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IMPLEMENTATION AND VERIFICATION OF A MASONRY PANEL MODEL FOR NONLINEAR PSEUDO-DYNAMIC ANALYSIS OF INFILLED RC FRAMES

Eleni SMYROU[1], Carlos BLANDON-URIBES[1], Stelios ANTONIOU[2], Rui PINHO[3] and Helen CROWLEY[4]

SUMMARY

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 paper, the implementation, within a fibre-based Finite Elements program, of an advanced double-strut nonlinear cyclic model for masonry panels is described. The accuracy of the model is first assessed through comparison with experimental results obtained from pseudo-dynamic tests of large or full-scale frame models. This is followed by a sensitivity study whereby the relative importance of each parameter necessary to calibrate the model is evaluated, so that guidance on the general employment of the latter can be given. Furthermore, a representative range of values for the geometrical and material properties of the infill panels has been also defined.

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. The highly nonlinear response of the infilled frames even at low-load levels makes a linear elastic approach inadequate. Therefore, the use of sophisticated constitutive relations is needed. A double-strut nonlinear cyclic model is employed here to describe the infill panels' behaviour. As Crisafulli [1997] points out the double-strut model is satisfactorily precise and less complicated compared respectively to a single- and a triple-strut model (see Figure 1) and able to represent more accurately the local effect between the infill and the frame.

For the needs of this paper, all analyses were carried out with the program SeismoStruct [SeismoSoft, 2006], within which the fore-mentioned model is implemented. SeismoStruct is an internet-downloadable fibre-based Finite Element package capable of predicting the large displacement behaviour of space frames under static or dynamic loading, considering both geometric nonlinearities and material inelasticity. Specifically, the sectional stress-strain state of beam-column elements is obtained through the integration of the nonlinear uniaxial material response of the individual fibres in which the section has been subdivided, thus fully accounting for the spread of inelasticity along the member length and across the section depth.

[1] *European School for Advanced Studies in Reduction of Seismic Risk (ROSE School), c/o EUCENTRE,*, Via Ferrata 1, Pavia 27100, Italy.
Email: esmyrou@roseschool.it
Email: cblandon@roseschool.it
[2] *SeismoSoft - Software Solutions For Earthquake Engineering*
Email: s.antoniou@seismosoft.com
[3] *University of Pavia, Department of Structural Mechanics*, Via Ferrata 1, Pavia 27100, Italy.
Email: rui.pinho@unipv.it
[4] *European Centre for Training and Research in Earthquake Engineering (EUCENTRE),* Via Ferrata 1, Pavia 27100, Italy.
Email: helen.crowley@eucentre.it

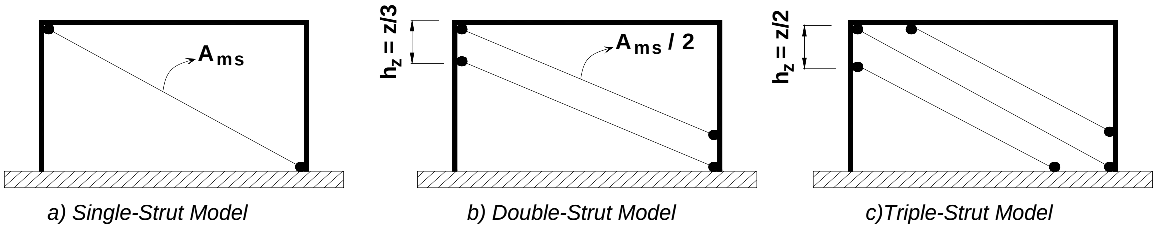


Figure 1: Modified strut models [Crisafulli, 1997]

For the numerical calculations static time-history analysis was employed. The accuracy of the model is assessed through comparison with experimental results obtained from pseudo-dynamic tests of a full-scale frame model, which was tested at the ELSA reaction-wall laboratory within the framework of the ICONS research programme [Pinto *et al.* 1999]. Two identical frames were constructed and tested, one bare and one infilled. The full-scale, four-storey, three-bay reinforced concrete frame considered in this paper was infilled with brick walls that included openings of different dimensions. Finally, after the calibration of the model sensitivity analysis is conducted to evaluate the importance of the parameters involved.

2. CASE STUDY FRAME

The case study RC frame can be regarded as representative of the design and construction practice of 40-50 years ago in countries such as Italy, Greece and Portugal. The frame was thus not expected to meet the modern code seismic design requirements, for which reason no specific detailing provisions were considered, no preferential inelastic dissipation mechanisms were assumed and no specific ductility or strength was provided [Pinto et al. 1999]. The frame is designed to withstand only vertical loads, while its resistance to horizontal loads, as far as the ultimate limit state is concerned, is approximately 8% of its weight. Similarly, the lateral resistance in terms of allowable stresses is 5% of its weight reflecting the common practice in those decades.

The four-storey frame consists of two bays of 5.0m span and one of 2.5m span. The inter-storey height is 2.7m, the slab thickness 0.15m with a width of 4.0m and effective width taken equal to 1.0m and 0.60m for long and short spans respectively. The overall mass is 179.23 tonne. All beams have equal geometry in all floors: 0.25×0.50m for the longitudinal beams, 0.20×0.50m for the transversal ones. The columns have the same geometrical characteristics along the height, i.e. 0.20×0.40m and 0.20×0.30m, apart from the second 'strong' column with dimensions 0.60×0.25m, that is the only one that works in its stronger axis (Figure 2). Duplication of the longitudinal reinforcement, which consists of smooth round bars, occurs at the bottom of the columns of the 1st and 3rd floor due to lap-splice for 0.70m. Stirrups are provided every 0.15m. The column reinforcement splicing, joints and stirrup detailing are characterized by a distinct lack of confinement.

The materials were chosen to have properties as similar as possible to those used in the late '70s in the construction in the southern European countries. The concrete used corresponds to a normal weight low strength concrete, class C16/20, but since compression tests on concrete specimens were carried out, the average values obtained for each type of element (beams, columns, slabs) of all floors were used in the numerical model. Furthermore, from best-fit of tensile strength tests on steel bar specimens, the mean mechanical properties were estimated. So the values of 204.5GPa and 2.8GPa for Young modulus and tangent modulus at the beginning of hardening were respectively considered. The yield stress is 343.6MPa, the ultimate strength is 451.5MPa and finally the ultimate strain is 22.9%. The non-load bearing infill panels were constructed after the frame, using hollow bricks, horizontally perforated with dimensions 0.120×0.245×0.245m and specific weight by m^2 of wall equal to 0.785kN/m^2. Plaster of 15mm was applied on both sides of the wall, causing an evident increase in stiffness and strength. Wallets representative of the masonry infill panels were tested in the horizontal, vertical and diagonal direction. The general layout of the location and the dimensions of the infill panels are given in Figure 2.

The experimental seismic response was obtained by applying pseudo-dynamic test method, i.e. a step by step integration technique to compute the displacement response of the frame that was subjected to three different, numerically specified seismic records, utilizing the non-linear restoring forces actually developed during the test. The input seismic motions were chosen to be representative of a moderate-high European hazard scenario. Consistent with the hazard, acceleration time-histories were artificially generated [Campos-Costa and Pinto, 1999] and finally three of increasing return periods of 475, 975 and 2000 years were used for the experiment.

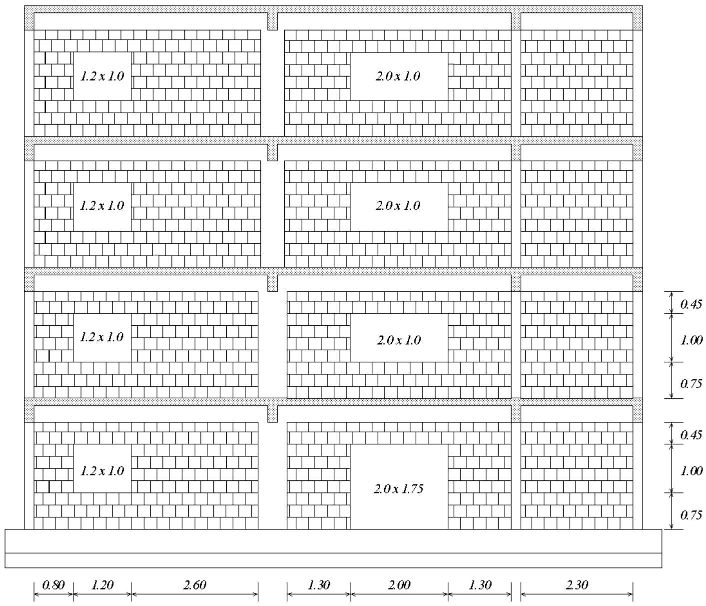


Figure 2: Elevation view of the infilled frame – Location and dimensions of openings [Pinto et al. 1999]

Further information about the case study frame as well as the tests conducted in ELSA can be found in Pinto et al. [1999], Carvalho et al. [1999], Pinho and Elnashai [2000].

3. MODELLING AND IMPLEMENTATION

3.1 Numerical Model of the Case Study Frame

The case study frame was modelled in the aforementioned FE package. Each structural member was subdivided into four inelastic beam-column elements with smaller length at the member ends so as to ensure the accurate modelling of expected plastic-hinge zones. Beams and columns are modelled as extending from the centre of one beam-column joint to the centre of the next. The mass is modelled as lumped mass, while the vertical loads defined in the model are applied in the beam-column nodes and simulate the self-weight of the frame, including the weight of the finishings and the masonry infill panels, as well as the live load. For concrete, the Mander et al. [1997] nonlinear model with constant confinement is used, while for steel the Menegotto-Pinto [1973] model with Filippou hardening isotropic rule [1983] is chosen.

The experimental values of base shear from the pseudo-dynamic tests on the bare frame were compared to the results obtained for the numerical model, exhibiting a satisfactory match as it is shown in Figure 3. It is noted that the bare frame was tested only for the 475yrp and 975yrp records. The latter was interrupted since the frame experienced severe deformations approaching collapse. Moreover, the fundamental period of the numerical model is 0.679sec, exceeding only by 6.6% the estimated first period of the test frame (0.637sec). Confirmation of the good match between the behaviour of the bare frame numerical model and the experimental frame has been carried out to ensure that the only variable in the verification of the infilled frame model was the infill panel model and thus other sources of uncertainty did not have to be considered.

3.2 Infill Panel Modelling

3.2.1 Implemented Model

The implemented four-node masonry panel elements are designed to represent the behaviour of infill panels in framed structures. Each panel is represented by five strut members, two parallel struts in each diagonal direction and a single strut acting across two opposite diagonal corners to carry the shear from the top to the bottom of the panel. This last strut acts across the diagonal which can be on compression and so connects different top and bottom corners depending on the deformation of the panel. The first four struts use the masonry strut hysteresis model, developed by Crisafulli et al. [2000], which consists of five rules that take into account the possibility of different stress paths (Figure 4), while the shear strut uses a bilinear hysteresis rule.

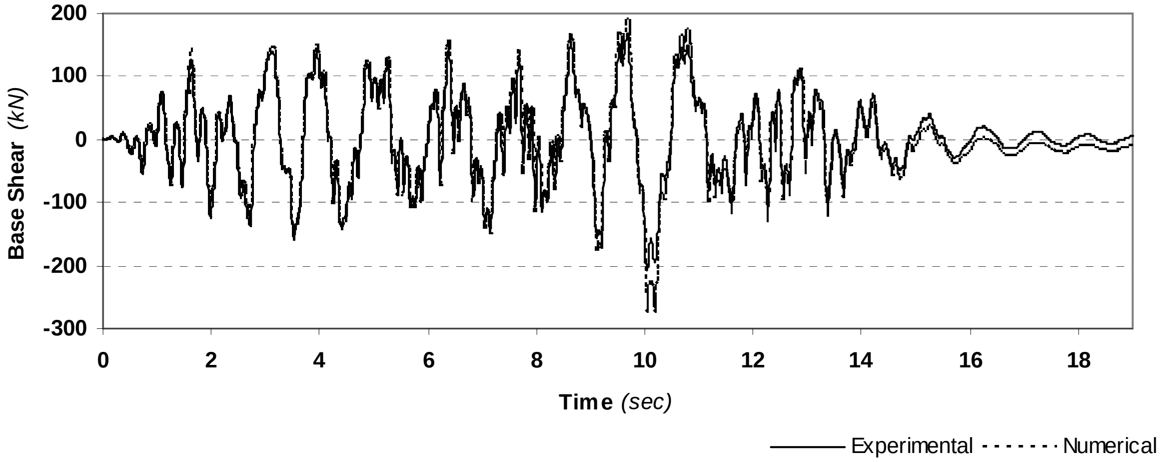
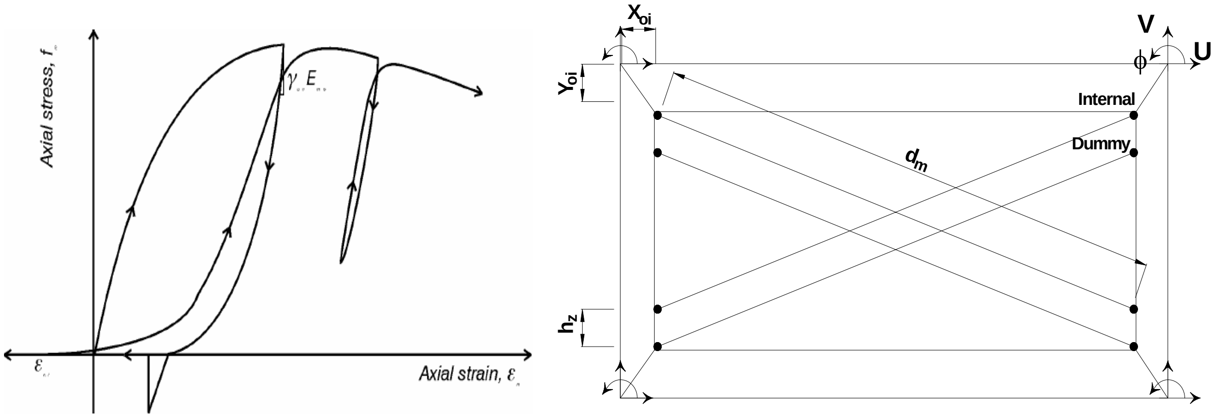


Figure 3: Comparison of the base shear of the bare frame (475yrp record)

As can be observed in the detailed configuration of the model given in Figure 4, the introduction of 4 dummy nodes is intended to represent the contact length between the frame and the infill panel, allowing to somehow approximately take account of the effect of local phenomena, while the 4 internal nodes stand for the frame-infill contact at the exterior part of the column and the beam. All the internal forces are transformed to the exterior 4 nodes where the element is connected to the frame. It should be finally noted that this strut configuration describes only the commonest mode of failure caused by shear since a model that would account for all types of masonry failure would not be practical due to the appreciable level of complexity and uncertainty involved [e.g. Crisafulli, 1997].



(a) (b)

Figure 4 : (a) Masonry strut hysteresis and (b) Infill panel element configuration

3.2.2 Mechanical and Geometrical Parameters

The calibration of the model entailed careful selection or calculation of the parameters involved. A number of mechanical and geometrical parameters are required to define the behaviour of the masonry struts. A list of the variables needed as input data is presented below as well as with recommendations for the selection of their values and the values that are finally implemented.

- Compressive strength $f_{m\theta}$: this is the parameter that mainly controls the resistance of the strut and has to be distinguished from the standard compressive strength of the masonry by taking into account the inclination of the compression principal stresses and the mode of failure expected in the infill panel. Its value for the long- and the short-bay panels is estimated as 5.0MPa and 3.5MPa respectively.
- Elastic modulus E_m: this parameter represents the initial slope of the strain-stress curve and its values exhibit a large variation. Different approaches can be found in the literature for the calculation of E_m, most of them relating the modulus of elasticity of masonry walls with the compressive strength of the material. These empirical equations result in their majority in a range of values between $400f_{m\theta}<E_m<1000f_{m\theta}$ [Crisafulli, 1997].

Accordingly, E_m values for the long- and short-bay panels are estimated as 5000MPa and 3500MPa respectively.

- Tensile strength f_t: this represents the tensile strength of the masonry or the bond-strength of the interface between frame and infill panel. Its presence offers generality in the model but it can even be assumed zero since it is much smaller than the compressive strength with insignificant effect on the overall response. That is why a uniform value of 0.575MPa, reported after diagonal compression tests, is used [Varum, 2003].
- Strain at max stress $\acute{\varepsilon}_m$: this represents the strain at maximum strength and influences via the modification of the secant stiffness of the ascending branch of the stress-strain curve. The value of 0.0012 provides the best results.
- Ultimate strain ε_u: this is used to control the descending branch of the stress-strain curve, which is modelled with a parabola to obtain better control of the strut response. For larger values such as $20\acute{\varepsilon}_m$, the decrease of the compressive strength becomes smoother. The value of 0.024 is implemented.
- Closing strain ε_{cl}: this parameter defines the strain after which the cracks partially close allowing compression stresses to develop. For large values its effect is not considered in the analysis. Suggested values between 0 and 0.003. Used value in the model 0.003.
- Bond shear strength τ_o and Coefficient of friction μ: the values for both parameters can be obtained from direct shear strength or from design specifications. Mann and Müller [1982] proposed an expression for reducing the usually overestimated values from shear tests. Paulay and Priestley [1992] indicate that τ_o values range between 0.1 and 1.5MPa and report a value of 0.3 for μ for design purposes, while a range from 0.70 to 0.85 for μ is obtained from literature survey by Atkinson et al. [1989]. In the model the values of 0.3 and 0.7 MPa are chosen for τ_o and μ respectively.
- Maximum shear stress τ_{max}: this is the maximum permissible shear stress in the infill panel and can be estimated using the expressions suggested in the modified Mann and Müller's theory [Crisafulli, 1997] according to the expected mode of failure. The value of 1MPa is adopted.
- Horizontal and Vertical offset, x_{oi} and y_{oi}: these represent the reduction of the infill panel's dimensions due to the depth of the frame members.
- Vertical separation between struts h_z: values of 1/3 to 1/2 of the contact length give satisfactory results. Equation (1) gives the contact length z, as defined by Stafford Smith [1966], who introduced the dimensionless relative stiffness parameter λ.

$$z = \frac{\pi}{2\lambda} \tag{1}$$

$$\lambda = \sqrt[4]{\frac{E_m t_w \sin(2\theta)}{4 E_c I_p h_w}} \tag{2}$$

in which $E_c I_c$ is the bending stiffness of the columns while the other parameters are explained in Figure 5.

- Thickness t_w: this stands for the thickness of the panel.
- Area of strut A: the strut area is defined as the product of the panel thickness and the equivalent width of the strut, which normally varies between 10% and 25% of the diagonal of the infill panel. There are numerous empirical expressions for the evaluation of the equivalent width that are presented in Figure 5.

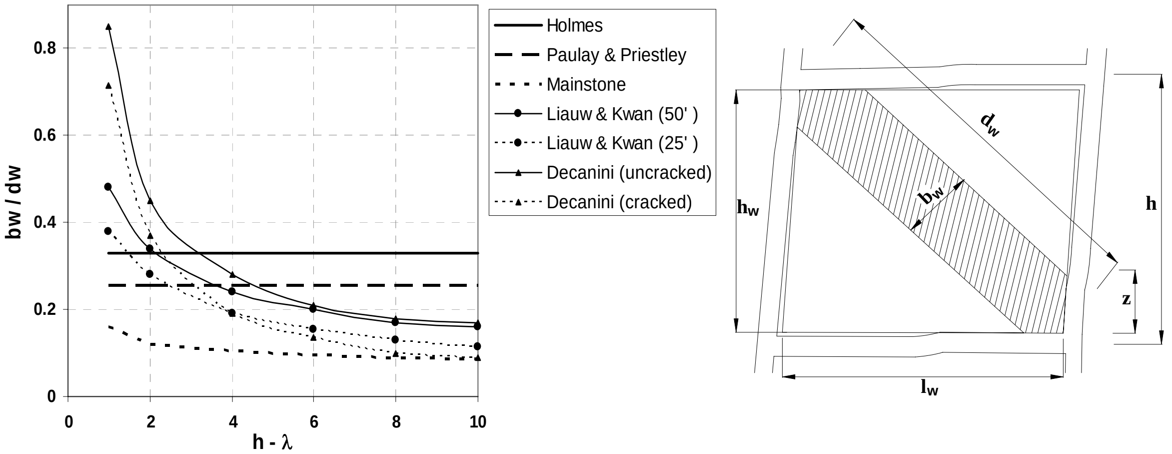


Figure 5: Variation of the ratio b_w/d_w as function of the parameter $h \cdot \lambda$ [Crisafulli, 1997]

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. In order to gain generality and achieve control of the variation of the stiffness and the axial strength of the strut, the value of the residual area is inserted in the model as percentage of the initial area. It is assumed that the area varies linearly as function of the axial strain so the two strains between which this variation takes place are required. Strains ranging between 0.0003 and 0.0009 are adopted while the residual area is considered 10% of the initial one.

The effect of the openings was taken into account by reducing the initially calculated equivalent strut area and thus the infill panels' stiffness. Several researchers [see CEB, 1996] suggest different reduction factors to describe the decrease of stiffness, depending on the dimensions and the position of the openings. Asteris [2003] proposes stiffness reduction factors for different opening percentages. In the model the strut area for the infill with the small window, the large window and the door is estimated as 70%, 60% and 50% respectively of the equivalent calculated strut area without considering the presence of openings. These percentages are slightly increased since in the hysteresis model the elastic modulus does not remain constant and additionally the fore-mentioned option to decrease the initial area is employed. Finally, no strength reduction due to openings was considered since in the literature this point is not fully clarified.

3.2.3 Empirical Parameters

A number of empirical parameters that participate in the model needs to be defined. A short explanation about their meaning is given below:

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 define 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.
γ_s: it represents the proportion of the panel stiffness assigned to the shear spring.
α_s: the reduction shear factor represents the ratio of the maximum shear stress to the average stress in the panel.

The suggested values presented hereinafter were obtained by Crisafulli [1997] after calibration of experimental data. However, out-of-range values were used for four of the parameters since this led to a better match with the experimental results.

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_s	0.5– 0.75		0.7
α_s	1.4 – 1.65		1.5

3.3 Results

The accuracy of the numerical model is evaluated by comparing the experimental results with the results obtained from the static-time history analyses. For that purpose the time-history of the base shear for each record is plotted (Figures 7 to 9), offering an immediate graphical comparison of the results. In order to compare the results numerically, a cumulative evaluation process has been used: the cumulative-absolute-base-shear values of both experimental and analytical analyses are normalized with the total-absolute-base-shear of the experimental tests. The results of this cumulative evaluation process, which are given in Figure 6, provide a representative picture of the cumulative error and thus of the deviation from the experimental values.

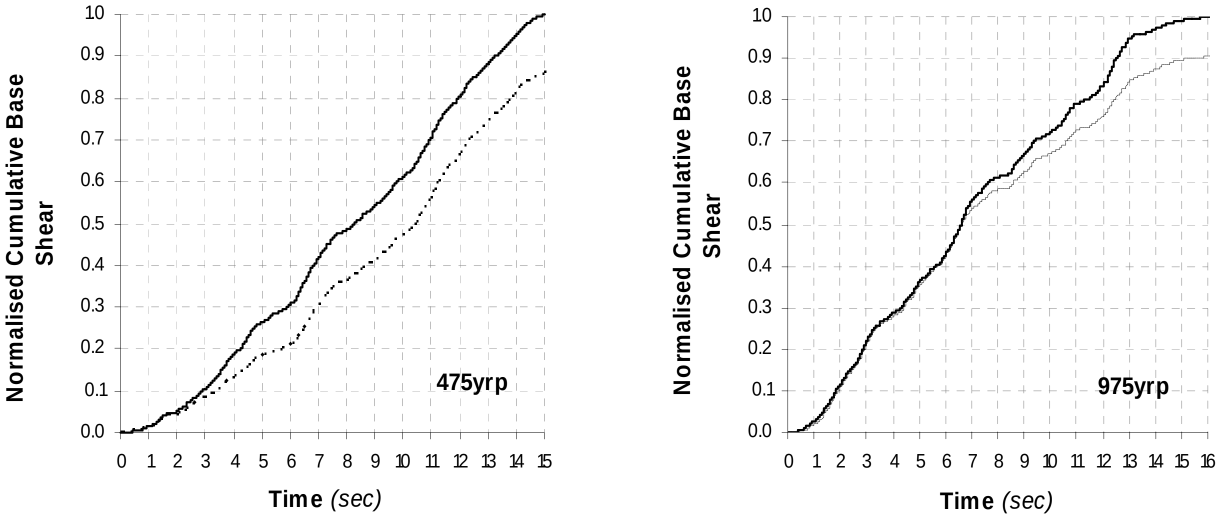


Figure 6: Normalised cumulative absolute values of base shear versus time for the 475yrp and 975yrp seismic input records

Finally, in an attempt to quantify the accuracy of the model, the peak values of base shear as absolute values and as percentage of the equivalent experimental ones is reported in Table 2.

Table 2: Comparison of base shear values

Input motion	Experimental Max. abs values (KN)	Numerical Max. abs. values (KN)	%
475yrp	763	793	1.04
975yrp	868	814	0.94
2000yrp	535	521	0.97

As far as the base shear time-history is concerned, the analytical results demonstrate a satisfactory match with those of the experiment, with differences occurring at the peak values for a limited number of cycles. The cumulative error for the 475yrp record exhibits the largest deviation, though this should be within an acceptable range. Although not plotted here due to space limitations, it has been found that the cumulative plot for the 2000yrp record matches perfectly with the equivalent experimental one, though the test under the 2000yrp record was interrupted since the frame approached imminent collapse and thus it is perhaps to be expected that the cumulative error is lower due to the reduced length of this record. Finally, in terms of maximum absolute values, the model manages to predict with satisfactory precision the peak value of base shear for all three seismic input records.

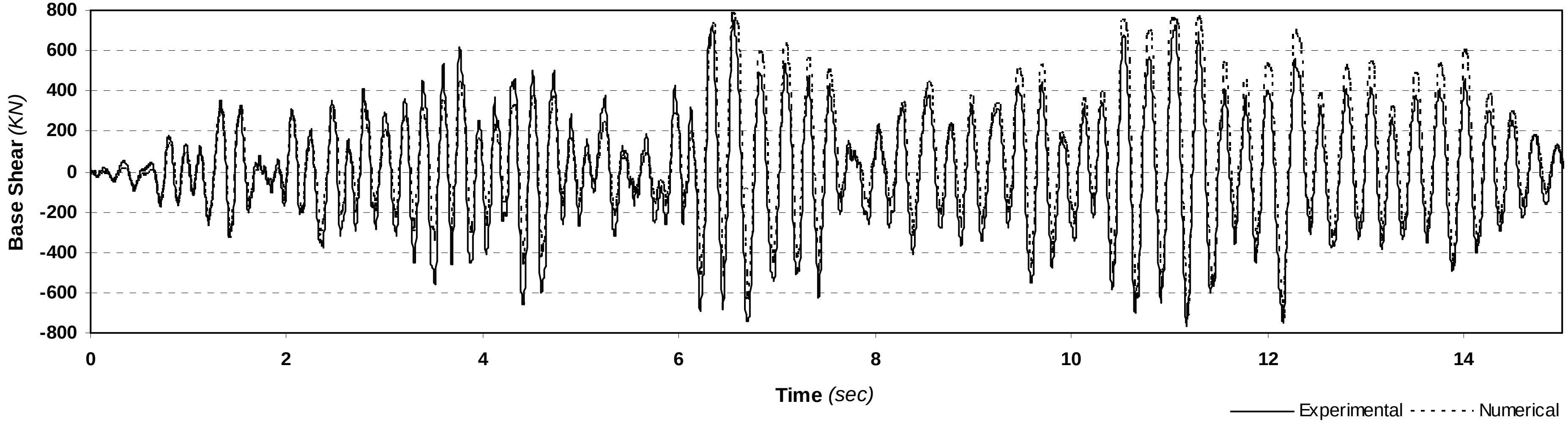


Figure 7: Comparison of the base shear of the infilled frame (475yrp record)

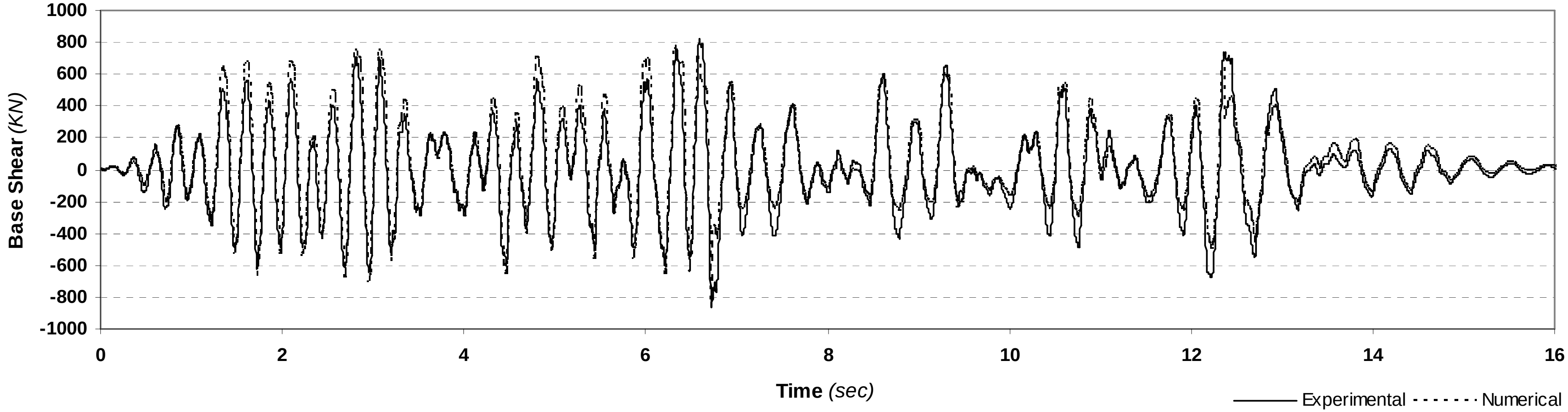


Figure 8: Comparison of the base shear of the infilled frame (975yrp record)

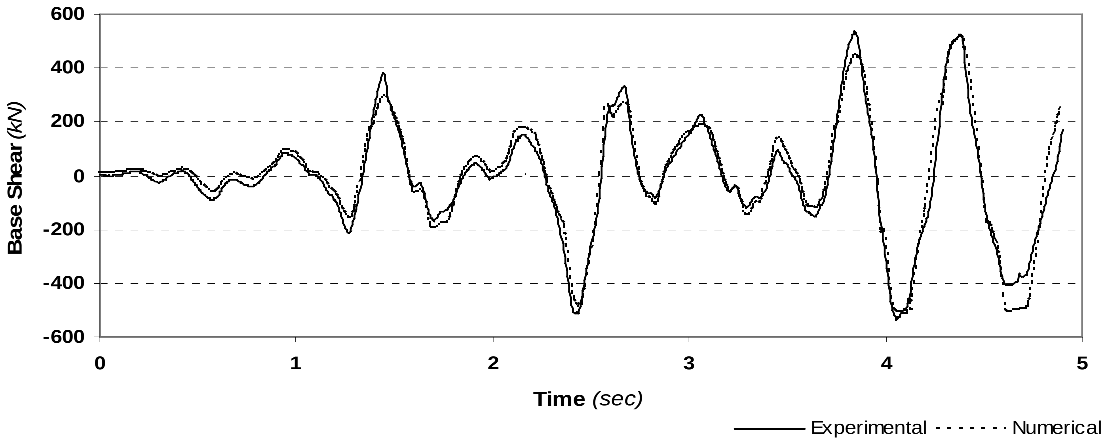


Figure 9: Comparison of the base shear of the infilled frame (2000yrp record)

4. SENSITIVITY ANALYSIS

In order to evaluate the relative importance of the less intuitive and harder to calibrate empirical parameters included in the model, a sensitivity analysis is conducted. The measure of the influence of each parameter is the difference between the final cumulative absolute value of base shear for the modified model and the equivalent cumulative value of the base model, both normalised by the latter. The results of this study are synoptically presented in Figure 11, in which all the empirical parameters that did not exhibit a significant influence on the behaviour of the base model (less than 1%) are not included.

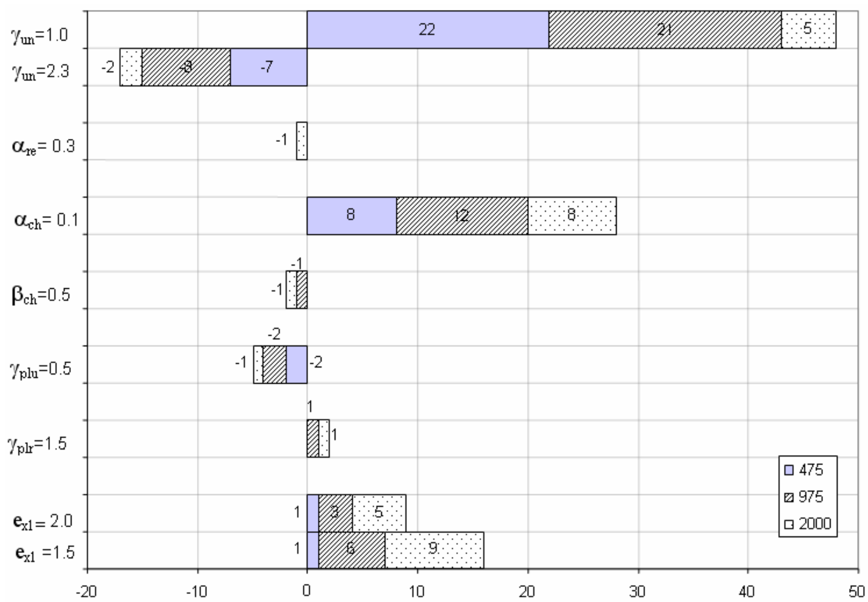


Figure 10: Sensitivity analysis results of infill model parameters in terms of deviation from the base model

The parameter γ_{un} has the most significant influence on the model's behaviour, exhibiting a wide range from -7% to 22% in the case of 2000yrp record. Considerable effect is induced also by α_{ch} with comparable values for all records, while e_{x1} affects mainly in the range of large displacements (2000yrp record). Since γ_{un} determines the shape of the loop by controlling the tangent modulus at the beginning of the unloading curve and e_{x1} controls the tangent modulus corresponding to the plastic strain of the unloading curve again, while α_{ch} defines the fatness of the loops, all parameters in fact decide the amount of dissipated energy.

5. CONCLUSIONS

The full-scale infilled RC frame, tested in reaction-wall laboratory, has been modelled in a fibre-based Finite Element program. A double-strut nonlinear cyclic model for masonry panels has been employed in order to describe the behaviour of the infill panels in an attempt to reproduce the response of a pseudo-dynamically tested frame, in which displacements were imposed. The presence of openings has been also taken into consideration. The numerical results obtained by static time-history have been compared to the experimental ones in terms of time-histories, cumulative and maximum absolute values of base shear, exhibiting a satisfactory match. Thus, the accuracy of the numerical model has been verified. The steps for the calculation or the selection of the geometrical, mechanical and empirical parameters included in the infill panel model are presented, accompanied by short explanation and representative range for each parameter, offering thus guidance on general employment of the infill panel mode. The calibration of the model is completed by conducting sensitivity analysis, in which the relative importance of the empirical parameters is evaluated.

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