Masonry infilled frame structures: state-of-the-art review of numerical modelling

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Abstract. This paper presents a state-of-the-art review of the nonlinear modelling techniques available today for describing the structural behaviour of masonry infills and their interaction with frame structures subjected to in-plane loads. Following brief overviews on the behaviour of masonry-infilled frames and on the results of salient experimental tests, three modelling approaches are discussed in more detail: the micro, the meso and the macro approaches. The first model considers each of the infilled frame elements as separate: brick units, mortar, concrete and steel reinforcement; while the second approach treats the masonry infill as a continuum. The paper focuses on the third approach, which combines frame elements for the beams and columns with one or more equivalent struts for the infill panel. Due to its relative simplicity and computational speed, the macro model technique is more widely used today, though not all proposed models capture the main effects of the frame-infill interaction.

Keywords: frame structures; masonry infill; equivalent strut model; finite element; nonlinear analyses.

1. Introduction

Masonry infilled frame structures are a construction typology widely diffused around the world. Interaction between the infill panel (mostly masonry) and the surrounding frame subjected to lateral loads has been studied since the late 50's. However, understanding this interaction is not straightforward because it depends on several parameters including the brick materials (clay, concrete, etc.), the mortar mechanical characteristics, the brick geometry (hollow or solid, *etc*), the workmanship quality, the relative stiffness between the frame and the panel, *etc*. Although they are generally considered as non-structural elements in the building model, the infills undoubtedly

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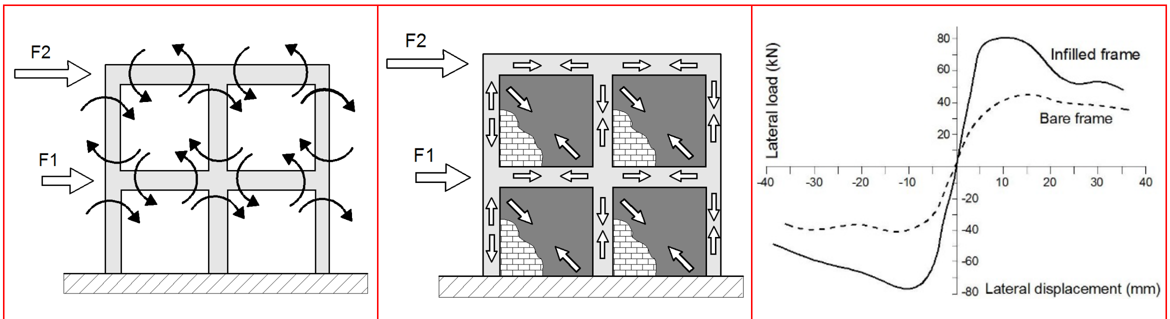
modify the frame seismic behaviour. When the infills are explicitly modelled, the above uncertainties make the modelling quite a complex issue. Furthermore, newer or recently refurbished buildings tend to have thicker (and thus stiffer and stronger) infills, in order to increase their energy efficiency, thus their influence on the seismic behaviour of the building increases.

In the case of relatively flexible frames, the bare frame carries the vertical load, whereas the frame and the infills jointly carry the horizontal seismic load, with a prevalent truss action mechanism in the infills (Fig. 1). The infill typically reacts along the direction between the upper corner of the windward column and the lower corner of the leeward column. For large displacements, the infill and the frame are in contact mainly in the above corners along so called "contact lengths". In the remaining two corner zones, the panel-frame contact is normally lost due to the difference in the deformation mode between the infill and the frame (*e.g.* Fig 2a and 2b).



(a) (b) (c)

Fig. 1 Behaviour of frames under lateral loads: (a) bare frame, predominant frame action; (b) infilled frame, predominant truss action (adapted from Kaushik *et al.* 2006); (c) qualitative comparison of the experimental cyclic response of a RC frame with and without infill panels (adapted from Kakaletsis and Karayannis 2008).

Five distinct in-plane failure mode categories are typically identified in the infilled frames (Asteris *et al.* 2011, Fig. 2):

a) ***Frame*** failure modes, which consist in the formation of plastic hinges in the beams and columns near the joints, or in the failure of beam-column joints, or, in very few cases, at the column mid-height. Frame failure may take place together with infill failure (Fig. 2a and 2c);
b) Infill ***sliding shear*** failure mode, in which the panel experiences horizontal sliding through multiple bed joints. It can occur when the mortar has poor mechanical properties and the infill aspect ratio is quite low, implying a significant horizontal component of the truss action (Fig. 2a and 2d);
c) Infill ***diagonal cracking*** failure mode, which consists of diffuse cracking along the panel compressed diagonal, which may take place when the frame is more flexible than the infill. It generally presents a stepped diagonal pattern along the mortar bed and the head joints. The cracking of the compressed diagonal does not imply collapse of the panel, which may develop a further resisting capacity. Sliding shear and diagonal cracking may take place as a mixed mode (Fig. 2a);
d) Infill ***diagonal compression*** failure mode, which consists of crushing of the panel centre. This failure mode usually occurs in slender infills, placed eccentrically with respect to the axis of the frame, and is accompanied by out-of-plane deformations and eventually collapse (Fig. 2b);

e) Infill ***corner crushing*** failure mode, which consists of crushing in a loaded corner area of the infill panel due to a biaxial compression state. This normally occurs when the structure has a weak infill panel surrounded by strong columns and beams with weak infill-frame interface joints (Fig. 2b).

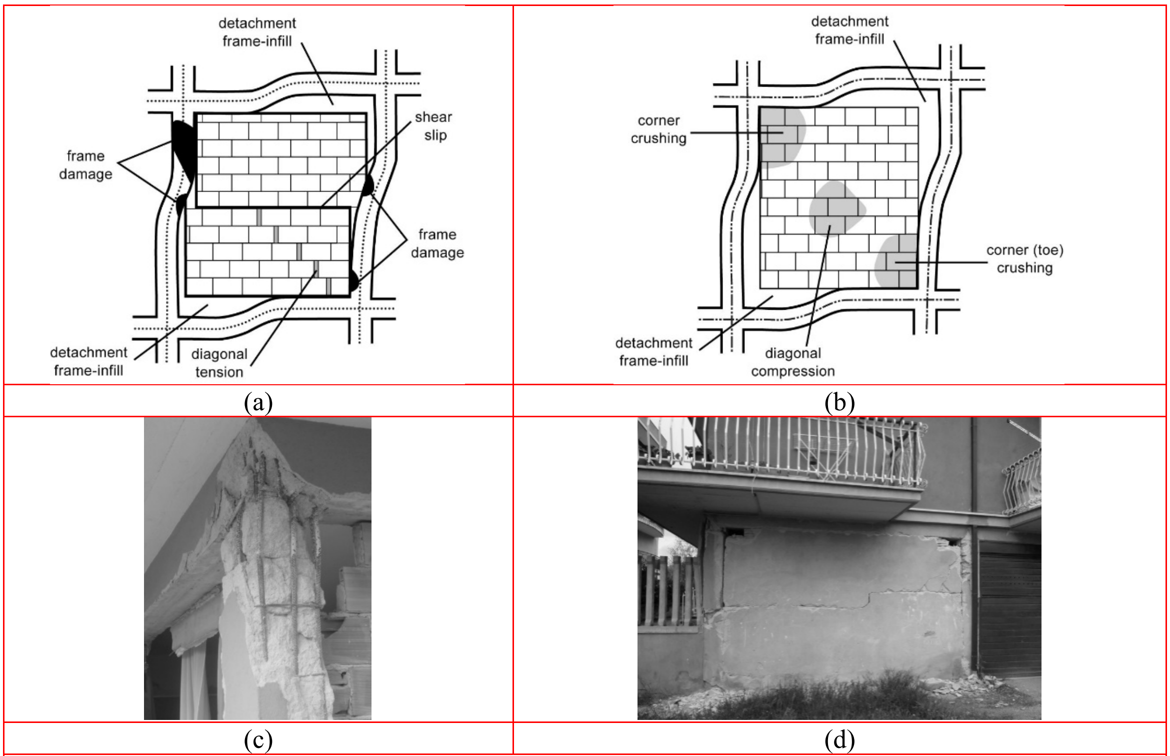


Fig. 2 Failure modes for infilled RC frames: (a) frame failure, sliding shear and diagonal cracking; (b) corner crushing and diagonal compression (adapted from Asteris *et al.* 2011) ; (c) example of frame failure; (d) example of sliding shear failure.

The above failure modes refer mostly to the in-plane response only. In order to maintain their in-plane load carrying capacity, the infills must not collapse due to out-of-plane forces. Out-of-plane collapse is a life safety limit condition. In new buildings, frames are stiffer and the infills (usually one-leaf) are very thick, thus the out-of-plane actions can be controlled. Angel *et al.* (1994) found that the out-of-plane strength reduction due to in-plane damage could be as high as 50% for panels with high slenderness ratios. This paper focuses on the in-plane behaviour of the infills. Work related to the interaction between in-plane and out-of-plane responses can be found in Angel *et al.* (1994) and Kadysiewskil and Mosalam (2009).

Mostly, design codes have long avoided considering the structural contribution of the infills, mainly because: a) the problem is not thoroughly understood, due to the high number of variables and uncertainties involved; and b) the infill behaviour and its interaction with the frame is strongly dependent on the frame geometry and the infill type, that varies considerably in different seismic areas (concrete blocks, hollow brick blocks, solid brick blocks, single or double skin infills, *etc.*). Some other codes, as EC 8 (EN 1998-1:2004, 2004), have introduced design principles for infilled RC frames for the design of new structures. For example, when irregularities in plan or elevation are seen in the structure due to the infills, some penalty factors are specified for the structure, while the design verifications of the entire structure is the same as for the bare frame (Fardis *et al.* 1999, Liberatore and Decanini 2011). Besides, some rules and recommendations are given to avoid local failure due to the interaction between the infill and columns, as short column effects or columns restrained by the infill just in one direction (corner columns). However, a recent study made by Liberatore and Decanini (2011) concludes that besides these recommendations, some nonlinear analyses are necessary to identify realistic damage patterns due to infill panels.

The influence of the infill panels on the frame performance may be either positive or negative, depending on a number of parameters such as: geometrical distribution of the infills in plan and elevation, strength variability, stiffness and ductility of the frames, infill aspect ratio, infill mechanical properties, distribution of openings and quality of the workmanship. All of the previous variables are presented in more detail in the next section that contains a brief discussion on the infill's influence on the overall behaviour of the frame. Major emphasis is done regarding works related to RC frames than steel ones. A review of major experimental tests (both monotonic and cyclic) follows, in order to illustrate the most critical aspects of the seismic behaviour of the infilled frames. In the third and central part of the paper, a state-of-the-art review of models for masonry infills used in frames is presented, especially for RC frames, with emphasis on frame modelling for nonlinear static and dynamic analyses.

2. Infill panel influence on the structural response of frames

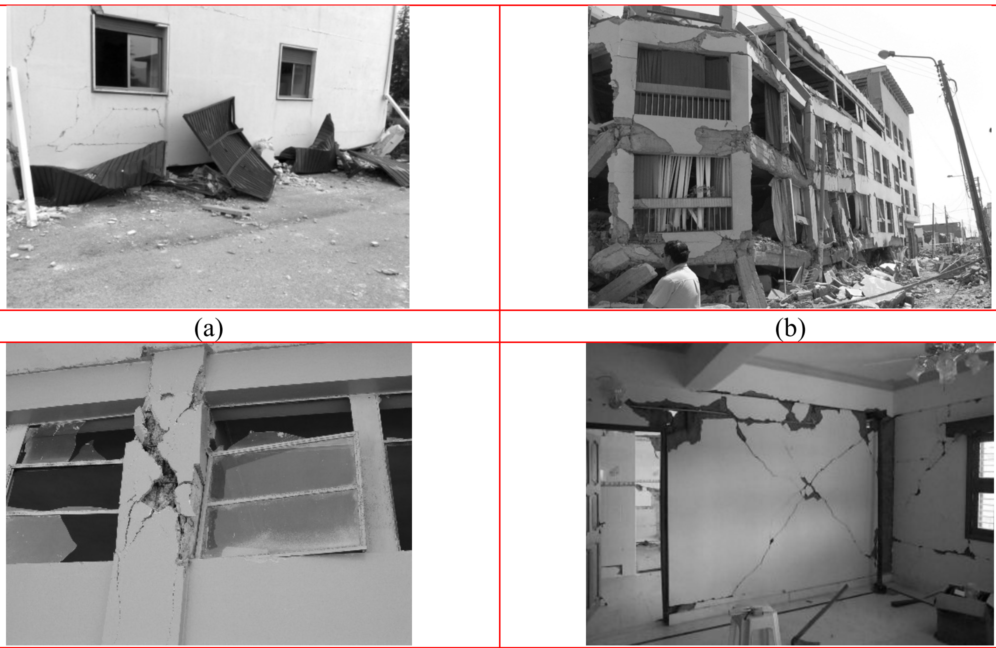
The in-plan and in-elevation geometrical distributions of the infill panels play a major role on the global response of structures subjected to lateral loads. A plan-irregular infill distribution may cause strong torsional effects, especially in the elastic domain, leading to larger-than-expected demands in the perimeter elements. Fardis *et al.* (1998) performed a shake table test on a single bay, two-storey, in-plan square RC frame structure with an eccentric arrangement of masonry infills. The frame was subjected to bidirectional ground accelerations. They clearly showed that the infills generated torsion on the structure. A common example of an in-elevation irregular distribution encountered all over the world is a frame structure with all storeys infilled, except for the base storey, typically used as commercial or parking space. Thias irregular configuration, when unaccounted for during design, may lead to well-known and too-often observed soft-storey mechanisms with large lateral drifts in the base columns, with the upper floors displacing predominantly as a rigid body (Fig. 3a, *e.g.* Liberatore *et al.* 2004). In other cases, when the frames are partially infilled with a stiff material or when the column height is partially restrained, a short column effect may take place. This configuration may trigger shear failures in the columns (Fig. 3b).

However, if the infills are well distributed and present in all storeys, those may provide most of the earthquake resistance and prevent collapse of relatively flexible and weak RC structures (Decanini *et al.* 2004, Fardis 2000, Kakaletsis and Kalayannis 2008). Infills give a significant contribution to the energy dissipation capacity, reducing the dissipation energy demands in frame elements and decreasing significantly the maximum displacements (Liberatore *et al.* 2004). It has experimentally seen that infills affect the response of the entire system basically through their strength and its correspond drift, but not always through their stiffness (Fardis 2000, Hashemi and Mosalam 2006, Griffith 2008, Baran and Sevil 2010). Kappos and Ellul (2000) shows that at serviceability level over the 95% of the energy dissipation is given by the infills, while at higher demand levels the those dissipate around 40% of the total energy, being the rest dissipated by the

RC frames. So, infills represent the first line of resistance under moderate and strong motions and should be considered in both analysis and design to avoid a brittle collapse.

Given the complexity of the frame-infill interaction, the overall lateral frame capacity cannot be computed as the mere sum of the frame and infill contributions (Shing and Mehrabi 2002). The relative stiffness and the relative strength between infill panels and columns govern the overall system failure sequence (Smith 1967). In general, if the panel is stiff with respect to the frame and the columns are not ductile, a shear failure in the frame may suddenly occur (Fig. 3b). On the other hand, when the frame elements are flexible, the panel infill is expected to fail, and the overall behaviour is ductile (Fig. 3c). In some cases infill panels work as shear walls, despite not being designed for this purpose, and prevent collapse of non-ductile concrete frames (Patel and Pindoria 2001).

A significant parameter affecting the panel behaviour is the panel aspect ratio, which influences the contact surface length between the frame and the panel and the inclination angle of the panel truss mechanism (Özgür and Sinan 2007). The contact length influences the panel truss width participating in the strut mechanism, while the angle of inclination of the truss affects the panel capacity. The presence, size and position of the openings lead to a reduction of the panel stiffness and strength (Syrmakezis and Asteris 2001, Mondal and Jain 2008, Mohebkhah *et al*. 2007, Papia and Cavaleri 2001, Asteris 2003, Fiorato *et al*. 1970) and the load pattern within the panel is modified. Mosalam *et al.* (1998) identified strut-and-tie models that can reproduce the behaviour of panels with openings. However, predicting the strength of an infill with openings by means of simplified trusses remains a difficult task.



(a) (b)

(c) (d)

Fig. 3 Post-earthquake damage in infilled RC frames: soft-storey mechanism in (a) 2009 L'Aquila, Italy and (b) 2007 Pisco, Peru earthquakes; (c) short column effect in 2007 Pisco, Peru earthquake; (d) diagonal cracking at infill panels in 2001 Bhuj, India earthquake (Patel and Pindoria 2001).

Workmanship also plays a major role in the response of structures, because precision and practice in the execution of the infill panels influence the mortar layer strength and the boundary conditions in the frame-panel contact zones.

Based on the experience from past earthquakes and years of research on the influence of the infills on the response of RC and steel frames, it is clear that both the stiffness and strength of the infills should be accounted for in the seismic analysis and design of new buildings (EN 1998-1:2004) and, more importantly, in the seismic vulnerability assessment of existing buildings. However, there are several variables/parameters that must be taken into consideration to properly describe the frame-infill interaction and the ensuing structural failures (Negro and Colombo 1997).

3. Brief review of experimental tests on frame-infill interaction

This section briefly reviews some of the main results of several years of experimental campaigns which have been carried out all over the world. Available test results mainly differ in the number of storeys and bays, in the scale of the tested specimens (full or reduced), in the load application scheme and in the boundary conditions. The experimental tests reported in the published literature are generally designed to have an infill diagonal cracking failure. Since the focus of this paper is on modelling, only a few, major tests on one-bay one storey and multi-bay multi-storey structures are reviewed. Three main test types are listed as: quasi-static (Mosalam *et al*. 1997, Amato *et al*. 2008, Kakaletsis and Karayannis 2008, Personeni *et al.* 2008, Baran and Sevil 2010), pseudo-dynamic (Negro and Verzelletti 1996, Mosalam *et al.* 1998, Dolsek and Fajfar 2002) and full dynamic testing (Fardis *et al.* 1999, Albanesi *et al*. 2008, Liu *et al*. 2011).

For monotonic tests on a one-bay single-storey masonry infilled RC frame designed to have infill rather frame failure, it is experimentally observed that as the lateral load increases and the deformation demand becomes large, diffuse cracking evolves starting from the middle of the panel and progresses along the tensile stress principal directions. As the lateral load increases, the shear deformation in the panel and the flexural deformation in the frames cause the detachment of the panel from the frame along part of its height, so the frame and the panel tend to separate except for the loaded contact zones at the infill corners of the compression diagonal (as schematically shown in Fig. 2a). In these corners, a biaxial compression stress state develops and the portion of the panel around the compressed diagonal works as a bracing element. At failure, the panel is strongly damaged by the presence of major cracks and the strength of the system relies on the frame structure only. If the frame is “weaker” than the infill, then plastic hinges develop in the columns before infill failure. After the panel fails, the lateral force – lateral displacement curve is characterized by a descending branch up to reach the residual strength and the response is basically controlled by the frame (Fig. 1c).

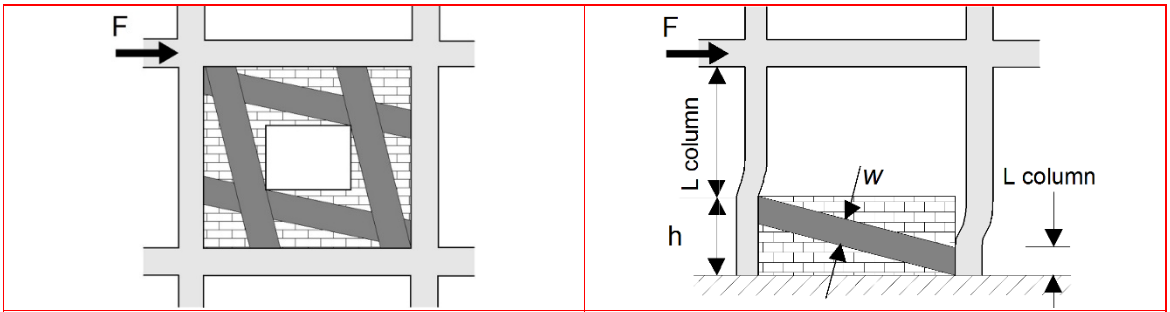
The above failure sequence takes place unless a column shear failure occurs first: when the panel is very stiff and the columns are not ductile, a sudden column shear failure occurs. Following the column shear failure, the frame-strength mainly relies on the panel and on the remaining, undamaged columns, and the strength at large displacements may be lower than the peak bare frame strength. A different behaviour occurs when a mortar layer fails in shear, developing a sliding surface, which divides the panel into two or more parallel parts, due to the low mechanical properties of the mortar. The lateral force is not only transferred by compression between the two loaded corners, but other compressive fields develop between the loaded corners and the opposite columns (Crisafulli 1997).

For cyclic loading of single bay frames, it is observed that the secant stiffness of the system, observed on a total lateral force – lateral displacement curve, rapidly degrades with the deterioration of the interfaces (contact zones) between panel and frame and with the crack spreading within the panel (Mallick and Servern 1967, Klingner and Bertero 1978, Amato *et al*. 2008). Kakaletsis and Karayannis (2008) tested seven 1/3 scale, single story, single bay infilled frames; the analysed parameters were the influence of the opening shape and the infill compressive strength on the lateral strength. They concluded that although to have opening infills, those can significantly improve the performance of RC frames, especially in terms of energy dissipation.

Al-Chaar (1998) evaluated the in-plane load capacity of five half-scale single storey models with different number of bays (3 single-bay frames, 1 double-bay frame, and 1 triple-bay frame). The results show that the number of bays influences the failure mode and the shear stress distribution. For example, for the double-bay frame the failure mechanism started with the formation of two hinges on the top of the windward and centre columns, following by semi diagonal cracking in the infill and shear cracking at the base of the leeward column. Fiorato *et al.* (1970) evaluated the vertical load influence on the infilled RC frame response. They tested 8 one-bay one-storey panels, 13 one-bay five-storey panels and 6 one-bay two-storey panels under static lateral in-plane loading. They observed that the vertical loads increase the frame lateral strength and stiffness due to the stiffening and strengthening of the columns as well as to the increase of the shear capacity of the infill. Furthermore, the response of the five-storey specimens indicates that as the structure becomes taller, it tends to behave as a slender wall, with flexural cracking developing at the base of the two base columns. As for tests on multi-bay multi-storey frames (Negro *et al.* 1995, Colombo *et al.* 1998, Fardis *et al.* 1998, Mosalam *et al.* 1998), it was confirmed that the vertical loads significantly influence the contribution of the panel to the lateral response, especially in the lower floors where the vertical loads are larger. The combination or larger gravity loads and larger shear forces in the lower storeys leads to more evident truss mechanisms in the lower floor panels.

The effect of the openings is investigated in Fiorato *et al.* (1970), Syrmakezis and Asteris (2001), Asteris (2003), Al-Chaar *et al.* (2003), Kakaletsis and Karayannis (2008), Smyrou *et al*. (2011), Mohammadi and Nikfar (2013). The general conclusion is that the openings reduce the lateral strength of the infilled frames and change the load path within the infill panel (Mondal and Jain 2008). However, the strength reduction of the overall system is not proportional to the area of the openings and is not easy to measure or predict. According to Kakaletsis and Karayannis (2008), infill with openings are cracking and separating from the surrounding frames at early stage before yielding occurs at the column reinforcement.

The opening position influences the strength of the infilled frame system too (Al-Chaar 2002). In the case of central openings, depending on the size of the opening itself, the corners might still develop a strut mechanism, or the forces in the panel are conveyed to the frame through four struts placed in the horizontal and vertical bands around the openings (Fig. 4). When the frames are partially infilled (Fig. 4b) the panel still reacts with a truss mechanism that causes a short column effect that may lead to brittle shear failure.



(a) (b)

Fig. 4 (a) Typical strut-and-tie approach for panels with central openings and (b) for partially infilled frames (modified from Al-Chaar 2002).

4. Numerical modelling techniques

The high cost of experimental campaigns and the need for accounting for the infills' contribution to the overall frame response led to the development of several numerical procedures, which are particularly important for the seismic assessment and retrofit of existing RC buildings. Three main approaches are identified for the infill models and in general immerse in the finite element method: micro-modelling, meso-modelling and macro-modelling. The first considers the detailed micro-modelling and the simplified micro-modelling; between the micro and macro modelling another technique level called meso-modelling could be considered.

All the above approaches mainly differ in the degree of modelling detail of the infill panel, more specifically as (Lourenço 1996):

- Detailed micro modelling. Bricks and mortar joints are discretized using continuum (smeared) elements, with the brick-mortar interface represented by discontinuous elements.
- Simplified micro-modelling. The bricks are modelled as continuum elements, while the behaviour of the mortar joints and of the brick-mortar interface are lumped in discontinuous elements. The mortar failure and separation between bricks can be numerically seen as in Fig. 5a.
- Meso-modelling. Bricks, mortar and brick-mortar interface are smeared out and the masonry is treated as a continuum; which means a new equivalent material representing all the infill panel obtained by a homogenization process. Contact, gap or spring elements can be considered for modelling the interface infill-frame and these elements will allow the separation of the infill from the bare frame as seen in Fig. 5b.
- Macro-modelling. It refers to analyses that use frame elements and typically takes the infill presence into consideration through equivalent strut models (Fig. 5c, 5d). This approach – faster and easier to apply with today's computational tools and speeds - is of greater interest for designers and engineers.

For the first two cases described above, the bare frame can be modelled considering the concrete and steel reinforcement as different elements; while in the last the bare frame is modelled with beam elements placing lumped plasticity at probable locations or using distributed fiber models. In the continuum models, the contact between bare frame and infill panel is usually modelled with an interface material. In this manuscript, more emphasis is given to the macro-modelling and simplified micro-modelling since those are the most common approaches used.

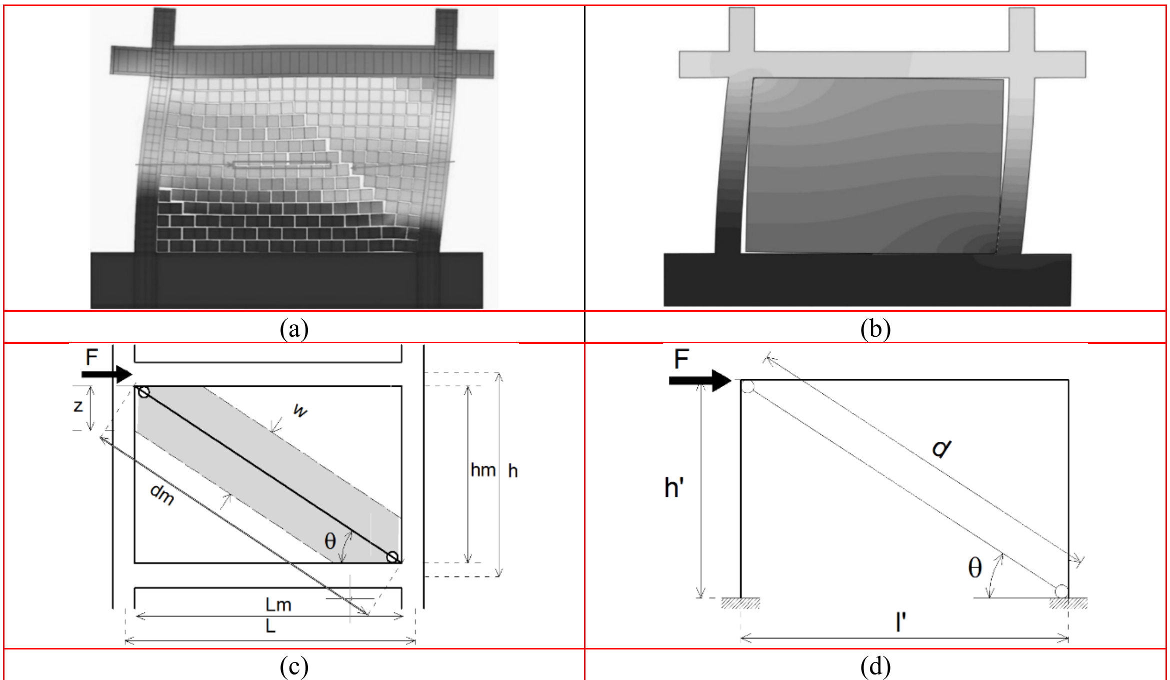


Fig. 5 Pushover finite element models for infill panels: (a) simplified micro model (adapted from Sattar 2013); (b) meso-modelling; (c) and (d) macro models, adapted from Crisafulli *et al*. 2000 and Amato *et al*. 2008, respectively.

5. Macro-modelling

Several simplified models are proposed to reproduce the major aspects of the infill-frame interaction mechanisms. Polyakov (1960) first observed that the panel could be considered as a bracing diagonal. Here the panel was represented by a diagonal strut element that took into account the lateral stiffness (a summary of the geometrical properties of the struts are shown in Table 1), and eventually the strength and post-peak behaviour of the panel (Fig. 5c, 5d).

For monotonic loads, one strut in the compression diagonal is needed, while two struts along the two diagonals are typically used for cyclic loads. The proposed models vary from single to multiple struts, from concentric to eccentric, from linear elastic to non-linear hysteretic. The diagonal strut is usually connected to the intersection points of the beam and column centrelines, which implies that the numerical strut length is greater than the physical diagonal infill length; however, according to Galli (2006) this length increment does not affect the reliability of the seismic response evaluation.

In nonlinear frame analysis, beams and columns are represented by line elements that take into account the flexural and possibly shear response through either lumped plasticity (spring elements) or distributed plasticity models. If the strut is connected to the beam-column intersection, there is no direct interaction between the strut and the shear response of the column (Crisafulli 1997). Multiple strut models have been proposed to at least partially address this issue (see Fig. 8). When openings are present, the problem complicates further.

For the strut geometric properties the cross area is typically given by the panel thickness times an equivalent width, *w*. The length of the strut, *d*, is given by the length of diagonal of the panel (Fig. 5c, 5d). The width can be computed considering the relative stiffness between the infill and the frames or indirectly evaluating the contact length between them. Recently, the influence of vertical loads has been incorporated into the width evaluation (Amato *et al*. 2008).

Additionally Kadysiewskil and Mosalam (2009) developed a model that combines the in-plane and out-of-plane effects for nonlinear dynamic analyses. For each infill panel, representing a single bay in a single storey, the model consists of one diagonal member as explained before. The difference is that the member is composed of two beam-column elements, joined at the midpoint node that must be assigned a lumped mass in the out-of-plane direction to account for the out-of-plane inertia forces (Fig. 6). The beam-column elements of the diagonal equivalent strut are force-based elements with the inelastic behaviour concentrated in the hinge regions near the midspan node. These regions (with total length equal to 1/10 of the strut length) are modelled with inelastic fibers. Elastic sections with low moment of inertia are assigned to the end of the beam-column elements attached to the surrounding frame. This model was calibrated so that a single diagonal strut can be used for both monotonic and cyclic loads.

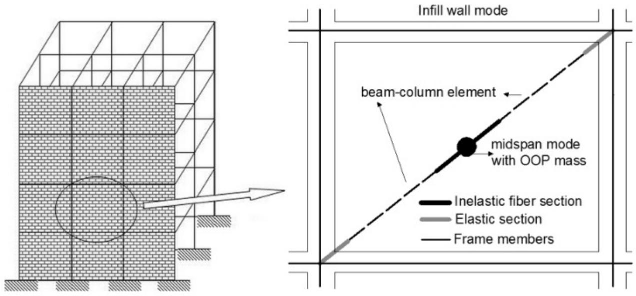


Fig. 6 Infill model by Kadysiewskil and Mosalam (2009), comprising a beam-column elements with fiber discretization (adapted from Kadysiewskil and Mosalam 2009).

The main issue in the strut modelling technique is the determination of the single strut width *w*. In several instances, the width *w* is computed considering the relative stiffness and/or the contact length between panel and frame. The typical equations to compute the equivalent width are presented in Table 1 in chronological order.

Table 1 Summary of expressions proposed to compute *w/d*

Author (year)	Equation	Observation
Holmes (1961)	$w/d = 1/3$	$\lambda_h < 2$,
Smith (1967)	$0.10 < w/d < 0.25$	The value graphically depends on λ_h
Mainstone (1971)	$w/d = 0.16\, \lambda_h^{-0.3}$	For λ_h see Eq. (1)
Mainstone (1974)	$w/d = 0.17\, \lambda_h^{-0.4}$	Adopted by FEMA-274 (1997), FEMA-306 (1998)
Bazan & Meli (1980)	$w = (0.35 + 0.022\beta)h_m$	$0.9 \leq \beta \leq 11$ For β see Eq. (3)

Table 2 (Continuation) Summary of expressions proposed to compute *w/d*

Author (year)	Equation	Observation
Hendry (1981)	$w = \frac{1}{2}\sqrt{z_b^2 + z_c^2}$	For z_b and z_c see Eq. (5)
Tassios (1984)	$w/d = 0.20\beta \cdot \sin\theta$	$1 \leq \beta \leq 5$
Liauw & Kwan (1984)	$w/d = \frac{0.95 sin2\theta}{2\sqrt{\lambda_h}}$	$25° \leq \theta \leq 50°$
Decanini & Fantin (1987) For uncracked panels	$w/d = 0.085 + \frac{0.748}{\lambda_h}$	$\lambda_h \leq 7.85$
	$w/d = 0.130 + \frac{0.393}{\lambda_h}$	$\lambda_h > 7.85$
Decanini & Fantin (1987) For cracked panels	$w/d = 0.010 + \frac{0.707}{\lambda_h}$	$\lambda_h \leq 7.85$
	$w/d = 0.040 + \frac{0.470}{\lambda_h}$	$\lambda_h > 7.85$
Paulay & Priestley (1992)	$w/d = 1/4$	$\lambda_h < 4$
Durrani & Luo (1994)	$w/d = \gamma . sin2\theta$	$\gamma = 0.32\sqrt{sin2\theta}\left(\frac{h^4 E_m t}{m E_c I_c h_m}\right)^{-0.1}$ $m = 6\left(1 + \frac{6 E_b I_b h}{\pi E_c I_c L}\right)$
Flanagan & Bennet (1999)	$w = \frac{\pi}{C \cdot \lambda_h cos\theta}$	C is an empirical value dependent on the in-plane drift displacement
Cavaleri *et al.* (2005) Amato *et al.* (2008)	$w/d = \frac{k}{z}\frac{c}{(\lambda^*)^\beta}$	For λ^* see Eq (6) c and β are coefficients that takes into account the Poisson module, k takes into account the vertical load and z is a geometrical parameter.

The relative stiffness between the infill and the column may be evaluated through the dimensionless parameter λ_h, first proposed by Smith (1967):

$$\lambda_h = h\left[\frac{E_m \cdot t \cdot \sin 2\theta}{4E_c \cdot I_c \cdot h_m}\right]^{1/4} \tag{1}$$

where t and h_m are the thickness and height of the infill panel, respectively; E_m and E_c are the masonry and concrete moduli of elasticity, respectively; θ is the inclination of the panel diagonal; I_c is the column moment of inertia and h is the column height to the beam centrelines. λ_h decreases as the column becomes stiffer than the masonry panel.

Furthermore, Smith (1967) proposed an expression to compute the contact length z between panel and frame, see Fig. 5c and 8. It stems from the analogy between the panel-frame contact problem and beam on elastic foundation subjected to a concentrated load:

$$z = \frac{\pi}{2\lambda_h} h \tag{2}$$

Alternatively, the relative stiffness may be evaluated using the following expression proposed by Bazan and Meli (1980), that define the dimensionless parameter β:

$$\beta = \frac{E_c \cdot A_c}{G_m \cdot A_m} \tag{3}$$

where A_c is the column gross area, A_m is the area of the masonry panel in the horizontal plane (length times thickness), and G_m is the panel shear modulus. The following limitations were proposed: $0.9 \leq \beta \leq 11$ and $0.75 \leq l_m/h_m \leq 2.5$

Hendry (1981) evaluated the relative stiffness (λ_b and λ_c) and contact lengths (Z_b and Z_c) considering separately the beam and column stiffness (units are in inches and ksi):

$$\lambda_b = \left[\frac{E_m \cdot t \cdot \sin 2\theta}{4E_c \cdot I_b \cdot h_m} \right]^{1/4} \tag{4a}$$

$$\lambda_c = \left[\frac{E_m \cdot t \cdot \sin 2\theta}{4E_c \cdot I_c \cdot h_m} \right]^{1/4} \tag{4b}$$

$$z_b = \frac{\pi}{2\lambda_b} \tag{5a}$$

$$z_c = \frac{\pi}{2\lambda_c} \tag{5b}$$

A more recent relative stiffness parameter λ^* was introduced by Papia and Cavaleri (2001) :

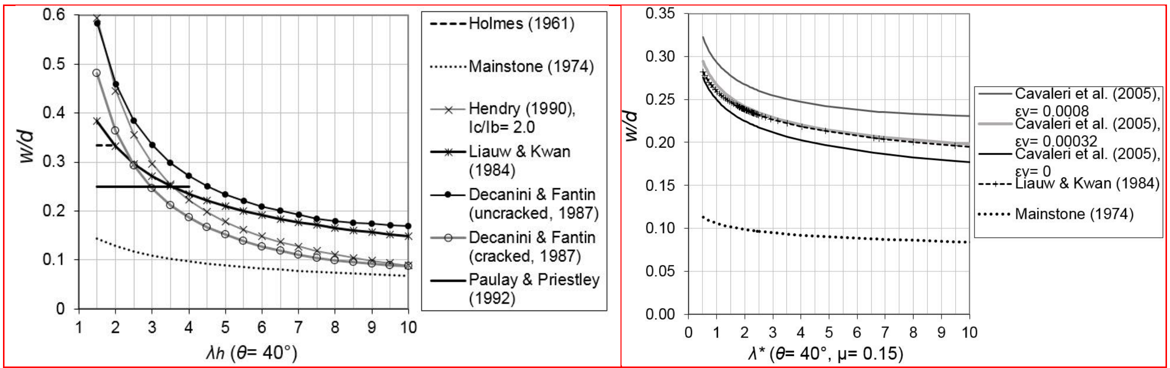
$$\lambda^* = \frac{E_m}{E_c} \frac{t \cdot h'}{A_c} \left(\frac{h'^2}{l'^2} + \frac{1}{4} \frac{A_c}{A_b} \frac{l'}{h'} \right) \tag{6}$$

where l' is length between the column centrelines, A_b is the beam gross area, E_m is the elasticity modulus of the masonry, and h' is the height between the base and the beam centreline (see Fig. 5d).

More recently, the vertical load influence on the contact length ratio was experimentally and numerically investigated by Cavaleri *et al.* (2005) and Amato *et al.* (2008). They found that the bounding effect of the surrounding frames is improved by the vertical load increment. Cavaleri's formulation is the only expression that explicitly includes the vertical load influence.

Some of the expressions reported in Table 1, and related to λ_h, are plotted in Fig. 7a, which shows how the *w/d* ratio varies with respect to the relative stiffness. Formulas containing constant values for the *w/d* ratio are not always adequate, but they are often used for their simplicity. For example, Holmes (1961) and Paulay and Pristley (1992) proposed constant values for *w/d* of 1/3 and 1/4, respectively. The first is limited for λ_h less than 2 and the second for λ_h less than 4. For values of λ_h less than 2, Hendry and Decanini's formulas give high values of *w/d*. The advantage of Decanini's formula is the reduction of the equivalent width if a cracked section of the masonry is considered. In all cases a reduction of *w/d* is seen when λ_h increases; which means a panel stiffer than the frame. Fig. 7b shows a comparison between the *w/d* ratio reported by Cavaleri *et al.* (2005), Mainstone (1974) and Liauw and Kwan (1984). Due to the different formulas to compute λ_h and λ^*

a similar procedure as in Papia *et al.* (2003) is followed: the geometry of the infilled frame is kept constant while the material properties are varied to compute a range values of λ_h and λ^*; then, the corresponding values of *w/d* are computed for Mainstone and Liauw and Kwan's formulas, and these values are plotted versus λ^*. According to Papia *et al.* (2003), Mainstone's formula gives low values because it does not take into account some parameters which could substantially change the lateral response on an infilled frame, and it underestimates the stiffening effect of the infill. Liauw and Kwan's expression gives closer values as Cavaleri's one if this last considers ε_v= 0.00032.



(a) (b)

Fig. 7 Variation of the *w/d* ratio with respect to the relative stiffness (a) λ_h and (b) λ^*.

In all strut models, the nonlinear strut behaviour is described by constitutive laws, which represent the initial stiffness, the strength and the panel post-peak behaviour in compression. For monotonic loading, a single strut in compression can be used. However, for cyclic loading the equivalent struts are usually placed in both directions (see Fig. 8). Only the model proposed in Kadysiewskil and Mosalam (2009) uses a single strut for cyclic loading. The strut constitutive laws should be different in compression and tension: in some cases, the tensile strength is totally neglected. Advanced modes, such as those in Crisafulli and Carr (2007) and Smyrou *et al*. (2011), incorporate hysteretic laws that represent the shear behaviour in the diagonal strut model too.

5.1 Single vs multiple strut models

The simplest model to represent the structural behaviour of the panel is a diagonal concentric strut, placed in one or both diagonal directions (see Fig. 5c, 5d). The use of a single concentric strut element in each diagonal direction has, however, some disadvantages, such as the lack of strut-column interaction, that in some cases may lead to column shear failure (Crisafulli *et al*. 2000), thus the idea to model the infill panel through a series of off-diagonal equivalent struts in each direction (see Fig. 8). Mosalam *et al*. (1998), for example, noted during a pseudo dynamic test on a two-storey two-bay infill steel structure that the section of maximum bending moment was shifted away from the beam-to-column connections. They concluded that the use of off-diagonal strut elements could represent these phenomena. Crisafulli *et al*. (2000) specified that using a single diagonal concentric strut model the local effects from the interaction between the infill panel and the surrounding frame cannot be well captured (*i.e.* location of potential plastic hinges).

Thiruvengadam (1985) used a multiple strut model made of pin-jointed diagonal and vertical trusses oriented in both directions; hence, both shear and vertical stiffness of the panel are accounted for. He divided the infill in subpanels; each subpanel was represented by diagonal equivalent struts in both directions (Fig. 8a). The area of each equivalent strut was computed based on the geometrical characteristics of each subpanel according to the following equation:

$$A_d = \frac{a^2 \cdot t}{4b(1+\nu)\cos^3\theta} \tag{7}$$

where A_d is the strut cross section area, a is the length of the subpanel, b is the height of the subpanel, t is the wall thickness, v is Poisson modulus and θ is the angle related to a and b (see Fig. 8a).

Mochizuki (1988) used multiple struts elements for representing slip failure in the infill; however, no clear information on the equivalent area calculations was given. Hamburger and Chakradeo (1993) studied steel-frame buildings with masonry infill, and gave special attention to the beam-column joint. They proposed the use of equivalent diagonal struts placed next to the openings (*i.e.* one infill wall with one opening, thus two struts are used). The struts should be tangent to the corner of the window opening. Looking at the results of the numerical model they recommended that the width of each equivalent strut should not exceed twice the infill thickness. Syrmakezis and Vratsanou (1986), who focused on the importance of the panel-frame contact length, used a five parallel compressive strut model. The width of each strut is proportional to the total width. For infills with *h/l* ratio between 0.50 and 2.00 an empirical equation was proposed to evaluate the total width of the compressive zone:

$$\frac{w}{h} = 0.64\frac{l}{h} + 3\left(\frac{d_c}{l} - 0.1\right) \tag{8}$$

where h and l are the height and length of the column and beam, respectively, measured at the centrelines; and d_c is the depth of the column parallel to the infill. They concluded that the number of struts is not very important for computing displacements but influences the bending moment distributions in the frame.

Chrysostomou *et al.* (2002) proposed a three parallel diagonal strut model for each direction to simulate the response of frames under earthquake loading. It is assumed that each strut works in compression only. One strut was placed along the beam-column connection nodes, and as in the previous cases, the other two were placed in critical points of the frame where a plastic deformation may occur (Fig. 8b). El-Dakhakhni *et al.* (2003) proposed a three-strut model for masonry-infilled steel frames, with a concentric diagonal element, and two off-diagonal parallel trusses intersecting the steel frame elements (Fig. 8c), where the contact length values take into account the plastic moment capacities. The total area, A, of the equivalent struts depends on the column contact length (α_c) and geometrical parameters of the infill.

$$A = \frac{(1-\alpha_c)\alpha_c \cdot h \cdot t}{\cos\theta} \tag{9}$$

The area of the central strut is *A/2*, and the area of the other two off-diagonal struts is *A/4*. The model can take into account the orthotropic behaviour of masonry panels and can predict the stiffness and the ultimate load capacity of the infill (Chrysostomou and Asteris 2012).

Using Mainstone (1974) strut width formula, Al-Chaar (2002) proposed to place the only diagonal strut in an eccentric manner with its nodes lying on the columns at the same distance as the contact length, where a column failure may be expected to occur due to shear. The model also considered a reduction factor due to existing damage of the infills or the presence of openings and the influence of out-of-plane behaviour on in-plane resisting capacity.

Crisafulli (1997), Crisafulli and Carr (2007) and Smyrou *et al*. (2011) formulated a strut model based on a four node element, which represented the infill panel through two parallel off-diagonal struts and a shear connector capable of accounting for the diagonal tension failure and the shear failure along the mortar joints (Fig. 8d). In their model, the width of each equivalent strut was half that of the single strut model. The single strut model width was between 10% and 40% of the diagonal panel length and can be computed by the equations reported in Table 1. The vertical separation of struts (h_z in Fig. 8d) varies between $z/3$ and $z/2$, being z the contact length computed by Eq. (2). Due to the contact length reduction between frame and infill, and to the masonry infill cracking, the area of the struts decreases as the strut axial displacement increases. According to results reported by Decanini and Fantin (1987) the equivalent strut width can decrease by about 20% to 50% due to cracking of the masonry panel.

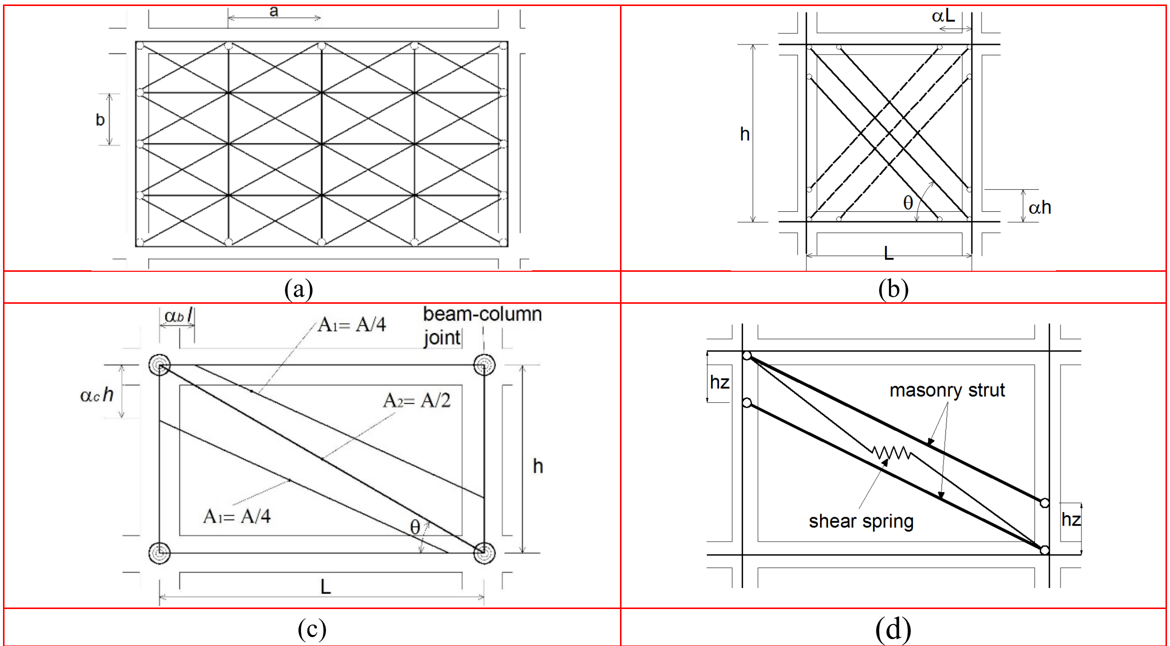
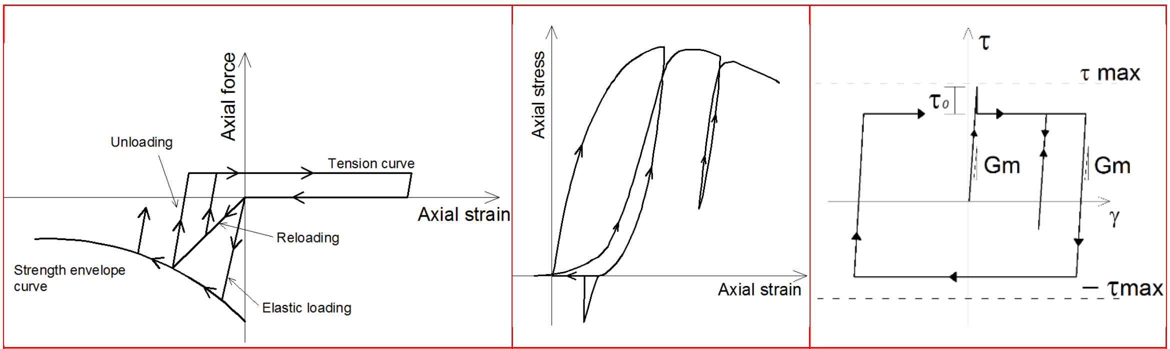


Fig. 8 Multiple equivalent (off-diagonal) strut models used by (a) Thiruvengadam (1985); (b) Chrysostomou *et al.* (2002), (Figure adapted from Asteris *et al*. 2011); (c) El-Dakhakhni *et al*. (2003); (d) Crisafulli and Carr (2007) - for simplicity, only struts in one direction are shown in (c) and (d).

5.2. Strut constitutive models

Some of the nonlinear uniaxial stress-strain relationships available in the published literature may be applied to any brittle material (such as the masonry infills), others are specifically proposed for the infills (*e.g.* Madan *et al.* 1997, Varum *et al.* 2005). The hysteretic models specifically proposed for the equivalent struts are characterized by three main parameters; the initial stiffness, the strength and a hardening/softening branch. In the case of reversal loading some stiffness and strength degradation may be expected as the strut element oscillates from compression to tension and vice-versa. Several axial force-axial displacement (or stress-strain) relationships have been proposed for the equivalent strut models. They are typically calibrated with respect to available experimental data. For example, Klingner and Bertero (1978) proposed a model based on a dynamic test of an infilled frame. In Fig. 9a, the model is represented in an axial force-axial deformation relation, where the compression strength envelope curve exhibits an exponential degradation beyond the peak strength. Even though the model assumes tension strength for the masonry panel, Klingner and Bertero did not include this contribution into their numerical analyses.

More recently, Crisafulli (1997), Crisafulli and Carr (2007) proposed a cyclic model that takes into account the non-linear response of the masonry in compression, tension and shear (Fig. 8c, 9b, 9c). The hysteretic response of the shear spring is modelled following an elasto-plastic rule with variable shear strength. Also, due to the shortening of the contact length, this model considers a progressive reduction of the strut area as the lateral load increases. Combescure and Pegon (2000) proposed a hysteretic law for strut elements with no tension strength, this model accounts for the compressive strength degradation under cyclic loading. The phenomena reproduced are the stiffness degradation due to cracking (mainly at the interface between the frame and the panel), the development of plastic strains, the softening due to crushing, the strength degradation under cyclic loading and the pinching associated with sliding. Kadysiewskil and Mosalam (2009) considered a simplified elasto-plastic behaviour of the infill panel for both tension and compression (with different yield forces in the two loading directions and a small post-yield stiffness inserted to minimize convergence problems). Their model considers both the in-plane and out-of-plane response of the infill, as well as the interaction between in-plane and out-of-plane load bearing capacities.



(a) (b) (c)

Fig. 9 Examples of hysteretic curves proposed for the equivalent struts representing masonry panels: (a) Klingner and Bertero (1978); (b) compression/tension and (c) shear cyclic response proposed by Crisafulli (1997).

6. Micro-modelling and meso-modelling

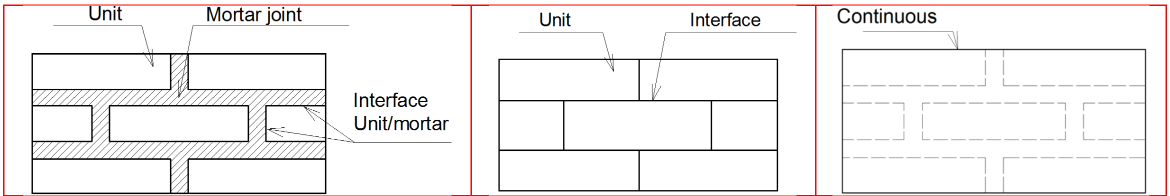
Over the last two decades, more powerful computational platforms and faster computers have eased the development of refined micro-models. This term properly refers to the representation of each single element belonging to the system, such as the bricks and the mortar layers of the masonry infill panel or the concrete and reinforcement of the RC frame (*e.g.* detailed micro-modelling). Typically, such a detailed discretization is obtained by means of the finite element method (FEM) or the discrete element method (DEM). When compared to the previously described macro-model approach, the computational time drastically increases and problems related to lack of numerical convergence are more often encountered.

Micro-modelling techniques (including meso-modelling, see Fig. 10c) are in general more precise than macro-models and can trace several possible failure mechanisms (for example, they can explicitly compute the contact zones between infill and frames and their evolution during a time history analysis). However, these techniques require calibration of a high number of parameters for the material constitutive laws and proficiency in the use of finite element procedures, hence resulting today more suitable for research purposes or calibration of simplified strut models.

The most relevant components of the micro and meso-models are the infill panel, the frame-infill interface and the RC bare; they are discussed separately in the following paragraphs and a summary of the relevant papers is showed in Table 2.

6.1. Infill panel

The brick units and the mortar can be modelled separately with a smeared formulation (Fig. 10a, detailed micro-modelling) but its application is for small test specimens and structural details (Asteris *et al.* 2013) due to the complication on the numerical convergence and computation time. Alternatively, the bricks are modelled using a smeared formulation and the mortar is represented by discrete interfaces without thickness (simplified micro-modelling). In this latter case, the dimensions of the masonry bricks are modified to maintain the same overall dimensions of the masonry assembly (Fig. 10b). In some cases, when brick failure is deemed unlikely, the brick units can be assumed as elastic in order to reduce the computational effort and to limit convergence problems. Vertical interfaces can be introduced in the brick middle to reproduce its possible tensile failure (see Fig. 12b). In case the nonlinear response of the brick is modelled through a smeared approach, appropriate constitutive laws must be used, which include failure criteria, hardening/softening rules, biaxial interaction, etc. Fig. 11 shows representative constitutive diagrams for quasi-brittle materials (Lourenço 1996).

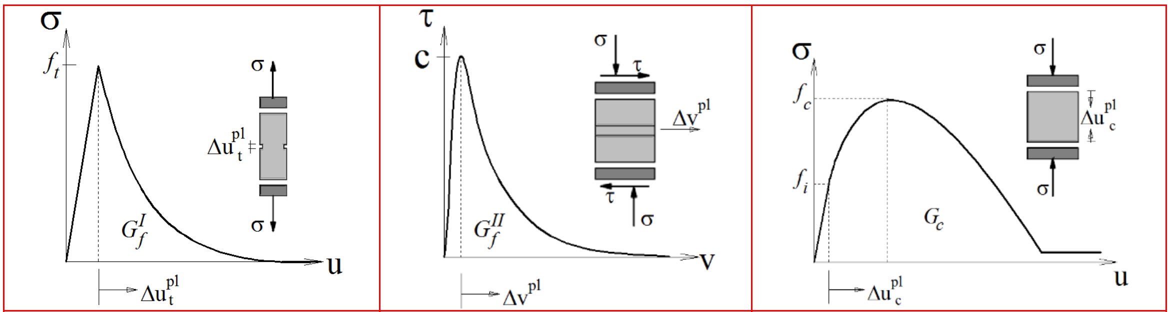


(a) (b) (c)

Fig. 10 Micro-model approach for the infill masonry panel; a) detailed micro-model; b) simplified micro-model with interface elements; c) meso-modelling.

Within the simplified micro-modelling approach, Rivero and Walker (1984) proposed to model the infill as a mesh of uncracked, homogeneous, isotropic and elastic triangular elements, combined with gap and joint elements for the infill-panel contact. The gap elements represent the space between the frame and the infill, while the joint elements simulate the boundary of continuity between the uncracked parts, and between the panel and the frame. Lotfi and Shing (1994) later proposed zero-thickness interface elements to represent the mortar joints. Mehrabi and Shing (1997) proposed a model that accounts for the joint closing and plastic compaction of the mortar layers.

Stevens and Liu (1992) proposed a constitutive model for plain concrete combining continuum damage mechanics with a strain-based plasticity. Lourenço (1996) developed an interface failure criterion for masonry characterized by a tension cut-off, Coulomb friction law and a compression cap model: the masonry assembly includes cracking, crushing and slip using the constitutive laws showed in Fig. 11. Carol *et al.* (1997) developed a general model for normal and shear cracking referred to the average plane of the crack. Guinea *et al.* (2000) presented a micro-mechanical model for analysing Mode-I fracture of brick masonry. The analysis was based on a detailed modelling of brick and mortar fracture by means of the fictitious (or cohesive) crack model. Alfaiate *et al.* (2005) studied mixed-mode crack propagation in concrete and masonry using different softening criteria and different mode-II fracture energy and cohesion. They found that the amount of shear stress in the discontinuities is the most significant factor affecting the structural behaviour. Attard *et al.* (2007) extended a finite element procedure that accounts for fracture in concrete to the simulation of tensile and shear fracture in masonry. They adopted a Mohr-Coulomb failure surface with a tension cut-off.



(a) (b) (c)

Fig. 11 Stress-displacement diagrams for quasi brittle materials, (a) tensile behaviour for mortar joints, (b) shear behaviour for mortar joints, (c) compressive behaviour of brick and mortar joints.

Al-Chaar and Mehrabi (2008) ran an extensive numerical campaign with the following models for the infilled frame components: the smeared-crack finite element formulation developed by Lotfi and Shing (1991) for the representation of the RC frame elements; the interface model developed by Mehrabi and Shing (1997) for the mortar layers and the panel-frame contact; truss elements for the longitudinal reinforcing; and elastic shell elements for the bricks. Perfect bond was assumed between concrete and reinforcement, and interface elements were introduced in the frame where a shear failure was expected. This complex model is able to accurately reproduce different failure modes.

As for the meso-modelling, which considers a unique, smeared material for the brick and mortar, Del Piero (1989) modelled masonry infill considering a homogeneous isotropic continuum medium with no tensile strength. Dhanasekar and Page (1986) proposed a nonlinear orthotropic model for brick masonry infills. Han and Chen (1987) developed a short-term, time-independent constitutive model for progressive failure analysis of infill panels. It included both the William and Warnke (1974) and the Hsieh *et al.* (1988) failure surfaces, nonuniform hardening, a nonassociated flow rule with changing dilatancy factor, linear softening for post-cracking in tension and multiaxial softening for post-failure in compression. The model is capable of describing unloading, reloading and multiple-crack formation. Lotfi and Shing (1991) proposed a smeared finite element formulation based on plasticity concepts for brittle materials. They assumed an isotropic behaviour for the uncracked masonry and an orthotropic nonlinear constitutive model for the cracked masonry, combined with a softening law for tension and a hardening/softening law for compression. Mosalam *et al.* (1997) reproduced an experimental test of a two-bay two-storey infilled frame representing the masonry panel with a continuum and homogeneous material obtained through a homogenization process. Full bond was considered between the frame and the panel, but only in the contact zone lengths. Asteris (2003) studied the lateral stiffness of infilled frames in the presence of openings. He used a procedure that implements four-node isoparametric rectangular elements with anisotropic material. The basic characteristic of this analysis is that the infill-frame contact zone is not simulated all over the wall, but only in the contact zones. The contact stresses are estimated as an integral part of the solution, and are not an input assumption.

6.2. Panel-frame contact

The panel-frame interface is typically filled with the same mortar used between the brick layers. As mentioned in Mosalam *et al.* (1997), the interface between the frame members and the infills constitutes a plane of weakness around the wall panel. In the most general approach, the panel-frame contact is modelled with contact elements. Riddington and Smith (1977) simulated the panel-frame contact by means of compression-only joint elements that allow relative displacements between frame and panel. Rivero and Walker (1984) used gap elements and joint elements, where the former represents the no-contact condition, while the latter is used to simulate contact. More recent studies have used inelastic interface elements to model the panel-frame contact. Asteris (2003) developed a step-by-step procedure that considers the contact length between panel and frame by means of a separation criterion and without using any interface element. Later, Asteris (2008) noted that the contact length changes during the seismic excitation due to the variation of vertical load, thus the contact length should be an integral part of the solution and should not be assumed as fixed over the duration of the earthquake in numerical simulations. Amato *et al.* (2008) used a constraint function method combined with axisymmetric 2D interface elements. The element was composed of two contact surfaces: the “contactor” and the “target”, which may come into contact without allowing penetration. No tensile strength was assigned to the joint. This method was successfully adopted for the computation of the contact length, but has not been used to reproduce the full interaction problem. Al-Chaar and Mehrabi (2008) modelled the infilled frame contact by interface elements that include non-linear behaviours in tension, shear and compression.

6.3. Frame members

For the bare frames, the failure may involve diffused flexural cracks and dominant shear cracks in the RC members. To model the RC nonlinear behavior, beam or plane elements are typically used (full three dimensional analyses with solid elements are still too demanding with today’s computational power). In the case of plane elements, a smeared crack model with a homogeneous material is used by Schmidt (1989) and Mosalam *et al.* (1997). However, Rots and Borst (1987) and Lotfi and Shing (1991) reported that smeared-crack elements have a stress-locking problem that does not allow for proper modelling of the shear cracks. To avoid this problem, more refined techniques have been recently developed that combine smeared and discrete crack models where the reinforcing steel is modelled as a smeared overlay or with discrete truss elements. For example, Fig. 12 shows the results of a simulation carried out by Stavridis and Shing (2010): each quadrilateral in the column is replaced by a module of smeared-crack and interface elements (Fig. 12a). Each module consists of four triangular smeared-crack elements connected with four, diagonally placed, double-node, zero-thickness interface elements. The steel bars for flexural and shear behaviour are connected to the concrete elements. Stavridis and Shing (2010) were successful in reproducing experimental tests on a single-bay single-story frame (Fig. 12c) by combing the above-mentioned model for RC frame (Fig 12a) and the simplified micro-micro modelling of Fig. 12b for the infill panel. In case of steel frames, the isotropic material can be modelled preferably with an elasto-plastic behaviour (Fonseca *et al.* 1998, Mosalam *et al.* 1998, Mohebkhah *et al.* 2007).

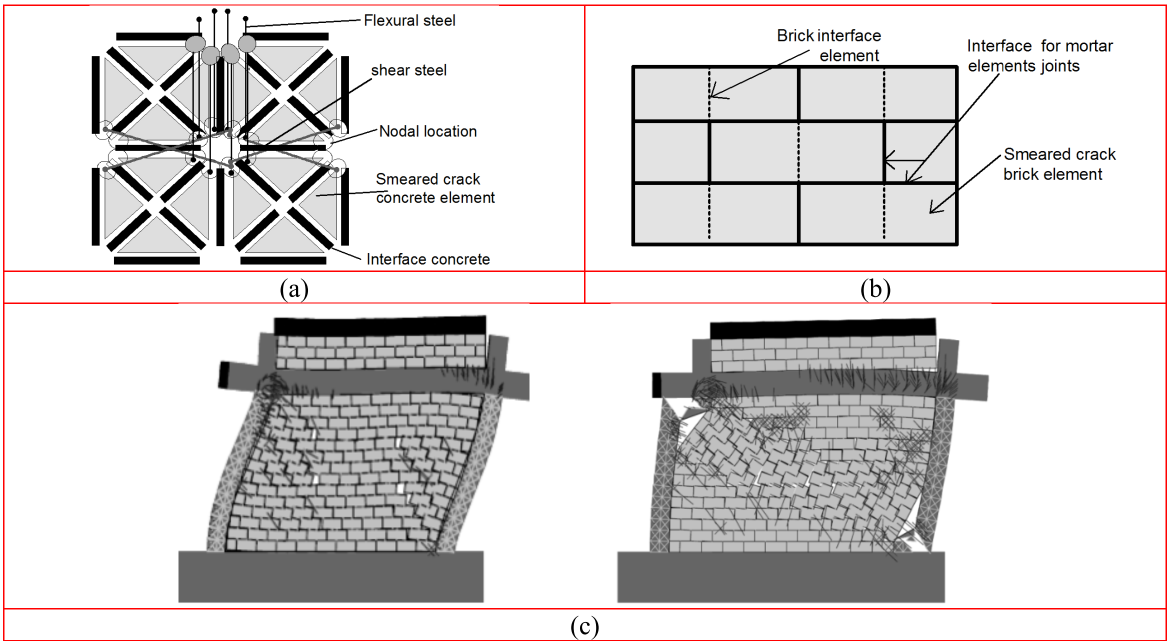


Fig. 12 Micro-model proposed by Stavridis and Shing (2010): (a) Finite element discretization of RC members; (b) finite element discretization of masonry infill (simplified micro-modelling); (c) numerical deformation and failure pattern of an infilled RC frame.

Table 2 Works related to the micro and meso-modelling of masonry panels and infilled frames. For the masonry wall: (1) detailed micro-modelling, (2) simplified micro-modelling, (3) meso-modelling.

Component	Element		Model approach	References
Masonry wall (infill panel)	1	Bricks	Continuum	Computationally expensive, just use for small specimens.
		Mortar	Continuum	
	2	Bricks	Continuum	Rivero and Walker (1984) Stevens and Liu (1992) Lofti and Shing (1994) Lourenco (1996) Mehrabi and Shing (1997) Carol *et al.* (1997) Guinea *et al.* (2000) Alfaiate *et al.* (2005) Attard *et al.* (2007) Al-Chaar and Mehrabi (2008)
		Middle bricks	Interface	
		Mortar	Interface	
	3	Homogeneous panel (brick+mortar)	Continuum	Dhanasekar and Page (1986) Han and Chen (1987) Del Piero (1989) Lofti and Shing (1991) Mosalam *et al.* (1997) Asteris (2003, 2008)
Panel-frame contact	Mortar		Interface Contact Gap	Riddington and Smith (1977) Rivero and Walker (1984) Schmidt (1989) Mosalam *et al.* (1997) Mehrabi and Shing (1997) Asteris (2003, 2008) Amato *et al.* (2008) Al-Chaar and Mehrabi (2008)
Bare frames	RC	Concrete	Continuum	Schmidt (1989) Al-Chaar and Mehrabi (2008) Stavridis and Shing (2010) Sattar (2013)
		Steel	Continuum (Truss)	
	Steel	Steel	Continuum	Fonseca *et al.* (1998) Mosalam *et al.* (1997) Mohebkhah *et al.* (2007)

7. Summary and conclusions

Accessing the role of the infills is particularly important for the seismic analyses of existing frames where the infills may severely affect the frame stiffness, strength, ductility and energy dissipation capacity. Many earthquakes have shown that masonry infills may have a significant impact on the local and global response of masonry-infilled frames. The infill-frame interaction has been studied experimentally, analytically and numerically in seismic prone countries. Some of the results show that uniformly and regularly distributed infill panels may improve the structural seismic behaviour due to a generalized increment of the lateral stiffness and strength at the early loading stages and due to the energy dissipation capacity. However, irregular infill distributions, or no uniform infill cracks and failures may trigger undesired soft storeys and/or torsional behaviour and partial infills may also trigger short column phenomena. The above problems are particularly important for older buildings that are often irregular, conceived and designed without consideration of capacity concepts or, worse, with outdated or no seismic details.

This paper presents a state-of-the-art review of the nonlinear modelling approaches available today for simulating the in-plane seismic response of masonry-infilled frames, especially for RC frames. The paper starts with a short overview on the seismic behaviour of masonry-infilled frames, followed by the description of published experimental test results. Three general numerical approaches has been described here: micro-modelling, meso-modelling and macro-modelling. The degree of precision and the number of required calibrated parameters to reproduce the in-plane infilled frame behavior varies for each approach.

FE models have been extensively used to simulate infilled frame structures under monotonic, cyclic, pseudo-dynamic and dynamic loads. The use of frame (macro) models is widespread because they are simpler and have reasonable computational times, even on personal computers. In this approach, the infills are typically modelled via equivalent struts. These struts may be concentric (they frame into the beam-column joint) or eccentric, single (in a single direction) or multiple. According to the authors' evaluation only multiple eccentric struts can correctly predict the column shear failures often observed in older buildings and induced by short column effects due to the infill-column interaction. The models proposed by Chrysostomou *et al.* (2002) and Crisafulli and Carr (2007) - represented in Fig. 8b and 8d, respectively – appear to be the most precise frame models, as they comprise off-diagonal struts in both directions. The geometry of the struts is always defined by the *w/d* ratio. Although several equations have been proposed for the *w/d* ratio (see Table 1 and Fig. 7), only the equation calibrated by Cavaleri *et al.* (2005) considers the vertical load influence and is thus suggested for use. Among the equations that do not consider the effect of the gravity loads, the Mainstone's equation has been adapted by FEMA-274 and FEMA-306, but according to Fig. 7 its *w/d* ratios give lower values than the ones computed by other researchers, which indirectly influence on the length of the contact zone between infill and frame. In summary, the macro model should be able to reproduce, among others, the effects of the vertical loads, the infill influence on the column bending and shear responses, the in-plane capacity reduction due to out-of-plane actions and the effects of the openings.

Meso and micro-models (plane or solid elements), on the other hand, require the calibration of a large number of parameters to model the nonlinearity of the components (*i.e.* mortar, brick, frame, *etc.*) and thus the input data structure may become quite complex and the computational effort too expensive for the analysis of an entire building. These models have, however, the advantage to accurately capture several failure modes starting from local stress-strain constitutive laws. Due to the high computational cost, it is seen in Section 6 that a detailed micro-modelling for infill panels can at the moment be used only for the analysis of small specimens or to calibrate simpler strut models. A good alternative is the simplified micro-modelling. In this case the mortar is reproduced by interface elements that capture compression, tension and shear failure, while the bricks are represented by 2D plane stress elastic elements. The interface model can be appropriately represented by the interface failure criterion by Lourenco (1996), which has been widely used by other researchers and is implemented in research and commercial FE software. Since the nature of the panel-frame contact is the same as the mortar, it can be represented by a similar interface model. Recent studies (Al-Chaar and Mehrabi 2008, Sattar 2013) indicate that an accurate representation of the RC bare frame is obtained by using 2D plane stress elements for the concrete material and truss elements for the steel reinforcement. The material model for the concrete should include tension and compression constitutive laws, while the truss can be modelled considering an elasto-plastic behaviour. In case of steel frames and isotropic elasto-plastic material can be used. The disadvantage of the simplified micro-modelling, and in general of the micro-modelling, is the increase in computational time when dealing with structures with more than one-bay one storey. When dealing with micro-models, it is finally important to consider that all tension and compression constitutive laws that show softening after reaching the peak response, necessitate careful calibration based on fracture energy principles in order to avoid localization in spread plasticity models and to correctly reproduce brittle failures in discrete cases.

As computational speeds increase, it is expected that modelling of RC or steel frames will include more and more often the masonry infills. For its simplicity and efficiency the macro modelling approach is more commonly used - both in research and practice - than the more refined meso and the micro approaches. On a parallel level, even increasing computational capabilities can lead to faster and more stable meso and micro models, but it is likely that these more refined models will continue to be used for the study of structural subassemblies rather than entire buildings.

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