DEVOPS-AN OPEN ACCESS JOURNAL
Open Journal

DevOps-An Open Access Journal

Review

On the Modelling of Infilled RC Frames through Strut Models

Marco Tanganelli; Tommaso Rotunno; Stefania Viti*

Department of Architecture (DiDA), University of Florence, via della Mattonaia 14, 50121 Firenze, Italy

Corresponding author
Stefania Viti
Department of Architecture (DiDA), University of Florence, via della Mattonaia 14, 50121 Firenze, Italy; E-mail: viti@unifi.it

Article information
Received: February 8th, 2023; **Revised:** May 3rd, 2023; **Accepted:** May 29th, 2023; **Published:** June 16th, 2023

Cite this article
Tanganelli M, Rotunno T, Viti S. On the modelling of infilled RC frames through strut models. 2023; 2(1). doi: https://doi.org/10.70705/ppp.doaj.2023.v02.i01.pp40-47

ABSTRACT

Infill panels largely affect the seismic response of framed constructions. The wide variety in their mechanical and geometrical features has produced many different models and assumptions in their analytical representation. In this paper the simplest and most diffuse analytical approach, based on the introduction of equivalent struts, has been checked. An overview is presented, focusing on the strut dimensions, strength and number. Two case-studies, taken by two different experi- mental campaigns, have been considered and reproduced. The obtained results have been compared to the experimental ones, and some parameters have been checked for selecting the model to use for analysis.

Keywords
Infilled RC frames; Strut-model; Masonry infill panels; Nonlinear response of infilled constructions; Strut strength.

INTRODUCTION

Infilled frame constructions have been extensively built during the last 200 years. The large diffusion of this type of buildings induced both an high interest of the scientific community in the evaluation of their seismic performance and a wide variety in their properties. The mechanical behavior of an infilled framed structure, indeed, depends on the mechanical properties of the structure and of the infill panels, but even on their mutual stiffness and the type of adopted connections, which can in- duce different collapse scenarios. Moreover, it is well known that masonry presents itself a wide variety of possible performances, due to the multiple combination among the constituting materi- als, and therefore having a behavior difficult to be predicted. As a consequence, many different ana- lytical procedures have been developed to represent the seismic behavior of the infilled frames, based on the observation of the mechanical properties of the single microelements constituting the system, or on a macroscopic approximated representation.

Moreover, the "quality" of the infill panels contribution to the seismic response of framed struc- tures is not obvious; the infill panels, in fact, increase the lateral stiffness and strength of the build- ings, with a consequent improvement of their safety. But some of the effects can be detrimental for the seismic response of the building, since the collapse—or detachment—of the infill panels can induce a reduction in ductility, with a possible activation of local collapse mechanisms. In the repre- sentation of the mechanical contribution of the infill panels to the seismic response of framed con- structions, therefore, the evaluation of the safest assumptions is not easy to pursue.

All the main International Technical Codes state the importance of including the infill panels in the representation of the seismic response of framed buildings. Nevertheless they differ with each other about the provided instructions for the representation. Eurocode 8 (EC 8-3, 2005) prescribes to take into account the infill panels contribution, without specifying the model to adopt to this purpose. The Federal Emergency Management Agency Code (FEMA 306, 1998; FEMA 356, 2000) suggests to rep- resent the infill panels contribution by introducing equivalent pin-jointed diagonal struts, having the same thickness and mechanical properties than the infill panels. Besides, the frame members are required to resist to the further internal forces induces by the equivalent strut.

The "equivalent strut" approach is the simplest way to account for the infill panels contribution; it has been introduced about 60 years ago (Holmes, 1961; Stafford Smith, 1966), and it is extensively used right now. The assumed cross section of the equivalent strut is usually fixed on the basis of the geometrical and mechanical properties of the infill material (Holmes, 1961; Mainstone, 1971), whilst the post-cracking behavior of the strut can be determined on the basis of experimental tests (Cavaleri & Di Trapani, 2015; Reinhorn, Madan, Valles, Reichman, & Mander, 1995), or by comparing the mac- ro-model results to the ones provided by nonlinear FE analysis (Asteris, Kakaletsis, Chrysostomou, & Smyrou, 2011; Durrani & Luo, 1994; Fardis, 1996). Since the point-jointed single strut does not

pro- vide any information about the distribution of shear force in the columns, equivalent multiple-struts models (Ali, 2009; Crisafulli, 1997; El-Dakhakhni, Elgaaly, & Hamid, 2003; Uva, Raffaele, Porco, & Fiore, 2012) have been also developed.

Many different—single and multiple—strut models have been proposed in these years, providing a large range of width for the diagonal strut. Moreover, the strength of the strut can be defined ac- cording to different assumptions, which refer to alternative collapse mechanisms of the panel. An exhaustive state of art of all possible analytical approaches to the problem of the infilled panels representation would be an arduous work, and is beyond the aim of this paper. This work is aimed at evaluating a range of variability, and the consequent possible reliability, of the standard strut models in representing simple infilled frames. Various formulations of strut models and alternative strength values of the infill have been compared, in order to evaluate, in term of maximum capacity, the range of results related to the more common assumptions. The considered assumptions have been applied to two different case-studies, which have been chosen after two different experimental surveys, respectively made by Al-Chaar, Issa, and Sweeney (2002) and Zhai, Kong, Wang, and Chen (2016). The two considered frames present different mechanical properties; namely, their ratio be- tween the infill panel and the frame stiffness is very different to each other.

The results found for the considered case-studies have been presented in Chapter 4. The capacity curves found by adopting all the considered assumptions in terms of strut width, stress and number have been shown and compared to the corresponding experimental curves. The comparison has been checked in terms of elastic stiffness and maximum base shear, which can be assumed as the most representative response quantities. In Chapter 5 the role of each of the considered assump- tions, i.e. the strut width, strength and number, has been checked, in terms of amount of scatter in results related to each choice. Finally, a joined lecture of the two case-studies has been proposed to evaluate a suitable criterion to select the strut-model to use for analysis.

2. The considered strut models assumptions

The structural behavior of infilled frames under horizontal actions represented through strut-models depends on three main issues (Chrysostomou & Asteris, 2012; Uva et al., 2012), i.e. the width and the mechanical characterization of the strut, which determine the stiffness and strength assumed for the panel, and the model setting, i.e. the number of the introduced struts and the consequent de- scription of each of them. In the following sections each of these issues has been presented. In Figure 1 a standard scheme of an infilled frame is shown, and the main symbols used in the follow- ing paragraphs to indicate the main geometrical and mechanical quantities are reported.

2.1. Strut width, w

Several contributions have been dedicated to the evaluation of the strut width. An exhaustive gath- ering of all the expressions set in these decades to quantify the width of the equivalent strut in beyond the scope of this work. Usually the thickness of the strut, tinf, is assumed to coincide to the panel one, whilst the width of the panel is defined after the panel dimension. Holmes (1961), as well as Paulay and Priestely (1992), proposed to relate the strut width directly to the diagonal length (see Table 1). Most part of strut models, however, assume more complex relationships to quantify the strut width. An important parameter for infill panels classification is the dimensionless quantity λh,

Based on the experimental and analytical data, Mainstone (1971) proposed a relationship between
λh and the strut width, which has been adopted by the FEMA guidelines (FEMA 306, 1998).
Some other researchers proposed to relate the strut width to the stress attained by the panel. Decanini and Fantin (1986) observed a correlation between the stress level in the panel and its ca- pacity to carry lateral loads (Flanagan & Bennett, 1999; Stafford Smith & Carter, 1969), and they provided two alternative expressions depending on the achievement of the panel cracking point. Durrani and Luo (1994) also proposed a semiempirical equation, on the basis of the experimental work of Mainstone (1974) and detailed numerical FEM analyses, by account for the stiffness of both beam and column of the frame.

2.2. Strut mechanical properties: Em, fm

According to the strut models, the stiffness of the infill panels depends on the strut dimensions and on the modulus of elasticity assumed for the infill material. Different relationships, very different from
each other (see Table 2), are available to quantify the modulus of elasticity (Em) of masonry. The stiff- ness of masonry panels, indeed, can vary very much depending on the mutual properties of bricks
and mortar. Due to large scatter in the Em evaluation, several researchers converge to the simple equation: Em = 1,000 fm, i.e. the same relationship adopted for concrete infills. In this work, the values of the elastic modulus of the equivalent strut have been defined according to this last relationship.

The quantification of the strut strength to assume for the infill representation is even more com- plex and controversial, and a large variety of values can be found in the technical literature. This wide scatter of strength values depends on the difficult prediction of the physical behavior of the system "frame + infill panel", which depends, in turn, on the specific and mutual properties of bricks, mortar and frame geometry, mechanical properties, strength and ductility. Since different collapse mechanisms can occur, a single and univocal value able to represent the strut performance is hard to find. This problem is usually faced by considering alternative strength values to assume for repre- senting the infill behavior, depending on the assumed collapse mechanism.

The infill collapse is usually related to three different in-plane mechanisms, despite even the out- of-plane failure can be possible (FEMA 306, 1998), under certain conditions. Such mechanisms are bed-joint sliding, diagonal tension and corner crushing.

The first cracking is usually related to the shear force in the panel, and it results in a stair stepped pattern of cracks in the mortar along the infill diagonal. For strong mortar, comparing to the units, the cracks show a linear trend, crossing both mortar and units, along the diagonal (diagonal crack- ing). For mortar much weaker of the units, instead, the cracks can propagate along the horizontal beds instead presenting a diagonal pattern (bed-joint sliding), leading to a more diffuse strength transfer between the panel and the surrounding frame. When the panel is sufficiently strong in shear, it can contribute to the global capacity of the system even after the cracking, until the crush- ing of the compressive corners along the panel diagonal (corner compression).

FEMA 306 is one of the International Code more supportive in the analysis of infilled frames by means of strut models. It indicates four limit conditions for the infill panel in the plane, referred to as much collapse scenarios, respectively related to sliding shear, compression, diagonal tension and general shear. The limit conditions are expressed in terms of the ultimate shear of the panel, and they refer to different resistant mechanisms.

The diagonal cracking (DC) failure is expressed by the following expression of the limit shear VDC (FEMA 396, Equations 8–11) (FEMA 306, 1998):

based on to the Mohr-Coulomb failure theory, after the assumption of a cracking stress equal to 1/20 of the expected strength of masonry in the horizontal direction, f□ m90, assumed in turn equal to 50% fm. Since in this work the numerical analyses are performed on strut-models only, a stress value for the strut has been found, by expressing the maximum force in the diagonal strut (DDC) as DDC = VDC / cos θ and by assuming an uniform stress distribution in the strut, i.e. fDC = DDC/(tinf w).

The bed joint sliding (BJS) usually occurs in conjunction with other failure modes and it is quanti- fied according to the Mohr-Coulomb failure criteria. The expression provided by FEMA 306, modified according to (Alva, Kaminski, Mohamad, & Silva, 2015), provides the following limit condition:

Where σg is the vertical stress due to the self-weight of the panel and μ is the coefficient of friction. Even in this case, the axial capacity of the strut, DBJS, can be determined as DBJS = VBJS/cos θ, and the strut stress fBJS is expressed as fBJS = DBJS/(tinf w).

The corner compression (CC) failure is defined on the basis of a strut-model, where the width is quantified according to the Mainstone (1971) proposal, and the compressive stress in the strut, fCC, is assumed to be equal to the expected strength of masonry in the horizontal direction, f□ m90 = 0.5 fm.

Several further formulations could be assumed for the width capacity quantification, provided by as much researchers. Special attention should be paid to the contribution given by Crisafulli (1997), who proposed suitable stress formulation based on the Mann and Muller (1982) theory, appositely modified. The Mann and Muller theory studies the behavior of unreinforced masonry after shear and compressive stresses, based on equilibrium considerations. It assumes that stress parallel to the bed joints can be neglected, so that no shear stress can be transferred by the head joints and a uniform distribution of shear and compressive stress inside the masonry panel. Crisafulli modified this last assumption, suggesting an alternative linear distribution, shown in Figure 2.

The limit stress values provided by Crisafulli (1997) refer to three different collapse mechanisms, related to the attainment of as much as limit stress values inside the strut, i.e. the shear friction (fSF), the diagonal tension (fDT) and the compressive (fC) ones. According to this approach, the stress to assume for the strut is the minimum among the three limit values. For standard geometries, i.e. for θ below 87° (Crisafulli, 1997) the collapse mechanisms are related to shear friction or diagonal ten- sion only, and therefore the possible stress values to assign to the strut are provided by the following relationships:

and Cn and Cs are coefficients determined by means of experimental analyses, and assumed equal to 1.5 and 2.0 respectively, while d and b are the brick length and height.

Liuaw and Kwan (1983a, 1983b) have derived different formulations for the limit stress values to assume for analysis of strut models, based on the plastic collapse theory. Even their approach con- siders three different collapse mechanisms, i.e. the corner crushing with failure in the column, the corner crushing with failure in the beam and the diagonal crushing mode.

2.3. Strut number

The number of struts to adopt in the modeling of infilled frames depends on the purpose of the analysis, and on the quality of the expected information. The most common models are made by 1 strut only; they are made by 1 strut for each frame diagonal (i.e. two struts for each frame), since they are assumed to be active only after compressive forces. The adoption of struts effective after compression only is usually acceptable, since the bond strength at the panel-frame interfaces and the tensile strength of the masonry are very low. In this case, therefore, only low tensile actions can be transferred.

The point-jointed 1-strut models can provide affordable information about the increase and distri- bution in shear force in the columns due to the infill panel interaction. To overcome this limitation, multiple-struts models (Ali, 2009; El-Dakhakhni et al., 2003; Uva et al., 2012) can be adopted, or ad- ditional restraints can be introduced at the ends of the strut (Mosalam & Ayala, 1996).

The need to know the effective stress distribution in the areas next to the corner among column, beam and diagonal strut induces to represent the system through multiple struts. The multiple-struts models (see Figure 3) consist in introducing two or three (for each direction) diagonal struts. In this way the amount of axial load in the struts is found together with the effective shear distribution in the members, with a consequent consistency between these quantities.

In multiple-struts models, the mutual distance between two consecutive struts, z, introduced by Stafford Smith (1966), plays an important role, since it influences the stress distribution in the frame, in-

dicating the areas subjected to shear concentration, where an higher ductility is required.

Usually the total area of all the struts, Ainf, is assumed to be the same of the single strut one, i.e. equal to the product of the thickness t and the width, w, quantified according to § 2.1, and each of the lateral struts are assume to have the 25% of the total stiffness, whereas the remaining 50% quote is assigned to the central one (Crisafulli, 1997; Verderame, De Luca, Ricci, & Manfredi, 2011). More complex models, having an higher number of struts or different geometry or resistant mechanisms (Crisafulli & Carr, 2007; El-Dakhakhni et al., 2003; Rodrigues, Varum, & Costa, 2010) can also be adopted.

3. The case-studies

Two different case studies have been considered in this work, belonging respectively to the experi- mental campaigns performed by Al-Chaar et al. (2002) and Zhai et al. (2016). The two considered systems differ to each other very much, as regards geometry, mechanical properties and relative stiffness between infill and frame.

The frame #1 (Al-Chaar et al., 2002) is made of reinforced concrete, and it has an infill panel made of bricks masonry, presenting high stiffness and strength. The experimental results have been found by means an in-plane monotonic loading, without any additional vertical loads, and the system can be assumed as scaled 1:2 comparing to the current dimensions of infilled frames. The case-study #2 (Zhai et al., 2016) is made of a reinforced concrete frame and a concrete blocks infill. The frame can be considered to be in scale 1:1, and it has been subjected to cyclic loading, besides a vertical load. The detailed description of the two case-studies can be found in the indicated references. In the fol- lowing sections the presentations of the two case-studies is limited to the information required for performing the analysis presented in Chapter 4.

3.1. Geometrical and mechanical description of the case studies

The experimental responses of the two case-studies, both the bare and the infilled ones, such as the shear force (Vbare_exp, Vinf_exp) and the elastic stiffness, measured for shear equal to 60% of the maximum (K60_bare_exp, K60_inf_exp), are shown in Figure 4. The two systems evidence a behavior very dif- ferent each other: in the case-study #1 the capacity of the infilled system is more than three times the bare frame one. Furthermore, the capacity of the system remains much larger of the bare frame's one even after the infill cracking. The system does not achieve large displacements, due to the predominant role of the masonry panel, characterized by a brittle behavior. The behavior of the case-study #2 is more typical of a RC frame, with a larger base shear and top displacement, and a smaller contribution of the infill panel, which vanishes after the panel collapse. The two systems dif- fer especially for the different role of the infill panel: in the first case the masonry infill plays the main role in the global behavior, whilst in the second one the infill panel, made of lighten concrete blocks, does not vanish the frame ductility. In Tables 3 and 4 all the main geometrical and mechanical information concerning the infilled systems are listed, while Table 5 shows the data re- ferred to columns and beams.

Furthermore, the stiffness and the shear capacity of the infilled frames have been checked, to re- late the "intrinsic" mechanical properties of the assumed strut models, compared to the bare frame ones, to the corresponding properties of the original system. As regards the frame, the stiffness (Kfr) has been quantified through a simplified expression, as the sum of the shear-type columns stiffness:

whilst the shear capacity of the frames, Vfr, has been found as the product of the columns sections and the shear stress (τc), assumes according to ACI 318-02 (American Concrete Institute, 2002) (rc = fc⁄6):

Vfr = 2 bchcrc (9)

The stiffness of the infill panels (Kinf), in turn, has been found after the following expression:

3.2. Data referred to the strut-models adopted for analysis

In Table 6 the width (w) found according to the considered strut-models, presented in § 2.1, have been listed for the two case-studies. Figure 5 shows the stress (fstr) values assumed for the strut. It should be noted that some of the considered stress values (provided by FEMA 306 for the diagonal cracking and the bed joint sliding collapse mechanisms) are not defined after the strut model, con- cerning instead the infill cross section (Ainf = tinf□ linf); in these cases, therefore, fstr is defined as the ratio between the compressive strength of the strut (DBJS, DDC) and the strut cross section, resulting differ- ent for each strut-model. The other stress values, instead (provided by FEMA for the CC and by Crisafulli for the BJS and DT), refer directly to the stress to give to the strut. As a consequence, some values of stress are constant for all the models, whilst some others are different for each considered strut width. As should be noted, the CC stress is larger than all the others, despite the BJS stress has similar (or even larger, for the case-study #2) values when the strut model proposed by Mainstone is adopted, since it leads to assume a strut with a depth much smaller than the other models.

The considered strut-models have been checked even in terms of stiffness (Kstr) and shear capacity (Vstr). Kstr has been quantified as the horizontal component of the axial stiffness of the struts (Einf Astr /Dinf), whilst Vstr has been found as the horizontal component of their axial capacity (Astr fstr), depending on the assumed stress for the strut.

In Figures 6 and 7 the ratios between strut and frame stiffness and shear capacity have been shown, respectively. As can be noted in Figure 6, the strut stiffness is sensitive to the assumed strut width, and therefore to the considered strut-models, whilst it is not sensitive to the considered stress values, since the same value of Young modulus has been assumed in all cases. In the same figure, the ratio between the stiffness of infill and frame, as defined in Equations (8) and (10), is even shown. The ratio between the strut and the frame shear capacity, instead, is sensitive both to the struts width and stress, as can be observed in Figure 7.

4. The seismic capacity of the case-studies

4.1. The numerical analysis

The Seismostruct platform (Seismosoft, 2006) has been adopted for the analysis. Each frame member has been divided in 4 branches. A fiber model has been assumed for representing the cross sections, where the concrete has been described through the Mander, Priestley, and Park (1988) relationship and the reinforce- ment has been represented by a bilinear model with a hardening ratio equal to 1%. The diagonal strut behavior has been represented by no-traction links introduced at its ends, described through asymmetric Takeda model. The pushover analysis has been performed by combined force and displacement control.

4.2. Experimental and numerical capacity curves

The capacity curves obtained by performing the numerical analysis have been compared to the ex- perimental ones. Figures 8 and 9 show the obtained capacity curves for the two case-studies, respectively. In each Figure the capacity curves, obtained by adopting the considered models and stress values, have been shown for the models with 1, 2 and 3 struts, respectively. In the same Figures the limit conditions assumed in the analysis, i.e. the achievement of the limit values for the concrete strain, chord rotation and shear force, are shown. All the limit conditions have been taken after the EC8 provisions: the ultimate concrete strain has been assumed equal to 0.0035, the limit chord rotation has been taken as ¾ of the ultimate rotation, defined according to Annex A of EC8 (Equation A1)(EC 8-3, 2005), as well as the limit shear (EC8, Equation A12) (EC 8-3, 2005).

As regards the case-study #1, the strut stress which better represents the experimental results is fBJS, provided by FEMA 306. Since fBJS, as fDC, has been derived for each strut model by the same hori- zontal force, all models provide the same results in terms of maximum base shear, whilst—as expectable—they differ to each other as regards the elastic stiffness. Even the strut stress values proposed by Crisafulli (fDT, fSF), which vary very much depending on the considered strut-model, pro- vide an acceptable approximation of the experimental results. The number of struts does not affect very much of the obtained capacity curves, in terms of maximum base shear or top displacement. When the 2-strut model is adopted, however, the collapse mechanism of the system changes, and the limit shear force is achieved in the columns for lower loads than in the other cases.

The case-study #2 differs very much from the #1, as regards the comparison between experimen- tal and numerical results. First of all the stress value more adequate to describe the real behavior of the system is the larger one (fCC), representing the corner crushing of the compressive strut. All the other stress values provide results well below the base shear found experimentally. Furthermore, except for fCC, the system is not sensitive to the strut width, since all the pushover curves are very close each other for all the considered strut-models.

To better compare the numerical results to the experimental one, the elastic stiffness and the maximum base shear provided by the numerical analysis have been checked. Figure 10 shows the comparison in terms of elastic stiffness. Since the stiffness has a degrading trend at the increasing of the system strain even in the first steps of the analysis; the K60,inf values refer to the stiffness cor- responding to a base shear equal to 60% of the maximum one, i.e. the stiffness of the equivalent bilinear curves.

As can be noted, all the adopted models underestimate the stiffness of the case-study #1, whilst the stiffness of the case-study #2 is approached, especially when fCC and fBJS are assumed. These re- sults are not surprising, since they are compatible to the comparison made between Kstr and Kfr, shown in Figure 6.

In Figure 11 the maximum base shear provided by the adopted models for the two case-studies has been shown, together with the experimental values. In the Figure, as well as in the following sections, two different values of the base shear are shown, referring, respectively, the maximum values provided by the analysis (Vmax_inf) and to ones at the achievement of the limit conditions (Vlim_inf). In some cases (case-study #1, 2-strut), indeed, the limit conditions imposed to the system are achieved much before the complete development of the inelastic sourced of the frame.

5. Discussion of the obtained results

The results obtained in this work underline the uncertainties in choosing a proper strut model to represent the behavior of infilled frames. The considered case-studies, in fact, evidence many differences in the comparison with the experimental results. As regards the choice of the model, the ones which provide higher width for the strut (H, D&F_UN) seem to better catch the initial stiffness of the frames. As regards the choice of the stress to assume for the strut, the case-study #1 is better rep- resented by the lowest values (fDC, fDT, fSF), whilst the case-study #2 is better simulated by assuming the higher strut stress fCC. Finally, the number of struts to represent the panel does not significantly affect the capacity curves, resulting however to be important for the evaluation of the collapse mechanisms of the systems. In this section the role of the main choices related to the strut model have been discussed, and a joint lecture is proposed for the two frames.

5.1. Effects of the model assumptions on the response quantities

The effects of each of the considered assumption has been measured in terms of scatter in the re- sponse quantities, i.e. the elastic stiffness, K60,inf, and the maximum base-shear, Vmax,inf, for the two case-studies. The scatter has been measured through the percentage difference (% DIFF), defined, for each quantity, according to the following expression:

Figure 12 shows the scatter found for the two case-studies as a function of the strut width, quanti- fied by considering, for each stress value and number of struts, the maximum and the minimum values provided by the considered strut-models. As can be noted, the stiffness of the case-study #2 is sensitive to the strut width only when the CC stress is adopted, evidencing a scatter below 10% in all the other cases. The stiffness of the case-study #1, instead, is sensitive to the strut width in all cases (scatter between 30% and 55%). As regards the limit base-shear, it should be reminded that the BJS and

DC stress values have been defined in order to have the same strut strength in all mod- els; consequently all the capacity curves achieve the same amount of base shear, unless achieve- ment of limit conditions.

each strut model and number, to the results provided by the considered stress values. The case- study #1 evidences a difference sensitivity to the assumed stress in the two response quantities. In fact, the stiffness K60,inf has a negligible sensitivity, except for the M model, which achieves the post- elastic range before 60% of the maximum base shear, where the stiffness is measured. The case- study #2, instead, evidences the same sensitivity to the stiffness and the base shear, with a scatter ranging between 10% and 40% depending on the assumed model.

Finally, the effects related to the number of struts assumed in the model have been checked, and shown in Figure 14. In this case the maximum and minimum values have been found, for each strut model and stress, by considering the three values related to the considered strut number. As can be noted in Figure 14, this assumption is almost negligible for the stiffness, with values of % DIFF below 5% (except for the case-study #2, when the CC stress is adopted). The number of struts, instead, plays a crucial role in the evaluation of the maximum shear force in the case-study #1. As it was al- ready highlighted, indeed, the number of struts affects the attainment of the limit conditions im- posed to the systems; namely, in the case-study #1 the 2-strut model provides a premature failure of the system, comparing to the other models.

5.2. Comparison between the results found for the two case-studies

In order to better understand, and to compare, the results provided by the analysis for the two case- studies, they have been expressed in terms of non-dimensional quantities. Namely, the increase in elastic stiffness (iK) and in base-shear (iV) provided by the diagonal strut has been introduced to measure the effect of the infill panel in the considered models. Such increase has been normalized through the maximum quantities of the corresponding experimental test, according to the following relationships:

The increase in the response quantities have been related to the amount of stiffness and shear, which have been expressed as the ratio between the strut and the frame quantities (Kstr/Kfr, Vstr/Vfr), as defined in § 3.2 and shown in Figures 6 and 7. The iK and iV values obtained through the analysis for the two case-studies are shown in Figures 15–17, together with the "error" of the numerical models. The "error" has been defined in non-dimensional terms, as:

In Figure 15 the increase in stiffness has been shown as a function of the ratio between the strut and the frame stiffness. As should be noted, both case-studies evidence an increase in stiffness at the increasing of Kst/Kfr ratio, even if the case-study #1 presents an iK amount much larger than the case-study #2. In the same figure the error trend is shown. As regards the case-study #1, all the strut models provide a stiffness increase well below the experimental value, and the error is over 20% in all cases. In the same diagram the ratio between the infill and the frame stiffness—as listed in Figure 4—is shown. All the strut models have a stiffness much smaller than the infilled one. The Holmes model, which has the higher stiffness, provides the best approximation of the experimental result.

As regards the case-study #2, the strut models provide a similar stiffness to the infill panel, and several strut-models approach the experimental result with a satisfactory approximation, when the

FEMA strut stress values (fCC, fBJS, fDC) are applied. The quality of the results can be evaluated even by checking the ratio between the infill and the frame stiffness, evidenced in the graphs. In both cases

the Kinf/Kfr quantity corresponds to good results comparing to experimental ones.

Figures 16 shows the increase in base shear provided by the considered strut models. Both the maximum shear attained by the whole capacity curves (max shear) and the value at the achieve- ment of limit conditions (limit shear) are represented. The maximum shear, indeed, is more suitable to express the effects of the width and stress assumptions made for the struts, whilst the limit shear provides important information about the role plaid by the strut number. As regards the case-study #1, there is a good proportionality between iV and Vstr/Vfr for Vstr/Vfr below 3.0. For larger shear amount, this proportionality vanishes, and the iV keeps the same value, whichever strut stiffness is considered.

By comparing the diagrams of iV to the ones expressing the "error", it can be noted that the nu- merical results which better approach the experimental results have Vstr/Vfr values between 0.8 and 1.5. By comparing the diagrams expressing the error values for "max" and "limit" base shear, shown in Figure 17, it can be noted that the results obtained by the 3-strut models are "better", i.e. closer to the experimental ones. Even when the "stronger" strut mode is adopted (H model combined to fCC) the overestimation of the system capacity is corrected by the achievement of the introduced limit conditions.

In the case-study #2, Vstr/Vfr keeps always below 1, and the proportionality between iV and Vstr/Vfr works for all the obtained results. Even in this case, the fBJS and the fDC results do not follow strictly such proportionality, since at the varying of the strut width the contribution of the infill remains constant, due to the already mentioned stress quantification. Even for the iV, the obtained results can be compared to the Vstr/Vfr ratio. In the case-study #1 this quantity corresponds to unacceptable results; this evidence suggests that, whichever the panel capacity can be, the adoption of a strut model requires a strut capacity below the double of the frame capacity. The iK and the iV values obtained for the two case-studies have been compared in Figure 18. As regards iV, only Vstr/Vfr values below 2.5 have been considered. As should be noted, both iK and iV—when expressed in terms of Vmax—evidence a strong proportionality to the assumed reference quantities. As can be noted, each case-study presents a different proportionality factor; this difference is not surprising, since they dif- fer very much for the relative contribution of the infill.

6. Conclusive remarks

In this work a numerical analysis has been performed to simulate

the seismic response of two differ- ent infilled frames, differing for dimensions and mechanical properties and—above all—for the relative stiffness and strength of the infill panel comparing to the frame ones. The effects of the variation in the main properties of the strut model, i.e. the strut width, stress and number, have been checked in terms of elastic stiffness and maximum base-shear of the infilled frames.

According to the obtained results, the following observations can be made:

• all the checked parameters, i.e. strut width, stress and number, affect very much the obtained results;
• the adopted strut-model should fit as much as possible the replaced panel in terms of relative stiffness and shear capacity between panel (strut) and bare frame;
• the shear capacity of the strut cannot exceed a value equal to 2–3 times the one of the bare frame;
• the three-strut models provide a better representation of the panel contribution, reducing even- tually the overestimation of its capacity, when a too high stress is assumed for the strut.

In order to achieve more general results, the suggested "input quantities" should be checked on a larger number of samples.

Notes about the article

• Various strut models, differing for width, stress and number have been applied
• Two different case-studies have been assumed for comparison
• Some mechanical parameters expressing the infill contribution have been selected
• The response of each frame has been checked in terms of stiffness and shear
• The suitability of each model has been checked through the selected parameters

Funding
The authors received no direct funding for this research.

REFERENCES

Al-Chaar, G., Issa, M., & Sweeney, S. (2002). Behavior of masonry-infilled nonductile reinforced concrete frames. Journal of Structural Engineering, 128(8), 1055–1063. doi:10.1061/(ASCE)0733-9445(2002)128:8(1055)

Ali, H. (2009). Half Scale Three-Storey Infilled RC Building; A Comparison of Experimental and Numerical Models (Dissertation for the Master Degree in Earthquake Engineering). IUSS, Pavia.

Alva, G. M. S., Kaminski, Jr., J., Mohamad, G., & Silva, L. R. (2015). Serviceability limit state related to excessive lateral deformations to account for infill walls in the structural model. Revista IBRACON de Estruturas e Materiais, 8(3), 390–426. doi:10.1590/S1983-41952015000300008

American Concrete Institute. (2002). ACI building code requirements for reinforced concrete (ACI 318-02). Farmington Hills: ACI.

Asteris, P. G., Kakaletsis, D. J., Chrysostomou, C. Z., & Smyrou, E. E. (2011). Failure modes of in-filled frames. Electronic Journal of Structural Engineering, 11(1), 11–20.

Cavaleri, L., & Di Trapani, F. (2015). Prediction of the additional shear action on frame members due to infills. Bulletin of Earthquake Engineering, 13(5), 1425–1454. doi:10.1007/ s10518-014-9668-z

Chrysostomou, C. Z., & Asteris, P. G. (2012). On the in-plane properties and capacities of infilled frames. Engineering Structures, 41, 385–402. doi:10.1016/j.engstruct.2012.03.057

Crisafulli, F. (1997). Seismic behaviour of reinforced structures with reinforced masonry infills (Phd Thesis). University of Canterbury, Christchurch.

Crisafulli, F. J., & Carr, A. J. (2007). Proposed macro-model for the analysis of infilled frame structures. Bulletin of the New Zealand Society for Earthquake Engineering, 40(2), 69–77.

Decanini, L. D., & Fantin, G. E. (1986). Modelos simplificados de la mamposteria incluida en porticos. Caracteristicas de rigidezy resistencia lateral en estado limite. Jornadas Argentinas de Ingenieria Estructural, 2, 817–836.

Durrani, A. J., & Luo, Y. H. (1994). Seismic retrofit of flat-slab buildings with masonry infills. In Proceedings of the NCEER Workshop on Seismic Response in Masonry Infills (Technical Report NCEER 94-004, 3–8). Buffalo, NY: National Center of Earthquake Engineering Research.

EC 8-3. (2005). Design of structures for earthquake resistance, part 3: Strengthening and repair of buildings, European standard EN 1998-3. Brussels: European Committee for Standardization (CEN).

El-Dakhakhni, W. W., Elgaaly, M., & Hamid, A. A. (2003). Three-strut model for concrete masonry-infilled steel frames. Journal of Structural Engineering, 129(2), 177–185. doi:10.1061/(ASCE)0733-9445(2003)129:2(177)

Fardis, M. N. (Ed.). (1996). Experimental and numerical investigations on the seismic response of RC infilled frames and recommendations for code provisions (ECOEST/PREC 8, Report No. 6). Lisbon: LNEC.

FEMA 306. (1998). Evaluation of earthquake damaged concrete and masonry wall buildings: Basic procedures manual. Washington, DC: Federal Emergency Management Agency.

FEMA 356. (2000). Prestandard and commentary for seismic rehabilitation of buildings. Washington, DC: Federal Emergency Management Agency.

Flanagan, R. D., & Bennett, R. M. (1999). In-plane behavior of structural clay tile infilled frames. Journal of Structural Engineering, 125(6), 590–599. doi:10.1061/(ASCE)0733-9445(1999)125:6(590)

Hendry, A. W. (1990). Structural masonry. London: MacMillan Education Ltd.

Holmes, M. (1961). Steel frames with brickwork and concrete infilling. Proceedings of the Institution of Civil Engineers, 19, 473–478. doi:10.1680/iicep.1961.11305

Liauw, T. C., & Kwan, K. H. (1983a). Plastic theory of infilled

frames with finite interface shear strength. Proceedings of the Institution of Civil Engineers (London), Part 2, 75(December), 707–723.

Liauw, T. C., & Kwan, K. H. (1983b). Plastic theory of non- integral infilled frames. Proceedings of the Institution of Civil Engineers (London), Part 2, 75(September), 379–396.

Liaw, T. C., & Kwan, K. H. (1984). Nonlinear behavior of non-integral infilled frames. Computer Structure, 18, 551–560.

Mainstone, R. J. (1971). On the stiffness and strength of infilled frames. Proceedings of Institution of Civil Engineering, Supplement (IV), 57–90.

Mainstone, R. J. (1974). Supplementary notes on the stiffness and strength of infilled frames (Current Paper CP 13/74). Garston: Building Research Station.

Mander, J. B., Priestley, M. J. N., & Park, R. (1988). Theoretical stress□ strain model for confined concrete. Journal of Structural Engineering, 114, 1804–1826. https://doi. org/10.1061/(ASCE)0733-9445(1988)114:8(1804)

Mann, W., & Muller, H. (1982). Failure of shear-stressed masonry—An enlarged theory, tests and application to shear walls. Proceedings of the British Ceramic Society, 30, 223–235.

Mosalam, K. M., & Ayala, A. (1996). Effect of geometrical configuration on the seismic response of infilled frame. In 11thWorld Conference on Earthquake Engineering (Number 1675). Mexico: Elsevier.

Paulay, T., & Priestly, M. J. N. (1992). Seismic design of reinforced concrete and masonry buildings (p. 744). New York, NY: Wiley. https://doi.org/10.1002/9780470172841

Reinhorn, A. M., Madan, A., Valles, R. E., Reichman, Y., & Mander, J. B. (1995). Modeling of masonry infill panels for structural analysis (Technical Report NCEER-95-0018). Buffalo, NY: National Center of Earthquake Engineering Research, State University of New York.

Rodrigues, H., Varum, H., & Costa, A. B. (2010). Simplified macro-model for infill masonry panels. Journal of Earthquake Engineering, 14(3), 390–416. doi:10.1080/13632460903086044

Seismosoft. (2006). Seismostruct version 5.2.2—A computer program for static and dynamic nonlinear analysis of framed structures. Retrieved from www.seismosoft.com

Sinha, B. P., & Pedreschi, R. (1983). Compressive strength and some elastic properties of brickwork. International Journal of Masonry Construction, 3(1), 19–27.

Smyrou, E. (2006). Implementation and verification of a masonry panel model for nonlinear dynamic analysis of infilled RC frames. Dissertation for the MSc in Earthquake Engineering. European School for Advanced Studies in Reduction of Seismic Risk (ROSE SCHOOL). Università degli Studi di Pavia.

Stafford Smith, B. (1966). Behavior of square infilled frames.

ASCE, Journal of Structural Division, 92(ST1), 381–403.

Stafford Smith, B., & Carter, C. (1969). A method of analysis for infilled frames. Proceedings of the Institution of Civil Engineers (London), 44(September/December), 31–48.

Uva, G., Raffaele, D., Porco, F., & Fiore, A. (2012). On the role of equivalent strut models in the seismic assessment of infilled RC buildings. Engineering Structures, 42, 83–94. doi:10.1016/j.engstruct.2012.04.005

Verderame, G. M., De Luca, F., Ricci, P., & Manfredi, G. (2011). Preliminary analysis of a soft storey mechanism after the 2009 L'Aquila earthquake. Earthquake Engineering & Structural Dynamics, 40(8), 925–944. doi:10.1002/eqe.1069

Zhai, C., Kong, J., Wang, X., & Chen, Z. (2016). Experimental and finite element analytical investigation of seismic behavior of full-scale masonry infilled RC frames. Journal of Earthquake Engineering, 20(7), 1171–1198. doi:10.1080/1