameters on 2D NLFEM model uncertainties and on the prediction of the structural response of R.C. structures subjected to cyclic loading.

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VALUTAZIONE DELLE INCERTEZZE DI MODELLO IN ANALISI FEM NON LINEARI DI PARETI A TAGLIO IN CEMENTO ARMATO SOGGETTE AD AZIONI CICLICHE

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SUMMARY: *Il metodo degli elementi finiti non lineari consente di valutare il comportamento complesso di elementi in cemento armato sia in condizioni di servizio che ultime. L'analisi agli elementi finiti non lineari può essere impiegata con successo per risolvere un ampio campo di problemi nell'ingegneria strutturale. In questo contesto, al fine di adottare software che implementano analisi agli elementi finiti non lineari per la verifica della sicurezza strutturale, il livello delle incertezze presentate da tale tipo di analisi deve essere fortemente quantificato.*
Possono essere ottenuti risultati sensibili che simulano la risposta degli elementi strutturali sottoposti ad un progressivo processo di carico incrementale. Tuttavia, come nel caso della valutazione sismica di strutture in cemento armato, gli stessi risultati non possono essere raggiunti nel caso di azioni cicliche.
In questo lavoro, sono stati considerati due codici commerciali di calcolo agli elementi finiti non lineari per valutare le incertezze nella simulazione del comportamento non lineare del calcestruzzo sotto azioni cicliche adottando modelli di tensionali piani. Sono state valutate le incertezze del modello resistente (inteso come rapporto tra i risultati sperimentali e quelli di analisi FEM) nella riproduzione dei risultati sperimentali relativi a sei prove di laboratorio su pareti a taglio noti dalla letteratura. La capacità complessiva dei due codici di prevedere il comportamento strutturale reale sotto carico ciclico è commentata e il valore medio, la varianza e il coefficiente di variazione delle incertezze di modello sono quantificate e discusse.

KEYWORDS: *Calcestruzzo armato, pareti a taglio, incertezze di modello, carico ciclico*

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