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ORIGINAL RESEARCH PAPER

Implementation and verification of a masonry panel model for nonlinear dynamic analysis of infilled RC frames

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Abstract The effect of infill panels on the response of RC frames subjected to seismic action is widely recognised and has been subject of numerous experimental investigations, while several attempts to model it analytically have been reported. In this work, the implementation, within a fibre-based Finite Elements program, of a double-strut nonlinear cyclic model for unreinforced masonry panels is carried out. The adequacy of the model in predicting the cyclic/seismic response of multi-storey infilled reinforced concrete frames is then verified through comparisons against experimental results.

Keywords Masonry panel model · Infilled RC frames · Dynamic behaviour · Nonlinear analysis

1 Introduction

Extensive research work has been carried out in order to predict the influence of infill panels as it is well known that their presence modifies significantly the structural behaviour of RC structures. Within the framework of performance-based assessment of existing structures, where an as correct as possible evaluation of a structure's performance at all response levels (i.e. from low to very high damage) is central, the accurate numerical modelling of the response of infill panels and their interaction with the surrounding reinforced concrete frames is therefore, of utmost importance.

Different modelling techniques have been used for the analysis of infilled frames, which can be divided into two main groups: (i) local or micro-models and (ii) simplified or macro-models. The plane finite elements and equivalent truss models are the typical examples of the first and second group, respectively. Micro-models can simulate the structural behaviour with great detail, provided that adequate constitutive models are used. However, they are computational intensive and difficult to apply in the analysis of large structures. Macro-models exhibit

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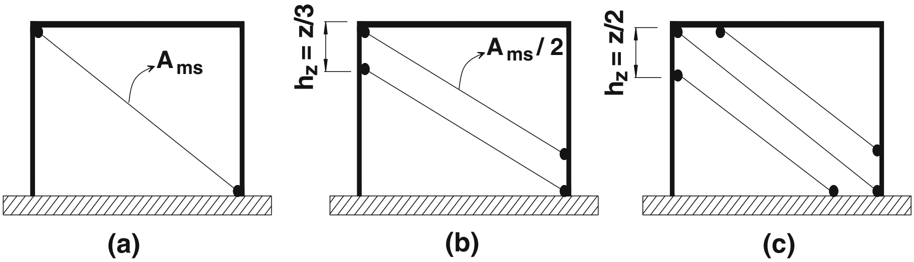

Fig. 1 Possible strut models (Crisafulli and Carr 2007). **a** Single-strut model. **b** Double-strut model. **c** Triple-strut model

obvious advantages in terms of computational simplicity and efficiency. Their formulation is based on a physically reasonable representation of the structural behaviour of the infilled frame. A detailed and thorough literature review of previous proposals on modelling of the response of infilled frames is beyond the scope of the present work, though readers may refer to other publications where such reviews have been carried out. Indicatively, the work of Crisafulli et al. (2000) is mentioned, where a detailed review of several tens of publications on this subject is described.

The present work aimed at the implementation in a given finite element code of one of the macro-models that have been proposed in the literature (Crisafulli 1997) and the subsequent verification of its adequacy for the prediction of the seismic response of multi-storey infilled reinforced concrete frames. The model, the characteristics of which are summarised in the sections hereinafter, is a multi-strut model, thus capable of somehow accounting for the local effects due to the presence of the panel in the surrounding frame without a significant increase in the complexity of the analysis.

2 Overview and implementation of the model

2.1 Element formulation

As already mentioned, the adopted model features the double-strut approach shown in Fig. 1b, somewhat a compromise between the simplified single-strut (Fig. 1a) and more elaborated triple-strut (Fig. 1c) modalities. As described in Crisafulli and Carr (2007), such double-strut approach is capable of providing a relatively good insight into the panel-frame interaction effects at a reasonable modelling and computational cost.

Each infill panel is thus represented by four axial struts and two shear springs, as shown in Fig. 2; each diagonal direction features two parallel struts to account for compression/tension forces and deformations across two opposite diagonal corners and a shear spring to account for bed-joint resistance and sliding. This latter spring, the presence of which reflects the importance of shear deformation/strength in the response of unreinforced masonry panels, acts solely across the diagonal that is in compression, hence its “activation” depends on the deformation of the panel.

As can be observed in Fig. 2, four internal nodes are employed to account for the actual points of contact between the frame and the infill panel (i.e. to account for the width and height of the columns and beams, respectively), whilst four dummy nodes (i.e. a second strut) are introduced with the objective of accounting for the contact length/width between the frame

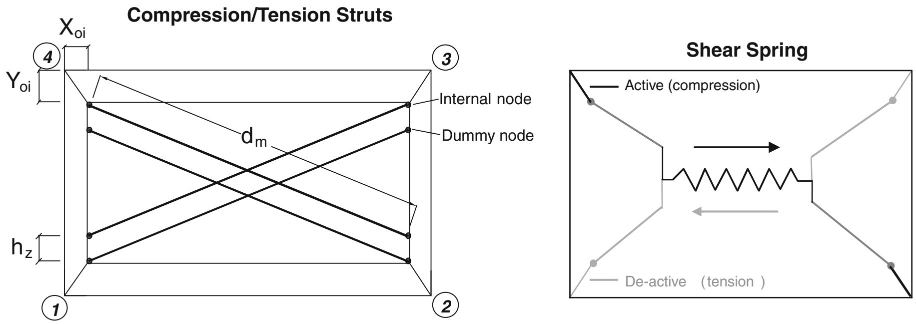

Fig. 2 Implemented infill panel model (Crisafulli and Carr 2007)

elements and the infill panel. In other words, the purpose of using the double strut approach is to consider the moments and shear forces that are normally introduced in the columns as a result of the eccentricity with which infill panels compress their adjacent frame members. The obtained displacements and forces in the dummy nodes are transferred to the adjacent internal nodes (see Fig. 2). The displacements perpendicular to the edges and the rotations are interpolated with cubic hermitian polynomials and the displacement in the direction of the edges is interpolated using a linear function. The transformation of the displacements and forces coming from the shear spring is simpler than for the struts, given the fact that only the step from internal to external nodes has to be carried out. It is important to note that the model does not have the capacity of modelling possible plastic hinges in the columns, even though it does consider the eccentricity of the struts. If the formation of plastic hinges in the length of the column is important, a different strut configuration should be implemented. All the internal forces are transformed to the exterior four nodes, where the infill panel element is connected to the frame. The interested reader is referred to Crisafulli and Carr (2007) for further numerical details on the transformation of the forces at the internal and dummy nodes, to the external forces at the four nodes where the infill panel element is connected to the frame.

In addition to the four corner nodes, the following parameters need to be assigned in order to fully characterise this element:

- hysteretic relationship for compression/tension struts, briefly discussed in subsequent sections
- hysteretic relationship for the shear struts, briefly discussed in subsequent sections
- infill panel thickness (t), which may be considered as equal to the width of the panel bricks alone
- strut area $1(A_1)$, defined as the product of the panel thickness and the equivalent width of the strut (b_w), normally varying between 10 and 40% of the diagonal of the infill panel (d_m), as concluded by many researchers based on experimental data and analytical results (see e.g. literature review by Crisafulli et al. 2000)
- strut area $2(A_2)$, introduced as percentage of A_1, and which aims at accounting for the fact that, due to cracking of the infill panel, the contact length between the frame and the infill decreases as the lateral and consequently the axial displacement increases, affecting thus the area of equivalent strut (see Crisafulli 1997)
- equivalent contact length (h_z), introduced as percentage of the vertical height of the panel, effectively yielding the distance between the internal and dummy nodes, and used so as to take into account the contact length z between the frame and the infill panel, as

defined by Stafford Smith (1966). For suggestions of values refer e.g. to literature review by Crisafulli et al. (2000)

- horizontal and vertical offsets (X_{oi} and Y_{oi}), introduced as percentage of the horizontal and vertical dimensions of the panel, and which obviously represent the reduction of the latter due to the depth of the frame members. In other words, these parameters provide the distance between the external corner nodes and the internal ones
- proportion of stiffness assigned to shear (γ_s), representing the proportion of the panel stiffness that should be assigned to the shear spring (values ranging between 0.5 and 0.75 are suggested by Crisafulli et al. 2000)

2.2 Cyclic compression/tension strut relationship

The compression/tension cyclic relationship proposed by Crisafulli (1997) builds upon a number of proposals/results published by previous researchers (e.g., Sargin et al. 1971; Bolong et al. 1980; Mander et al. 1988; Otter and Naaman 1989; Naraine and Sinha 1989; Stevens et al. 1991; Subramaniam and Sinha 1995; Xinrong 1995) and features, in addition to the compression envelope and its loading, unloading and reloading rules, the effects of small inner cycles, tension softening and tension stiffening. The model (Fig. 3) calls for the definition of six material parameters, starting from the initial Young modulus (E_m), a property that can be estimated by means of experiments or through the employment of a wide range of different expressions proposed by a number of researchers (e.g., Sahlin 1971; Sinha and Pedreschi 1983; Hendry 1990; San Bartolomé 1990; Paulay and Priestley 1992). The compressive strength ($f_{m\,\theta}$), referring to the diagonal capacity of the infill panel and characterising the capacity of the masonry in the direction of the principal stress (f_1), is also defined, as well as the tensile strength (f_t), which represents the tensile strength of the masonry or the bond-strength of the interface between frame and infill panel. With respect to the compressive strength, it is typically assumed to coincide with the diagonal that links two opposite corner nodes (at an angle with respect to the horizontal line defined by the beams) and again, experimental data or expressions found in the literature may be used (see e.g., Crisafulli 1997). As far as the tensile strength is concerned, its presence does offer generality in the model and it is often assumed as equal to zero, since it will have a relatively minor effect on the overall response. Finally, it is noted that the model does take into account the small cycles, as well as the local contact effects of cracked material on the hysteretic response.

Moreover, the strain at maximum stress (ε_m), an experimentally defined parameter that typically varies from 0.001 to 0.005 (see e.g., Crisafulli 1997), and the ultimate strain (ε_{ult}), often considered as equal to 20 ε_m, in the absence of experimental data (see e.g., Crisafulli 1997), are required in the model, in conjunction with the closing strain (ε_{cl}), which defines the strain after which the cracks partially close allowing compression stresses to develop and its value typically ranges from 0 to 0.003, as Crisafulli (1997) suggests. Finally, the strut area reduction strain (ε_1) and the residual strut area strain (ε_2), associated to the strut area reduction mentioned above, need to be specified in the model. For these two parameters, between which it is assumed that the area varies linearly as function of the axial strain, it is obviously difficult to find supporting experimental evidence, given the empirical nature of the strut area reduction scheme. Reasonable values for ε_1 may be in the range of 0.0003–0.0008, whilst for $\varepsilon_2$values between 0.0006 and 0.016 may be considered (e.g., Smyrou 2006).

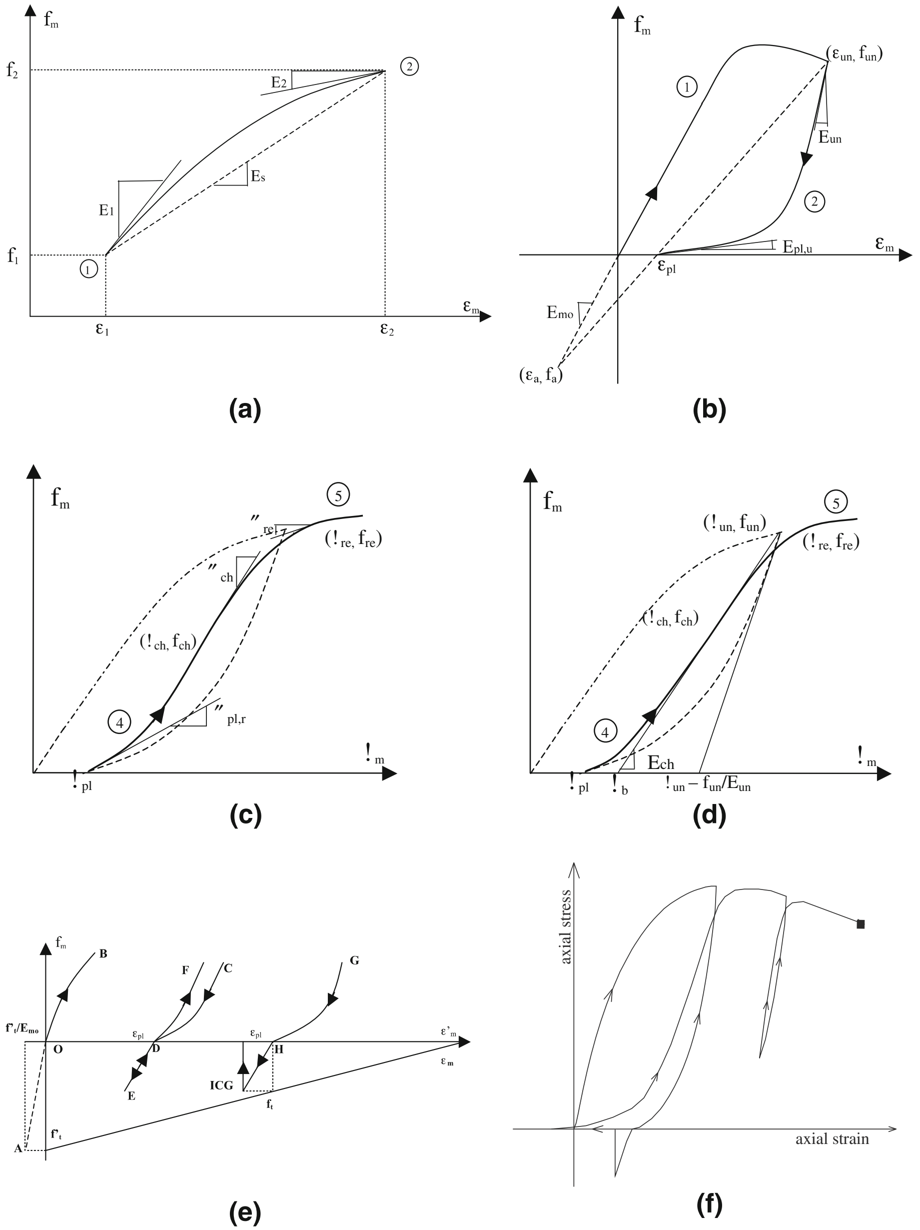

Fig. 3 Compression/tension cyclic relationship proposed by Crisafulli (1997). **a** Unloading and reloading. **b** Stress–strain curves for unloading. **c** Parameters associated with reloading. **d** Definition of change point for unloading. **e** Rule for tensile behaviour. **f** Masonry strut hysteretic response

In addition to these material mechanics parameters, a set of nine empirical factors associated exclusively to the cyclic rules need also to be defined in the model. A short explanation about their meaning is given below, while their values are given in Table 1.

Table 1 Empirical parameters

	Suggested values	Limit values	Used value
γ_{un}	1.5–2.5	≥1	1.7
α_{re}	0.2–0.4	≥0	0.2
α_{ch}	0.3–0.6	0.1–0.7	0.7
β_{a}	1.5–2.0	≥0	2.0
β_{ch}	0.6–0.7	0.5–0.9	0.9
γ_{plu}	0.5–0.7	0–1.0	1.0
γ_{plr}	1.1–1.5	≥1	1.1
e_{x1}	1.5–2.0	≥0	3.0
e_{x2}	1.0–1.5	≥0	1.0

γ_{un}: it defines the unloading modulus in proportion to E_{mo} and modifies the internal cycles, not the envelope.
α_{re}: it predicts the strain at which the loop reaches the envelope after unloading.
α_{ch}: it predicts the strain at which the reloading curve has an inflexion point, controlling the loops' "fatness".
β_{a}: it defines the auxiliary point used to determine the plastic deformation after complete unloading.
β_{ch}: it defines the stress at which the reloading curve exhibits an inflection point.
γ_{plu}: it defines the modulus of the hysteretic curve at zero stress after complete unloading in proportion to E_{mo}.
γ_{plr}: it defines the modulus of the reloading curve after total unloading.
e_{x1}: it controls the influence of ε_{un} in the degradation stiffness.
e_{x2}: it increases the strain at which the envelope curve is reached after unloading and represents cumulative damage inside repeated cycles, important when there are repeated consecutive cycles inside same inner loops.

These empirical parameters are required for the definition and establishment of a rule in the most general form, and somehow come as a natural consequence of the complex behaviour of infill panels. However, the sensitivity study carried out by Smyrou et al. (2006) seems to show that only three of these parameters do play a significant role in the quantification of the energy dissipation capacity of the infill panel (i.e. the remaining parameters are not expected to have a major impact if changed from the default values proposed by Crisafulli 1997) and suggests feasible ranges for these three coefficients.

2.3 Cyclic shear relationship

Crisafulli (1997) proposes for the masonry panel that the shear strength is computed independently of the failure mechanism (shear friction failure, diagonal tension failure, compression failure) being developed in the infill panel. This is a typical pragmatic approach, combining two shear resistance mechanisms (bond strength and the friction resistance between the mortar joints and the bricks), which effectively means that the shear strength can be expressed as the sum of the initial shear bond strength τ_0 and the product of coefficient of friction μ with the absolute value of the normal compressive force in the direction perpendicular to the bed joints. The values of τ_0 and μ can either be evaluated by direct shear tests or obtained from design specifications. However, whilst the former may lead to an overestimation of the values (Wan and Yi 1986; Riddington and Ghazali 1988), the latter tend to be over-conservative, so

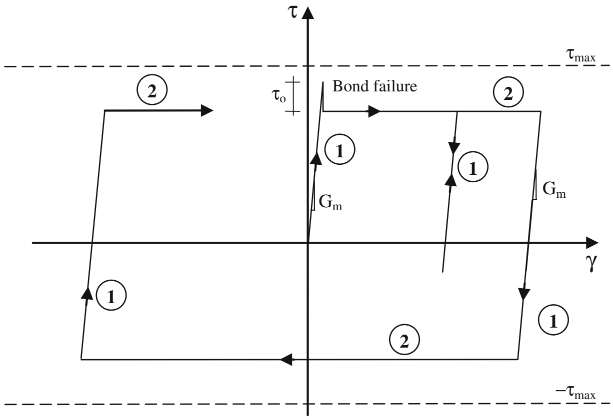

Fig. 4 Shear cyclic relationship proposed by Crisafulli (1997)

care should be taken in the approach adopted; Mann and Müller (1982), for instance, proposed expressions for reducing the usually overestimated values from shear tests. Four parameters explained hereinafter, i.e. the shear bond strength, the friction coefficient, maximum shear strength and reduction shear factor, need to be defined in order to fully characterise this response curve (Fig. 4).

The shear bond strength (τ_0) was experimentally measured by a very large number of researchers (see e.g., literature review by Smyrou 2006), leading to an equally ample range of possible values and/or several empirical expressions, all of which depend on different parameters and must be cautiously used considering the numerous variables that affect the shear bond strength. The friction coefficient (μ) is another parameter reported to vary between a range as large as 0.1–1.2, from a number of published works (e.g., Sahlin 1971; Stöckl and Hofmann 1988; Atkinson et al. 1989; Hendry 1990; Paulay and Priestley 1992). It is noted that some values proposed in literature are intended for design applications, rather than assessment, and hence tend to be conservatively low. The maximum shear strength (τ_{max}) is the largest shear stress that may be mobilised by the infill panel and, as stated above, depends on the failure mechanism (shear friction failure, diagonal tension failure, compression failure) developed in the latter. In the absence of additional and more precise information, one may pragmatically assume this value to be equal to the sum of τ_0 with the product of μ by the normal compressive strength of the masonry units. Finally, the reduction shear factor (α_s), an empirical parameter aimed at representing the ratio between the maximum shear stress and the average stress in the panel, may range between 1.4 and 1.65 (Crisafulli 1997). A value of 1.5 is typically assumed.

It is clarified, though, that the hysteretic response may differ from the one depicted in Fig. 4, in which the normal stress is assumed to remain constant, while in reality the normal stress is subjected to changes as the panel deforms in shear.

2.4 Modelling of openings

The presence of openings in infill panels constitutes an important uncertainty in the evaluation of the behaviour of infilled frames and much work has been devoted since 50's to investigate the influence that different configurations of openings in terms of size and location might have on strength and stiffness. Unfortunately, though in a way understandably given the large number of variables and uncertainties involved, agreement on this topic has not yet been reached and research has led to diverse quantitative conclusions and recommendations.

Engineering judgement and experience, coupled with a thorough consultation of the literature on this topic, must therefore, be used in order to decide on how the presence of openings in the structure being studied should be taken into account. As an expedite recommendation, one might perhaps suggest that the effect of openings on the response of an infilled frame can be pragmatically taken into account by reducing the value of the Strut Area (A_1), and hence of the panel's stiffness, in proportion to the area of the opening with respect to the panel. As shown by Smyrou (2006), if a given infill panel features openings of 15–30% with respect to the area of the panel, good response predictions might be obtained by reducing the value of A_1 (i.e. its stiffness) by a value that varies between 30 and 50%. As far as the strength of the infill panel is concerned, and given the extremely varied nature of the observations made on this issue by several researchers in the past, one could perhaps suggest that, in the absence of good evidence, no change in its value should be introduced to take into account the presence of standard openings (i.e. openings that are not larger than 30% of the area of the infill panel).

3 Verification of the infill panel model with test results

The infill panel model described above was implemented in SeismoStruct (SeismoSoft 2006), an internet-downloadable fibre-based Finite Elements package capable of predicting the large displacement behaviour of space frames under static or dynamic loading, considering both geometric nonlinearities and material inelasticity. The accuracy of the model is assessed through comparison with experimental results obtained from cyclic and pseudo-dynamic tests of large or full-scale frames, all of which featuring common geometrical and material characteristics, as well as loading conditions. Brief description of the tests, as well as presentation of the results of the numerical analyses, follows hereinafter, noting that preliminary verification of the accuracy in estimating the response of the bare frames for the majority of case studies was carried out (see Smyrou 2006), so as to ensure that the infill panel would remain as the only "verification variable". Finally, it is noted that the interested readers may reproduce the results by considering (i) the actual geometrical properties of the experimental case-study structures reported in the relevant references, (ii) the actual material properties reported by the executors of the tests (all corresponding references are provided) and (iii) employing the default values suggested in this manuscript for empirical parameters.

3.1 One-storey single-bay frames

Crisafulli (1997) carried pseudo-static tests in two single-bay infilled frames constructed to a reduced scale of 3/4, in order to validate his initially implemented model. Actuators were used to apply lateral and vertical forces, simulating thus the gravity loads and overturning moment corresponding to a typical two-storey building with infill panels (Fig. 5). Detailed information about the material properties and the test arrangements can be found in Crisafulli (1997). The results obtained with current implementation of the model, which was carried out independently and using a different software with respect to the original attempts, seem to be reasonably satisfactory (Fig. 6).

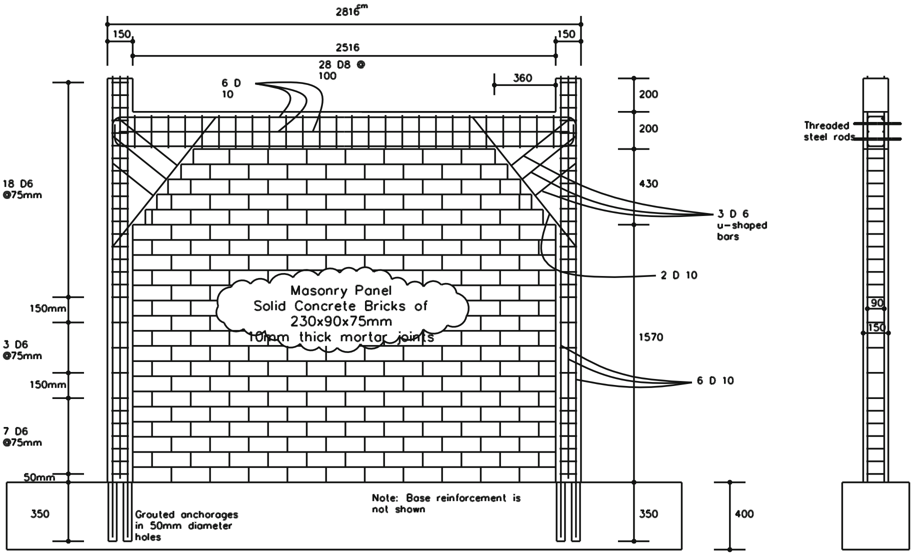

Fig. 5 Infilled frame tested by Crisafulli (1997)

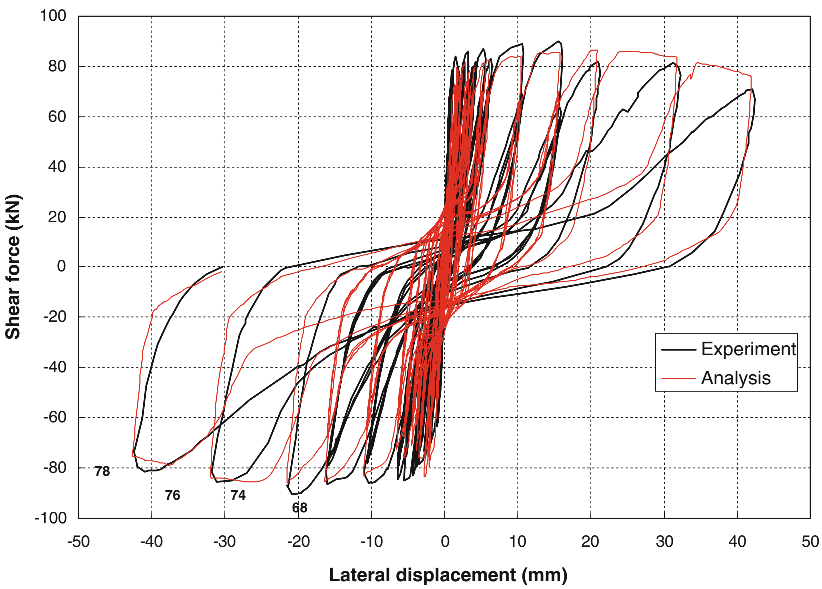

Fig. 6 Experimental and numerical response curves for frame tested by Crisafulli (1997)

Colangelo (1999) tested an infilled frame (Fig. 7) in reduced scale, built to comply with the requirements of an earlier version of Eurocode 8 (CEN 1995). The specimen was tested with a pseudo-dynamic load using the E-W component of the Friulli earthquake. Detailed description of the test-rig, the structural material properties, as well as of the loading regime, can be found in Biondi et al. (2000). Again, relatively accurate response estimations (in this

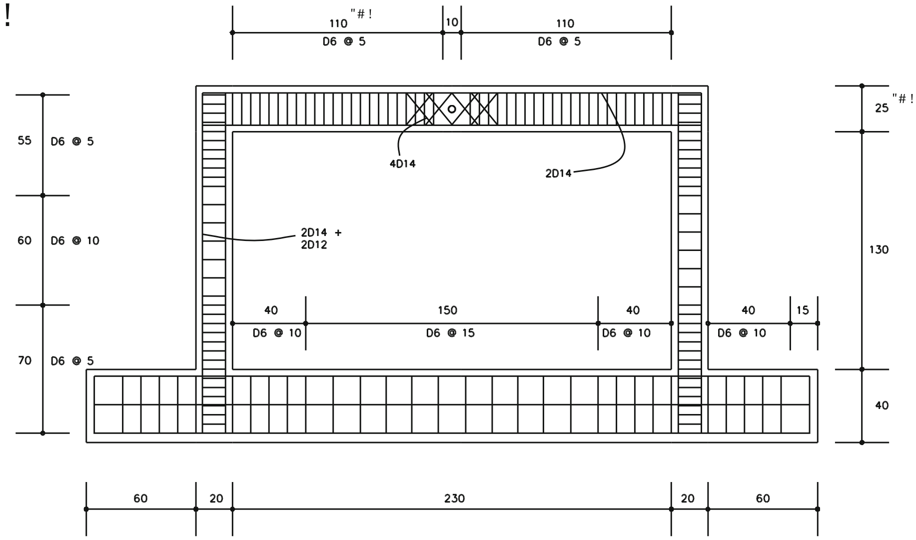

Fig. 7 Infilled frame tested by Colangelo (1999)

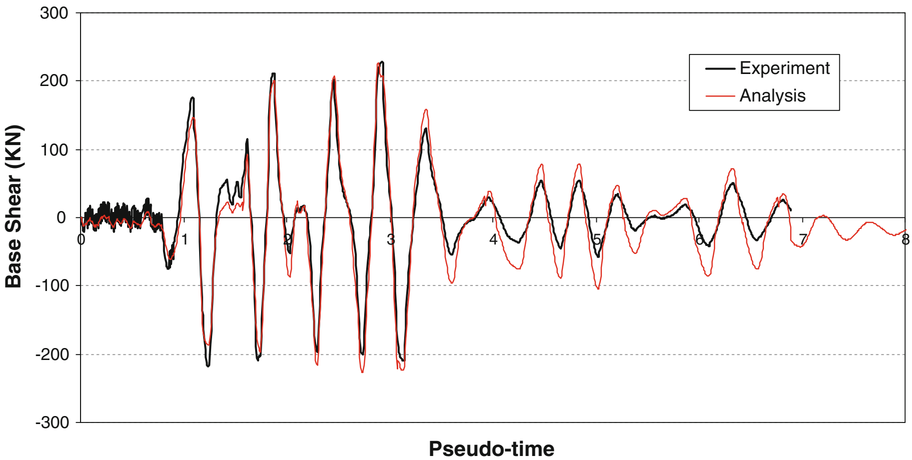

Fig. 8 Experimental and numerical base-shear histories for the frame tested by Colangelo (1999)

case of the shear mobilised in "time" by the frame, as it is subjected to a displacement history) seem to be obtained (Fig. 8).

3.2 Multi-storey 2D frame

The RC frame pseudo-dynamically tested at the ELSA laboratory (Joint Research Centre, Ispra) within the framework of the ICONS research programme (Carvalho and Coelho 2001) was also selected herein as case study. The full-scale, four-storey, three-bay, RC frame is representative of the design and construction practice of the 50–60 s in Southern Mediterranean countries, hence not meeting modern seismic design requirements. The frame included

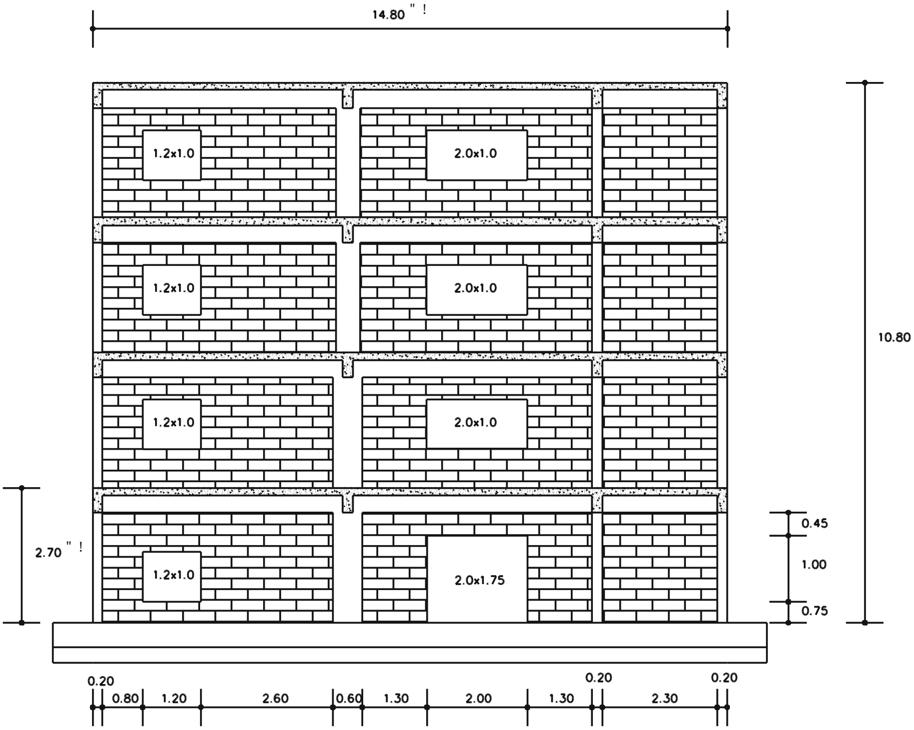

Fig. 9 ICONS frame (Carvalho et al. 1999)

infill panels with openings of different dimensions (Fig. 9). Further information on the ICONS frame, as well as on the tests conducted in ELSA, can be found in Pinto et al. (1999); Carvalho et al. (1999); Pinho and Elnashai (2000) and Varum (2003).

The infilled frame (Fig. 9) was tested under three records of increasing return period of 475, 975 and 2,000 years. The latter was interrupted since the frame experienced severe deformations approaching imminent collapse. As a first step in the model verification process, "pseudo-dynamic analyses" were carried out, thus imposing on the structure the same displacement histories that had been applied during the pseudo-dynamic tests. As expected, the agreement was very good (see Smyrou 2006). Since, pseudo-dynamic tests are, in principle, capable of simulating the actual dynamic response of framed structures, it is more meaningful, or at least more useful, to compare the experimental results with those stemming from dynamic analyses (note that a discussion on the degree to which pseudo-dynamic tests may indeed simulate the dynamic response of a structure is beyond the scope of the present work).

The structural model employed to run the preliminary pseudo-dynamic simulations was thus employed to run the dynamic analyses, after the introduction of the inertia mass, base accelerograms and a tangent stiffness-proportional viscous damping of 3% in order to somehow account for possible non-hysteretic damping sources. Since the hysteretic damping is directly considered through the nonlinearity in the material constitutive laws, a relatively lower value than the commonly adopted 5% is justified. The representative comparisons for the 975 years return period input motion are indicatively shown in Figs. 10 and 11 above indicating a somewhat satisfactory reproduction of the dynamic response of the infilled frame,

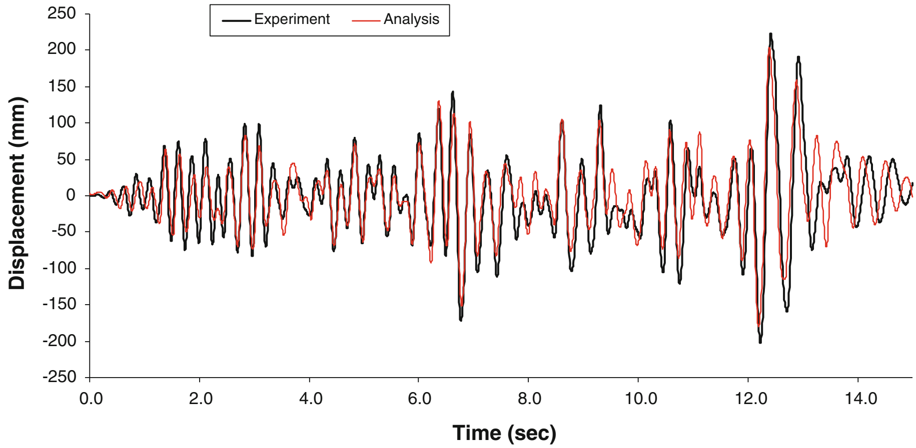

Fig. 10 Experimental and numerical roof displacement histories for the ICONS frame (975-yrp record)

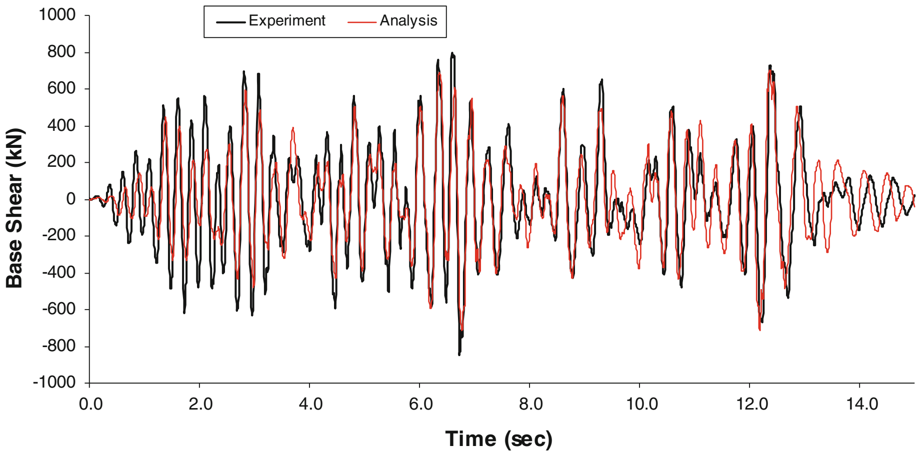

Fig. 11 Experimental and numerical base-shear histories for the ICONS frame (975-yrp record)

both in terms of frequency content and absolute values of displacements and base shear. Note that equally satisfactory results were acquired for the other records too.

It is noted that, as for all other case-studies considered, the steel, concrete and masonry material properties employed to define the numerical model corresponded to those obtained from tests carried out within the scope of the ICONS project, as described in Carvalho and Coelho (2001); Pinto et al. (1999) and Varum (2003). Where experimental calibration of some of the infill panel parameters was not possible, the default values proposed by Smyrou et al. (2006) were employed. The infill panel openings were taken into account by reducing the value of (A_1 (i.e. the compression/tension strut cross-section) by a value that ranged between 30 and 50%, as a function of the window and door size. It is certainly recognised that some fine-tuning and tweaking of the numerical model was carried out, in order to define such openings-driven reductions.

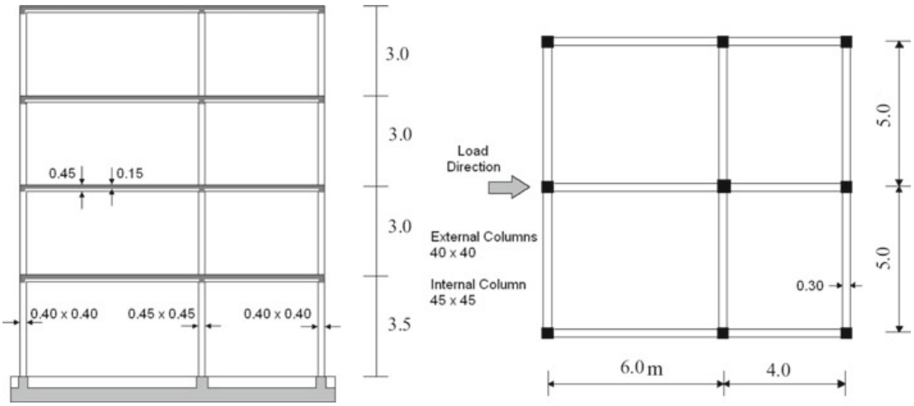

Fig. 12 Four-storey frame tested by Negro and Verzeletti (1996)

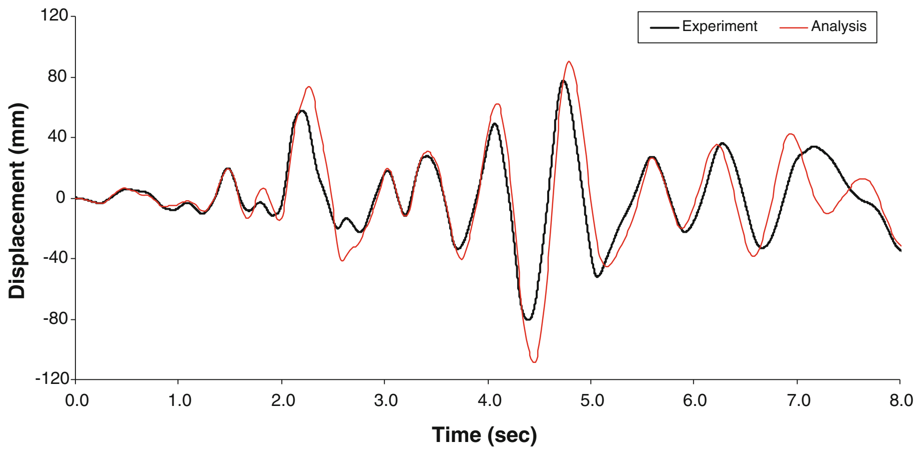

Fig. 13 Experimental and numerical roof displacement histories for the frame tested by Negro and Verzeletti (1996)

3.3 Multi-storey 3D frame

This final case-study consists of an infilled full-scale four-storey building (Fig. 12) designed according to initial versions of Eurocode 8 (CEN 1995) and Eurocode 2 (CEN 1991), tested at the ELSA laboratory (Joint Research Centre, Ispra). An artificial accelerogram derived from the 1976 Friuli earthquake was employed and the structure was tested pseudo-dynamically. Full details on the structure's geometrical and material characteristics may be found in Negro and Verzeletti (1996) and Negro (1997). As with the previous case, preliminary verifications were carried out by means of "pseudo-dynamic analyses", then followed by dynamic runs, some results of which are shown in Figs. 13 and 14, attesting once again the reasonable adequacy of the infill panel model.

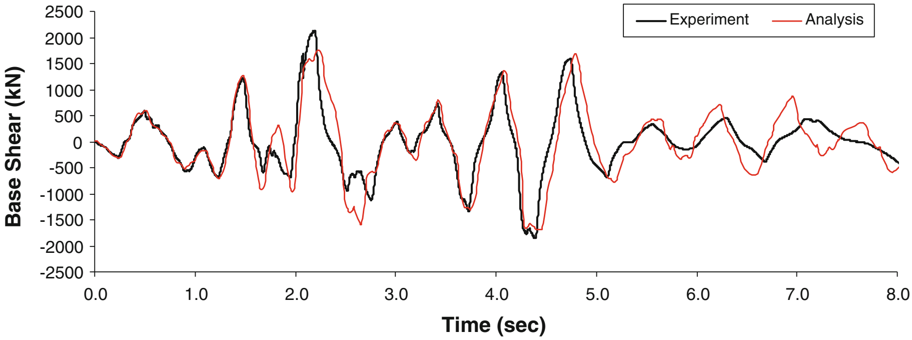

Fig. 14 Experimental and numerical base-shear histories for the frame tested by Negro and Verzeletti (1996)

4 Closing remarks

A double-strut nonlinear cyclic model for masonry panels was implemented in a fibre-based Finite Elements program and employed to simulate the response of different infilled frames. A relatively detailed description of the calculation/selection of the geometrical, mechanical and empirical parameters that are needed to characterise the infill panel element was given, with the view to assist readers that may wish to implement and/or use the model.

Comparison with experimental results seemed to attest the relatively good accuracy and versatility of the model, even in those cases where the infill panels feature the presence of openings. It is nonetheless acknowledged that the latter was obtained only after some degree of fine-tuning and tweaking of the model, something which is obviously not possible in true numerical predictions. It is remarked that some discrepancy originating from possible disagreement between the pseudo-dynamic (test) and dynamic (analysis) responses could also be expected. Much further research work in this area (i.e. modelling of infill panels with openings) is clearly required.

It is noted that the model verification was carried out only at global level, hence one important aspect that remains to be investigated is the adverse local effects that the infill panels may cause due to their interaction with the surrounding frame. If the shear forces, developing at the contact between the infill panel and concrete members are excessive, the result may be a brittle failure. Future work should assess if the model manages to capture such local phenomena in adequate manner or not.

Finally, it is recalled that, as described in the body of the manuscript, the implemented element is capable of describing only the commonest of modes of failure, since a model that would account for all types of masonry panel failures could result being unpractical, due to the appreciable level of complexity and uncertainty involved.

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