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# Seismic Design of Masonry and Reinforced Concrete Infilled Frames: A Comprehensive Overview

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**Abstract:** Despite the extensive research on the seismic behavior of infilled frames since 1960's, there is no general consensus towards a unified approach. Purpose of this study is to review the existing analytical and experimental research, as well as code provisions, on masonry and on reinforced concrete infilled frames. The basic characteristics of infilled frames and how these affect seismic performance are summed up. Different approaches of the equivalent strut, broadly used for modeling of the infilled frames, are thoroughly discussed.

**Keywords:** Infilled Frames, Masonry Infills, Reinforced Concrete Infills, Interface, Failure Modes, Strut Model

## Introduction

The most frequently encountered Infilled Frames are the unreinforced brick masonry panels built in the space between columns and beams in a reinforced concrete building. Brick infill walls are commonly used all over the world, e.g., in countries of southern Europe, Asia and South America. The brick infills have proved to increase the seismic response of bare reinforced concrete frames in terms of strength, stiffness and energy dissipation capacity (Abrams, 1994; Bertero and Brokken, 1983; Govidan *et al*., 1986; Manos *et al*., 1995). The presence of a regular pattern of infills in layout and in height of the structure prevents energy dissipation from taking place in the frames (Negro and Verzeletti, 1996). Masonry infills continue to govern the overall response of buildings with reinforced concrete moment-resisting frames even after cracking of the masonry walls (Murti and Nagar, 1996).

In modeling of a new concrete building, the contribution of the masonry infills to the lateral resistance is generally ignored. The reinforced concrete structural elements are designed to resist the entire seismic demand. However, in case the masonry infills may have negative effects on the global response of a building, then the infills should be included in the structural model (CEN, 2004; Fardis, 2009). Typical examples are the irregular distribution of infills (Negro and Taylor, 1996; Negro and Colombo, 1997) and the case of partial-height infill walls that do not extend to the full height of the column (Fig. 1 and 2) which may result in columns experiencing non-ductile shear failure rather than responding in a predominantly flexural manner (Moretti and Tassios, 2006; 2007; Yuen and Kuang, 2015).

In the assessment and rehabilitation of existing substandard reinforced concrete buildings the infills play a decisive role and therefore they have to be included in the structural model. Determination of reliable stiffness and response characteristics of an existing building requires the inclusion of the infills and evaluation of the reduced value of stiffness because of cracking (Crowley and Pinho, 2006).

Fig. 1. Short (captive) column damage because of masonry walls not extending at full height of the column (1999 Athens, Greece, earthquake of 5.9 M)

Fig. 2. Building collapse due to the existence of short columns (1999 Athens, Greece, earthquake of 5.9M)

Furthermore, the addition of infills is a very commonly applied method for consolidating substandard RC buildings, rather than strengthening each deficient frame member (Knoll, 1983).

In a Reinforced Concrete (RC) building the existence of adequate infills may reduce seismic demand of the structural elements and also may increase the lateral strength of concrete frames up to 500% and the stiffness up to 20 times, compared to the respective reinforced concrete values of the bare frame structure (Erdem *et al*., 2006; Murti *et al*., 2006). Different types of infills may be used to strengthen and upgrade the seismic resistance of existing RC frames:

- Masonry infills (Comberscure *et al*., 1996; Griffith, 2008)
- Reinforced concrete infills (Chrysostomou *et al*., 2012)
- Precast panels (Baran and Tankut, 2011; Frosch *et al*., 1996; Higashi *et al*., 1980; 1984)
- Steel braces (Karalis *et al*., 2010; NISTIR 5741, 1995; Pincheira and Jirsa, 1995; Youssef *et al*., 2007), or even installation of energy dissipation devices (Fukuyama and Sugano, 2000)
- Retrofitting of unreinforced masonry infill walls with composite materials e.g., use of fiber reinforced polymer anchors and wraps, or cementitious composite overlays (Erdem *et al*., 2006; Koutromanos *et al*., 2013)

Sugano (1996), reviewed different techniques for seismic rehabilitation of existing RC frames, based on tests on 1/3-scale tests (in general), over a 20-year period and displayed schematically the effectiveness of the techniques considered (Fig. 3). The shear resistance of the bare frame is $V_o$. All the infilling techniques displayed result in significant increase of stiffness, shear resistance and energy dissipation capacity. The highest shear strength, $V_w$ and stiffness are obtained for RC infill walls cast simultaneously ("monolithically") with the frame. According to the results shown on Fig. 3, RC infills offer more stiffness and strength enhancement than masonry infills, while precast panels offer the least seismic enhancement among the filling methods depicted.

The present paper reviews the in-plane behavior of reinforced concrete frames with masonry and RC infills and reviews the different types of available models, with emphasis on the engineering model most often used, that of the diagonal strut. Code provisions for the design of infilled frames are presented and their applicability to masonry and RC infilled frames (i.e., frames with RC infills) is discussed.

## Seismic Behavior of Infilled Frames

Research on infilled frames initiated in 1960's and consisted in experimental investigations on steel frames with brick-masonry filler walls, the loads being applied to the test frames in the plane of the wall.

Infilled frames are "non-integral" (a term used by Liauw and Khan, 1983), given that they are composed of two distinct parts, i.e., the concrete frame and the infill. A composite infilled frame behaves differently from a similar frame with the same stiffness characteristics, yet composed of only one component. The strength and the stiffness of an infilled frame depend on the respective characteristics of each component of the infilled frame, but also on the degree of connection between the infill and the frame. When an infilled frame is subjected to racking load, two phases may be distinguished:

Phase A: Prior to the occurrence of relative slip and detachment along the interfaces between the infill and the frame. The infilled frame behaves as a whole (Fig. 4a).

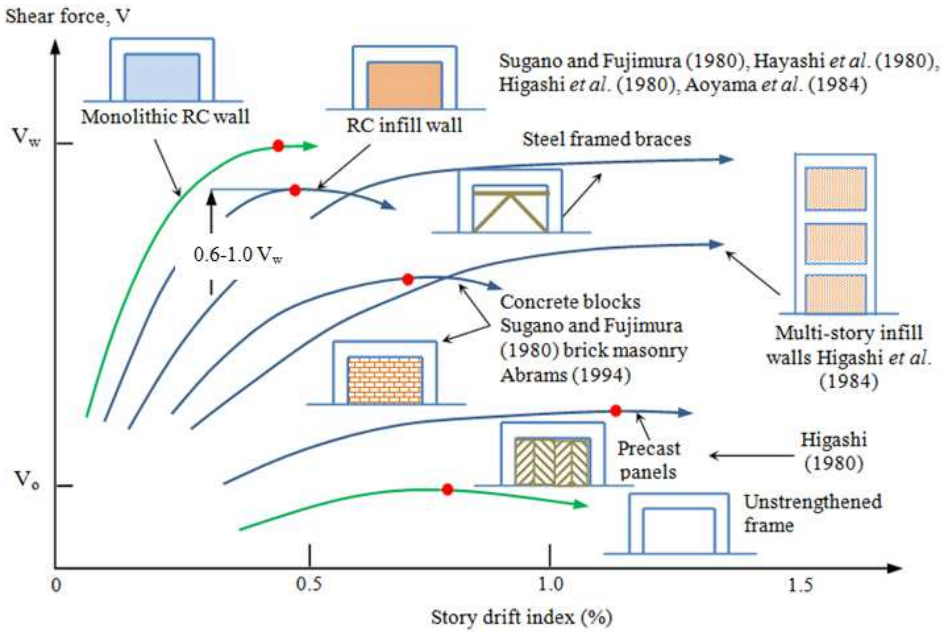


Fig. 3. Typical shear force-story drift diagrams from test results on Reinforced Concrete (RC) frames strengthened with different techniques and subjected to in-plane horizontal loading as reported by Sugano (1996)

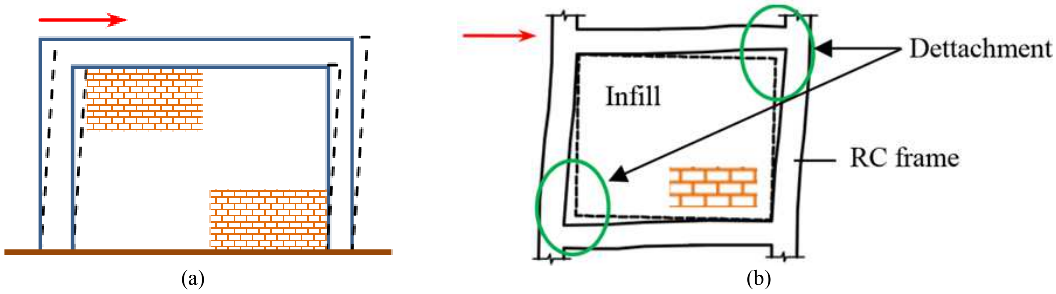


Fig. 4. Reinforced concrete infilled frame subjected to horizontal loading (a) prior to separation between infill/frame: monolithic behavior (b) after separation of infill/frame

Phase B: After the advent of a critical value of relative slip between the infill and the frame. The frame deforms in a flexural mode, whereas the infill deforms mainly in shear (Paulay and Priestley, 1992). When separation between the infill and the frame has occurred, the windward infill diagonal is under compression, while the leeward diagonal, along which the panel is separated from the frame at the corners, is subjected to tension (Fig. 4b). Hence, the observation of Polyakov (1960) that subsequently of the detachment between frame/infill the infill may be substituted by an equivalent strut along the diagonal under compression.

## Models for Infilled Frames

Infilled frames behave in a highly non-linear manner and therefore their modeling is complex. The models of infilled frames fall into two major categories:

- Micro-models, which intend to model in detail (e.g., through finite elements) the infill, the frame and also the interface between the infill and the surrounding frame
- Macro-models, which aim at depicting the global behavior of the infilled frame, making many simplifying assumptions. The most broadly used macro-model for the design of infilled frames is obtained by substituting the infill by an equivalent strut model with pinned ends.

### *Finite Element Models: Micro-Modeling of Infill*

The finite element method allows for the precise modeling of the infill, the frame and also of the behavior along the infill/frame interface. Different elements may be used for modeling the various components of the infilled frame: Beam or continuum elements for the frame, continuum elements for the infill and interface elements for the simulation of the frame/infill interface that enable the calculation of slip and separation. The laws for the materials may be assumed non-linear or linearly elastic. In case of an elastic analysis the number of parameters involved is considerably reduced, thus rendering easier a qualitative estimation of the influence of modeling simplifications and of the different parameters that affect the accuracy of the analytical results (Doudoumis, 2007).

Mallick and Severn (1967) first applied finite element modeling on infilled frames for the calculation of the elastic stiffness of one-bay single-story infilled frames, taking also into account the separation and slip between frame and infill. Since then, a number of refined micro-models have been suggested for detailed elastic or inelastic analysis (Koutromanos *et al*., 2011; Moaveni *et al*., 2013; Yuen and Kuang, 2015).

Micro-models allow in general more detailed modeling of infilled frames, e.g., openings in infills, influence of different methods of connection between infill/frame on the overall behavior. On the other hand, micro-models lead to more complicated analysis and also involve the assumption of the values of more parameters.

### *Strut Models: Macro-Modeling of the Infill*

The equivalent diagonal strut is a rational engineering model for infilled frames, broadly applied for the assessment of existing reinforced concrete structures (Karayannis *et al*., 2011; Moretti *et al*., 2013; Mulgund and Kulkarni, 2011). Owing to its simplicity, the strut model is proposed for the design of infilled frames by several codes (ASCE, 2007; M.I.P., 1997; KAN.EPE, 2013).

The diagonal strut consists of the same material as the infill, has the same thickness, $t_{inf}$, as the infill panel and an equivalent width, w. Several formulas proposed by researchers for the calculation of the strut width, w, will be given in a following section.

The simplest version of the model is one strut along the diagonal under compression ("windward" direction), displayed in Fig. 5 and 6. The area of the diagonal strut, $A_d$, is calculated by Equation 1:

$$A_d = t_{\text{inf}} \times w \tag{1}$$

Where:

$A_d$ = The area of the diagonal strut
$t_{\text{inf}}$ = The actual thickness of infill
$w$ = The width of the equivalent strut

The ends of the strut are pinned and are usually assumed to coincide with the intersection of the centerlines of the frame members (Fig. 6), which results to a strut length longer than the infill diagonal, $r_{\text{inf}}$. This difference, however, is practically insignificant.

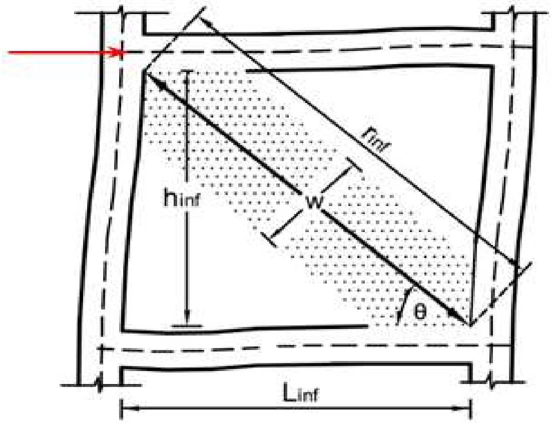


Fig. 5. Characteristics of infill and strut model

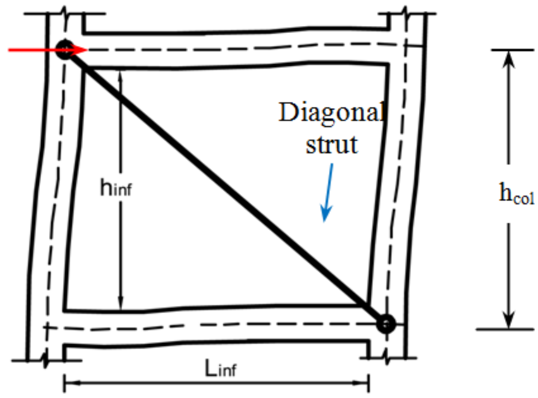


Fig. 6. Diagonal strut model for an infilled frame, usually assumed to coincide with the intersection of the centerlines of the frame members

For linear elastic analysis only the area, $A_d$, the strut length, $r_{\text{inf}}$ and the modulus of elasticity, $E_{\text{inf}}$, of the material of the infill are required to calculate the elastic stiffness of the strut, $K_{strut,el}$, from Equation 2:

$$K_{strut,el} = \frac{A_d \times E_{\text{inf}}}{r_{\text{inf}}} \tag{2}$$

A formula based on the beam concept proposed by Fiorato *et al*. (1970) as reported by Mehrabi *et al*. (1996), for calculating the lateral stiffness of a masonry infilled frame, is described by Equations 3 and 4:

$$K_b = \frac{1}{(1/K_{sh}) + (1/K_{fl})} \tag{3}$$

where, $K_{sh}$ and $K_{fl}$ are the shear and flexural stiffnesses of a cantilever composite structural element ("beam") consisting of a masonry panel and two RC columns.

It is noted that for the shear stiffness, $K_{sh}$, only the masonry wall panel is considered assuming that shear is uniform across the wall, while for the flexural stiffness, $K_{fl}$, the whole composite section is used. Hence, these stiffnesses may be calculated as follows:

$$K_{sh} = A_w \cdot G_w / h_{\inf} \quad (4a)$$

$$K_{fl} = 3E_c \cdot I / h_{col}^3 \quad (4b)$$

$$G_w = E_w / \left(2\left(1+\nu\right)\right) \quad (4c)$$

Where:

$h_{\inf}$ = The height of the infill
$A_w$ = The infill horizontal cross-sectional area
$G_w$ = The shear modulus of the infill
$E_w$ = The modulus of elasticity of the infill
$E_c$ = The modulus of elasticity of concrete
$h_{col}$ = The height of the frame (Fig. 6)
$\nu$ = The Poisson's ratio, assumed to be 0.15
$I$ = The moment of inertia of the equivalent concrete cross-section of the composite beam

Mehrabi *et al*. (1996) point out that the shear-beam model overestimates the lateral stiffness, a fact that the authors attribute to the cracks of the infill and to slip and detachment of the frame/infill interface.

For non-linear analysis, however, the axial force-axial displacement relationship of the strut has to be considered. Generally, the relationship adopted for the strut depicts the global hysteretic response (i.e., stiffness and response degradation) of the infilled frame and not only the behavior of the infill (that is depicted by the diagonal strut). Numerous relationships have been proposed to describe the hysteretic behavior of the infilled frame through the diagonal strut, some of which are discussed subsequently, in the section "Modeling of the Hysteretic Behavior".

## *Various Layouts of Struts*

The simplest version of the strut model is a single strut along the diagonal under compression. Flanagan *et al*. (1994) recommended the use of a compression only truss member in each direction (Fig. 7a) and, in case a similar element is not available for the analysis, they proposed the use of a tension-compression truss member (Fig. 7b) with half the strut area (0.50 $A_d$) in each diagonal direction (their conclusions being based on tests on steel frames with masonry infills).

Crisafulli *et al*. (2000) pertain that that the strut in tension should be present either if shear connectors are used at the interfaces, or if the infill panel is reinforced with horizontal or vertical reinforcement. Further on, that the omission of the diagonal strut in tension is accurate only in case the bond strength at the infill/frame interfaces and the tensile strength of the masonry are very low.

Zarnic and Tomazevic (1988), based on the results of 28 specimens of masonry infilled frames subjected to cyclic lateral loading, proposed the strut model illustrated in Fig. 8b for evaluating the lateral resistance and deformability of the infilled frame after separation between frame/infill and inclined cracking of the infill have occurred. Following the formation of an inclined crack in the masonry infill (Fig. 8a), the part of the column over the end of the cracked infill (length z in Fig. 8b) is more free to deform and behaves as a short or captive column (Fig. 1). The increased shear force thus acting on the RC column may cause damage to the RC columns of the infilled frame, as happened to the column shown in Fig. 9, in a one-story building in Kalamata, Greece, during the 6.2 M earthquake of September 1986.

In order to describe the local effects resulting from the interaction between the contact areas of infill and the frame through the strut macro-model, one of the following options are possible: (a) a strut similar to that of Fig. 8b, (b) a more-fold strut model, e.g., Fig. 10, or (c) the single strut model can be used but a contact length between the strut and the frame members should be calculated so as to make possible the check against possible local failure (ASCE, 2007).

The choice of the number of struts, as parameter of modeling the infill, affects the bending moment and the sheer force diagrams in the frame members. Chrysostomou *et al*. (2002) proposed the use of three compression parallel struts, as depicted in Fig.10a for modeling the strength and stiffness degradation of masonry infill walls in steel frames. The properties of the three struts are calculated by means of the principle of virtual work and it is assumed that the central strut deteriorates faster than the two outer struts. El-Dakhakhni *et al*. (2003) proposed the use of a three non-parallel strut model (Fig. 10b) for estimating the stiffness and the lateral load capacity of concrete masonry infilled steel frames in which the infills fail in corner crushing.

Crisafulli *et al*. (2000) performed numerical analyses on a single RC masonry infilled frame under static lateral loading assuming linear elastic behavior, considering the three different strut models shown in Fig. 11, where $A_d$ is the area of the diagonal strut, as calculated by Equation 1. The authors compared the results of the three strut models with those of a finite element analysis in which non-linear effects were considered to simulate the separation of the infill/frame interfaces. They found that the bending moments and the shear forces are better approximated by Model C, are underestimated by Model A and that they are considerably overestimated by Model B. The axial forces are more or less similar in all models. The authors concluded, however, that the simple strut model may be applied with reasonable accuracy when the analysis is focused on the overall response of the structure.

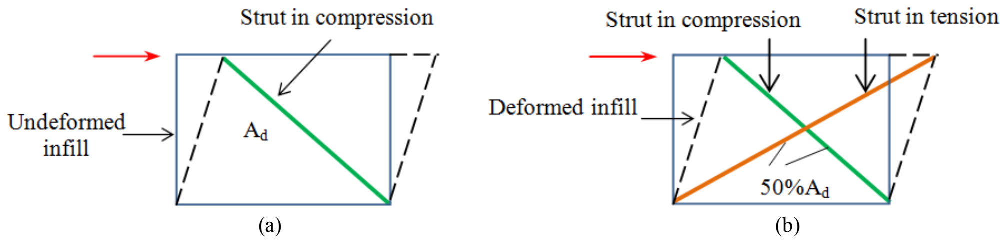


Fig. 7. Infill modeled by (a) one strut in compression, (b) one strut in compression and one strut in tension

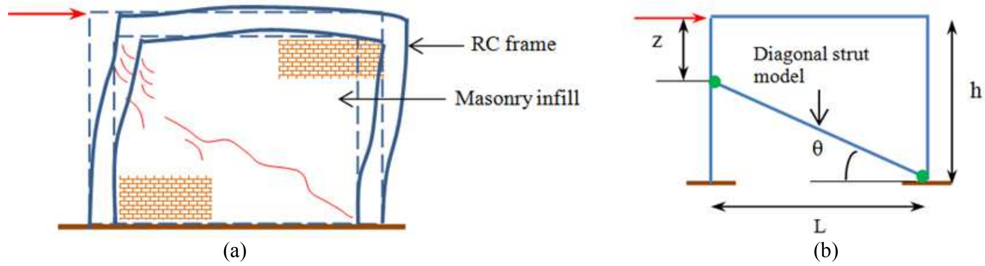


Fig. 8. (a) Diagonal cracking of infill leading to bracing of frame with the lower, triangular part of the infill (b) diagonal strut model to depict the failure in (a). Zarnic and Tomazevic (1988)


Fig. 9. Damage of RC column caused by failure of the adjacent masonry infill (city of Kalamata, Greece, at the earthquake of 6.2M, in September 1986)

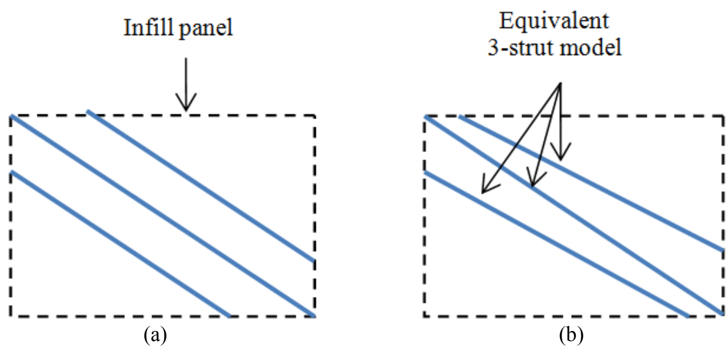


Fig. 10. Three-strut models

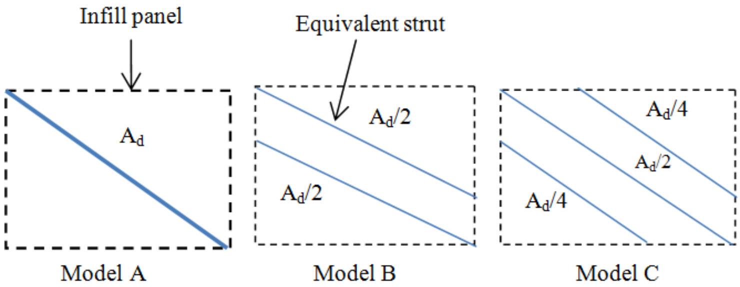


Fig. 11. Different strut models considered in the analysis of a single infilled frame by Crisafulli *et al*. (2000) for the purpose of comparing the force diagrams in the frame members

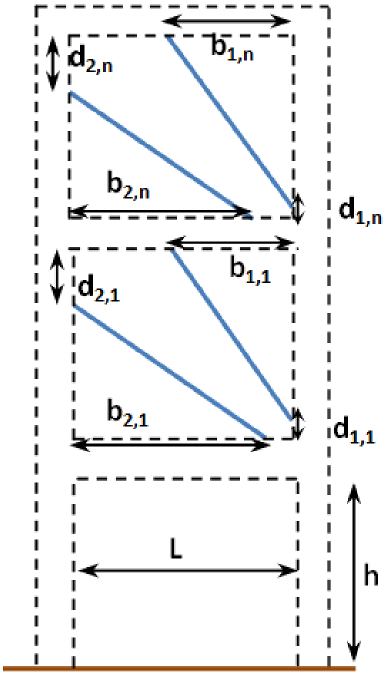


Fig. 12. Two-non-parallel strut model with different characteristics for the first and upper stories, proposed by Fiore *et al*. (2012)

Fiore *et al*. (2012) proposed modeling the infill panel by two non-parallel struts, the location of which is defined as a function of the aspect ratio L/h of the infill, where L, h are the length and height of the infill, respectively (Fig. 12). Different empirical formulas are provided in order to calculate the location of the struts in the first (1) and in the upper (n-) stories, i.e., the parameters $d_i$, $b_i$ shown in Fig. 12. The model was validated through finite model analyses in which the infill/frame interface was modeled through one dimensional joint elements. The results of an analysis of a 5-storey building, without infills in the ground story, are given. The infills were modeled both through single-strut and the two- strut model proposed. The results showed that the two-strut model reproduces well both the global behavior of the infilled frames in terms of displacements and also the local effects on frames in terms of stresses, bending moments and shear stresses.

Torrisi *et al*. (2012) presented a combined macro-model for the simulation both of masonry infilled frames and of confined masonry. The masonry panel is represented by six pinned struts, three along each diagonal, with a hysteretic rule to describe the masonry strut (axial stress Vs. axial strain of the strut). The frame columns are modeled through a composite macro-element with non-linear springs and degrees of freedom, for which hysteretic laws for flexure and shear are provided. The analytical results relate well with experimental results.

It should be noted that the adoption of different values of strut width and of strut layouts, leads to considerably different results (Crisafulli *et al*., 2000; Moretti *et al*., 2014; Papia *et al*., 2003), regarding the stiffness and the overall response of the infilled frame.

### *Calculation of the Equivalent Strut Width*

Many formulas have been proposed for the calculation of the strut width, *w*, (which is the basic parameter of the model), most of them empirical. The values derived for the strut width, *w*, vary between one third and one tenth of the length of the infill diagonal.

Holmes (1961), based on tests on masonry infilled steel frames subjected to racking load was the first to implement the characteristics of an equivalent compression diagonal strut for modeling the infill. The strut had the characteristics of the infill panel and a strut width, *w*, derived from Equation 5:

$$w = \frac{1}{3} r_{\mathrm{inf}} \tag{5}$$

where, $r_{\mathrm{inf}}$ is the length of the diagonal of the infill.

The strut model was further developed by Stafford Smith (1966) who, assuming a beam-on-elastic-foundation analogy and through analytical and experimental investigation on diagonally and laterally loaded square masonry infilled steel frames, concluded that the diagonal stiffness and strength of a masonry panel depend not only on its dimensions and physical properties but also on its length of contact with the surrounding frame. Further investigations by Stafford Smith and Carter (1969) came to the conclusion that it is mainly the flexural stiffness of the column and not of the beam, that influences the stiffness of the infilled frame. They introduced the parameter $\lambda$ (Equation 6) to express the relative stiffness of the column of the frame to the infill and the parameter $\alpha$ (Equation 7) to describe the length of contact between the infill and the column:

$$\lambda = \sqrt[4]{\frac{\left(E_w t_{\inf} \sin 2\theta\right)}{4 E_f I_{col} h_{\inf}}} \tag{6}$$

$$\frac{\alpha}{h} = \frac{\pi}{2\lambda h} \tag{7}$$

Where:
$E_w$ = Young's modulus of elasticity of infill
$t_{\inf}$ = Thickness of infill
$h_{\inf}$ = Height of infill
$I_{col}$ = Moment of inertia of the column
$E_f$ = Young's modulus of elasticity of column
$\theta$ = Angle whose tangent is the infill height-to-length aspect ratio in radians (Fig. 5)

Stafford Smith and Carter (1969) produced curves that related graphically the strut width, w, to the parameter $\lambda$ (Equation 6) for different panel proportions. In their analysis they assumed the contact zone between the infill and the beam to be constant and equal to half the length of the infill.

Mainstone (1971) proposed Equation 8 to calculate the equivalent strut width, w, which depends on the parameter λ (Equation 6) and on the diagonal length, $r_{\inf}$, of the infill. The expression of Mainstone is used in FEMA 306 (1999) and also in ASCE (2007):

$$w = 0.175\left(\lambda h_{col}\right)^{-0.4} r_{\inf} \tag{8}$$

Where:
$h_{col}$ = Column height between beam centerlines
$\lambda$ = A parameter calculated from Equation 6

Liauw and Kwan (1984), based on analytical data and assuming values for the angle, $\theta$, of the equivalent diagonal strut equal to 25° to 50°, proposed Equation 9 to estimate the equivalent strut width:

$$w = \frac{0.95 \sin 2\theta}{2\sqrt{\lambda}} r_{\inf} \tag{9}$$

Where:
$r_{\inf}$ = The length of the infill diagonal
$\lambda$ = The parameter calculated from Equation 6

Paulay and Priestley (1992), based on analytical results of masonry infilled RC frames, proposed a conservative value for the strut width (Equation 10):

$$w = \frac{1}{4} r_{\inf} \tag{10}$$

It should be pointed out that the values of the strut width, $w$, calculated by the various equations, are valid prior to separation of the infill from the frame which, according to Paulay and Priestley (1992), is expected to occur at 50% of the lateral shear resistance of the infilled frame. After separation occurs, the stiffness degrades and this should be taken into account by reducing the strut effective width, w. Paulay and Priestley (1992) suggest that the natural period of the frame should be calculated according to the structural stiffness after the advent of separation.

Given that the equations cited above are derived from tests on frames with masonry panels simply bared by the frame, all these provisions are valid in the absence of mechanical connectors between infill and RC frame.

### *Modeling of the Hysteretic Behavior*

For the equivalent strut model in linear elastic analysis, the elastic stiffness of the strut is calculated according to Equation 2 by the geometrical characteristics of the strut and modulus of elasticity of the infill. For non-linear modeling of the infilled frame, the hysteretic behavior of the components should be accounted for. Infilled frames cannot be modeled as elasto-plastic systems because of their response and stiffness degradation that gradually occur under cyclic loading.

Hysteretic behavior in a strut model may be introduced by a suitable axial force-axial deformation diagram depicting both the stiffness and the force degradation of the strut. The hysteretic models should be calibrated against experimental data and are representative of an infilled frame with specific characteristics of the individual components and connection along the interface. Various researchers have proposed different force-displacement relationships for the strut model, several of which are grouped in the paper of Crisafulli *et al*. (2000).

Klinger and Bertero (1976) attempted to reproduce the experimental results of one-bay, 3-story RC frame with clay-infill using a strut model. The infill was modeled by two equivalent diagonal struts, one in each direction, with strut characteristics calculated according to Equation 8, proposed by Mainstone (1971). Three different hysteretic mechanical behaviors for the struts were used successively, involving a slight increase in complexity from one model to the next so as to achieve better approximation of the experimental results. The results from the analysis of the modeled infilled frame were compared to the actual experimental behavior. The model displayed in Fig. 13 was found to agree better with the test results, as it depicts the two distinct phases observed at reloading of the tested RC frame infilled with clay blocks:

(a) Upon reversal of loading and until the vertical panel cracks close and the panel is returned to its undeformed configuration, lateral strength and stiffness are essentially zero (path OEE', in Fig. 13) and (b) Following panel crack closure, the panel reloads, but with reduced stiffness and strength (OB, OB') compared to the initial elastic behavior (OA); this hysteretic law for the equivalent strut idealization depicts the stiffness and strength degradation of the infilled frame during cyclic loading (Klinger, 1980).

In case of masonry infilled frames, the major part of pinching is attributed to closing of the vertical cracks of the masonry panel, as was observed also by Klinger and Bertero (1976). In case if RC infills, however, the pinching effect observed is practically entirely attributed to relative slip at horizontal interfaces between infill and frame (Moretti *et al*., 2014; Oesterle *et al*., 1976). In Fig. 14 is illustrated the diagram of horizontal load versus slip at the interface between a reinforced concrete frame and a RC infill of a 1/3-scaled model tested in the Laboratory of Concrete Technology and Reinforced Concrete Structures of the University of Thessaly (Perdikaris *et al*., 2012). The RC infill aspect ratio was $L_{inf}/h_{inf} = 1.20$ and the infill was connected to the frame through 6-mm dowel bars (at 100 mm center-to-center spacing) only along the horizontal interfaces with the frames, placed with epoxy resin in holes drilled in the frame.

It is noted that when an infilled frame is modeled though the diagonal strut model, pinching effect of the hysteresis response results from the non-linear behavior of the strut model (Fig. 13), while in real infilled frame structures pinching is mainly caused by slip along the frame/infill interface in case of a RC infill and/or closing of the cracks in case of a masonry infill panel.

Leuchars and Scrivener (1976), in order to describe the response of an infilled frame subsequent to inclined cracking of the infill panel (Fig. 8a) that potentially may cause damage to both columns of the frame, proposed the "knee-braced frame concept" shown in Fig. 15. The friction element added allows for consideration of horizontal shear sliding.

Crisafulli and Carr (2007) proposed a similar model for masonry infills, displayed in Fig. 16. The shear spring represents the behavior of the infill subsequent to shear failure of the infill, either along mortar joints, or due to diagonal tension failure. The spring's stiffness is a fraction of the equivalent strut stiffness and its' hysteretic response is modeled through an elastoplastic rule with variable shear strength, depicting: (a) The elastic response prior to bond-shear failure and (b) Sliding.

Puglisi *et al*. (2009) introduced the concept of a plastic concentrator, situated at the point in which the diagonals interconnect (Fig. 17). It is assumed that all inelastic effects are lumped at the concentrator, which also achieves coupling between the two struts. However, as the authors admit, the model does not consider stiffness degradation, which is essential in the case of infilled frames.

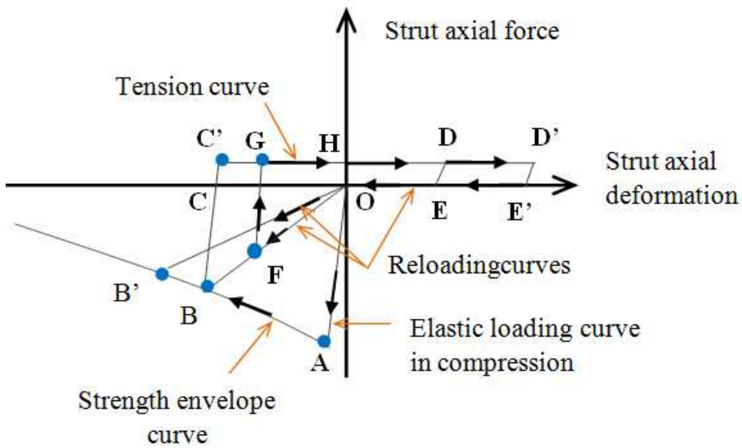


Fig. 13. Typical hysteretic behavior of the strut model proposed by Klinger and Bertero (1976)

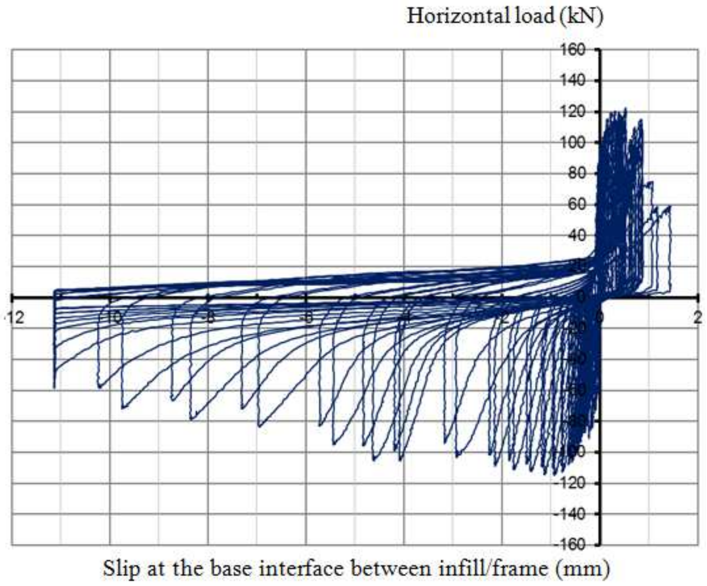


Fig. 14. Horizontal load vs. sliding at horizontal frame/infill interface at the base of 1/-3-scaled RC infilled frame subjected to quasi-static cyclic horizontal displacements (Perdikaris *et al*., 2012)

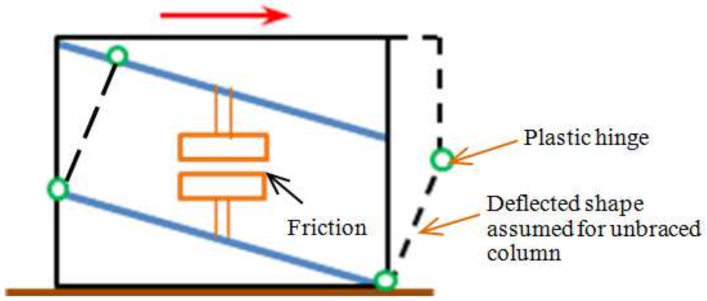


Fig. 15. Model that represents the response of an infilled frame after formation of plastic hinges at the columns of the frame subsequent to inclined cracking of the infill (Leuchars and Scrivener, 1976)

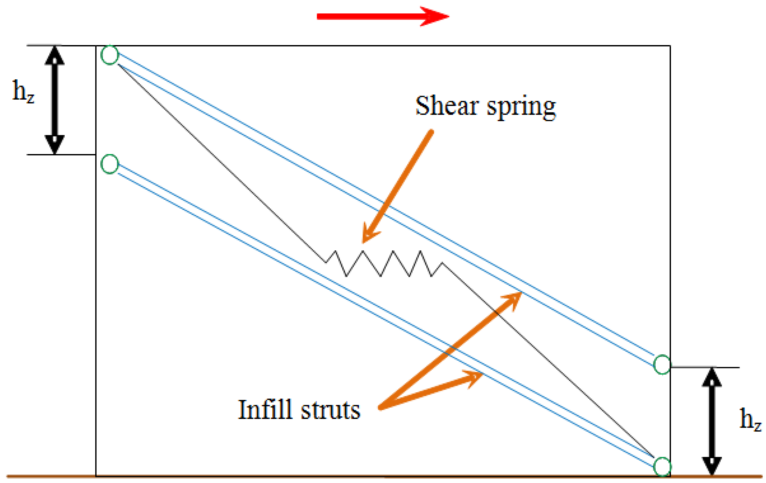


Fig. 16. Multi-strut model for masonry infills with shear spring to represent shear failure of the masonry panel (Crisafulli and Carr, 2007)

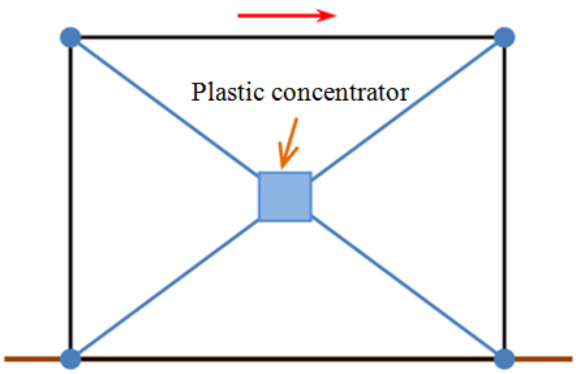


Fig. 17. Plastic concentrator suggested by Puglisi *et al.* (2009)

## *Code Provisions for Infilled Frames*

Specific code provisions for the design of infilled frames are scant. FEMA 306 (1999) and ASCE (2007) recommend the use of the equivalent diagonal strut for modeling infills, the width, *w*, of the strut being calculated from Equation 8, proposed by Mainstone (1971). The strut contact lengths with the column and the beam, $l_{ceff}$ and $l_{beff}$, respectively, are calculated by formulas given in Equations 11a to 12b. These contact lengths are used to assess the interaction between the infill and the frame. The standards apply both to masonry and to reinforced concrete infills:

$$l_{ceff} = \frac{w}{\cos\theta_c} \tag{11a}$$

Where:

$$\tan\theta_c = \frac{h_{\inf} - \dfrac{w}{\cos\theta_c}}{L_{\inf}} \tag{11b}$$

$$l_{beff} = \frac{w}{\sin\theta_b} \tag{12a}$$

Where:

$$\tan\theta_b = \frac{h_{\inf}}{L_{\inf} - \dfrac{w}{\sin\theta_b}} \tag{12b}$$

In CSA S340.1-04 (Anderson and Brzev, 2009) the equivalent strut width, *w*, is calculated according to Equation 13. The width w depends on the contact length between strut/column, $\alpha_h$ and the contact length between strut/beam, $\alpha_L$, calculated by Equations 14 and 15, respectively, where $\theta$ is the angle of the diagonal strut with the horizontal: tan $\theta = h_{\inf}/L_{\inf}$. For the evaluation of the compressive strength of the strut, CSA S340.1-04 (Anderson and Brzev, 2009) suggests the use of another width, $w_e$, for the strut (Equation 16). It is noted that CSA provisions are basically intended for the design of masonry infilled frames:

$$w = \sqrt{\alpha_h^2 + \alpha_L^2} \tag{13}$$

$$\alpha_h = \frac{\pi}{2}\sqrt[4]{\frac{4E_f I_{col} h_{\inf}}{E_m t_{\inf}\sin 2\theta}} \tag{14}$$

$$\alpha_L = \pi\sqrt[4]{\frac{4E_f I_b L_{\inf}}{E_m t_{\inf}\sin 2\theta}} \tag{15}$$

$$w_e = \min\left[(0.5w), 0.25\sqrt{h_{\inf}^2 + L_{\inf}^2}\right] \tag{16}$$

In the formulas adopted by FEMA 306 and CSA S304.1-04, the strut width for the calculation of in-plane bearing stiffness of infilled frames depends on the ratio of the flexural stiffness of the column to the stiffness of the infill, a factor that was initially suggested by Stafford Smith and Carter (1969) and has been consecutively confirmed by experimental and analytical studies (Papia *et al.*, 2003; Anderson and Brzev, 2009).

It is observed that the FEMA 306 and CSA S304.1-04 expressions for calculating the equivalent strut width refer to infills not connected to the frame (Anderson and Brzev, 2009), because they are derived from research on frames (both from steel and reinforced concrete) with masonry infills. However, in the case of a strong RC infill/frame connection, the overall stiffness of the infilled frame is underestimated when the equivalent strut width is calculated according to the above code provisions (Moretti *et al.*, 2014).

European code for seismic design EN1998-1 (CEN 2004) does not include any provisions for the design of infilled frames. The Greek code for rehabilitation (KAN.EPE, 2013) assumes an equivalent strut width w = 0.15 $r_{\inf}$ ($r_{\inf}$ being the length of the infill diagonal), while the Italian code for seismic design (M.I.P, 1997) assumes an equivalent strut width w = 0.10 $r_{\inf}$.

The in-plane lateral strength of infilled frames is determined from expressions that estimate the shear resistance of the different potential failure mechanisms. The minimum shear resistance among the various mechanisms considered determines the failure mechanism most likely to occur.

The in-plane failure mechanisms of infilled RC frames, both for masonry and for RC infills, for which FEMA 306 (1999) offers detailed expressions, are:

- Sliding shear failure of infill
- Compression failure of infill

- Diagonal tension failure of infill
- Shear failure of infill panel (with and without panel reinforcement)
- Flexural failure of RC frame members
- Shear failure of RC frame members
- Failure of joints of RC frame members
- Bond-slip failure of joints of lap-splice connections at the base of RC columns (in non-ductile frames)

Although FEMA 306 provisions may be applied both for masonry and for RC infills, it should be recalled that these recommendations originate from masonry infilled frames. Moretti *et al*. (2014), based on 1/3-scale tests of RC infilled frames, have shown that several of these failures are unlikely to occur in RC infilled frame.

For the out-of-plane resistance of masonry infills FEMA 306 (1999) includes an expression based on recommendations of Angel and Abrams (1994), while CSA S304.1-04 does not contain similar provisions.

## Failure Modes of Infilled RC Frames

### *Masonry Infilled Frames*

The characteristic failure modes of masonry infilled frames subjected to in-plane seismic loading are schematically displayed in Fig. 18. In masonry infill panels, failure initiates from the weaker component, which is the mortar. Bed-joint failure may occur either as sliding failure along one or more horizontal bed-joints, caused by direct shear mechanism (Fig. 18a and b), or as failure along the diagonal under tension (Fig. 18c). The mechanisms of these types of failure have the following characteristics, as reported by Anderson and Brzev (2009):

The bed-joint sliding failure mechanism (Fig. 18a and b) occurs along horizontal interfaces and results in separation of the infill in two or more parts. The separated parts of the masonry infill allow the adjacent columns to deform freely, which may result in the “knee-braced” mechanism reported by Zarnic and Tomazevic (1988) and shown in Fig. 8. Subsequent to bed-joint sliding the behavior of infilled frames is governed by the characteristics of the frame members: (a) In case the frame has adequate detailing to exhibit ductile behavior, plastic hinges are formed at the ends of the un-sustained length of the column (“captive column”) and a ductile mechanism is formed which enables energy dissipation through friction along the bed joints and at the plastic hinges of the RC columns. (b) In case the frame is not designed according to modern provisions, the captive columns are likely to fail in shear, thus leading to brittle overall behavior of the infilled frame. When sliding along horizontal bed joints occurs, the mechanism of the diagonal strut cannot develop and sliding becomes the governing failure mechanism (Anderson and Brzev, 2009).

Diagonal tension failure (Fig. 18c) is caused by sliding failure in the mortar along bed joints having the general direction of the main diagonals. Cracks start in the center of the infill and have the direction of the compressive strut (i.e., perpendicular to the diagonal under tension) and cracks propagate later along the diagonal to the corners of the panel. Tensile cracks may be followed by failure in compression at the corners of the infill (Fig. 18d). The onset of diagonal cracking may not be considered as failure. Ultimate load is governed by either the capacity of diagonal strut in compression, or bed-joint sliding shear resistance.

According to ASCE (2007) and FEMA 306 (1999), a potential captive column mechanism may occur at the parts where the compressive concrete strut is in contact with the frame members, i.e., at the lengths $l_{ceff}$ and $l_{beff}$ shown in Fig. 19 and calculated according to Equations 11 and 12. In order to exclude the development of a similar failure, these parts of the frame members have to be designed so that they may carry the sheer force that corresponds to the formation of flexural plastic hinges at the end cross-sections of the frame members. In case of a potential captive column with length $l_{ceff}$ the minimum shear resistance of the column in order to avoid shear failure is calculated by Equation 17:

$$V = 1.25\frac{M_p^+ + M_p^-}{l_{ceff}} \tag{17}$$

Where:

$M_p^+, M_p^-$ = The flexural capacities of column
$l_{ceff}$ = The contact length between strut and column, calculated from Equation 11a

It is noted that when infilled frames are modeled through the diagonal strut concept, attention is paid to the geometrical characteristics of the strut model, i.e., number of struts, equivalent strut width, while the interaction between infill and frame is often overlooked. This simplification may lead to inaccurate results regarding the seismic performance of the structure, especially for buildings designed and constructed according to older codes. Lack of capacity design and absence of adequate detailing, render the frame members vulnerable to brittle failure. Hence, in formulating the strut model, all the possible modes of failure of the infilled frame should be taken into account (Kyriakides, 2011; Fotakopoulos *et al*., 2013; Liauw and Kwan, 1985; Mehrabi and Shing, 2003).

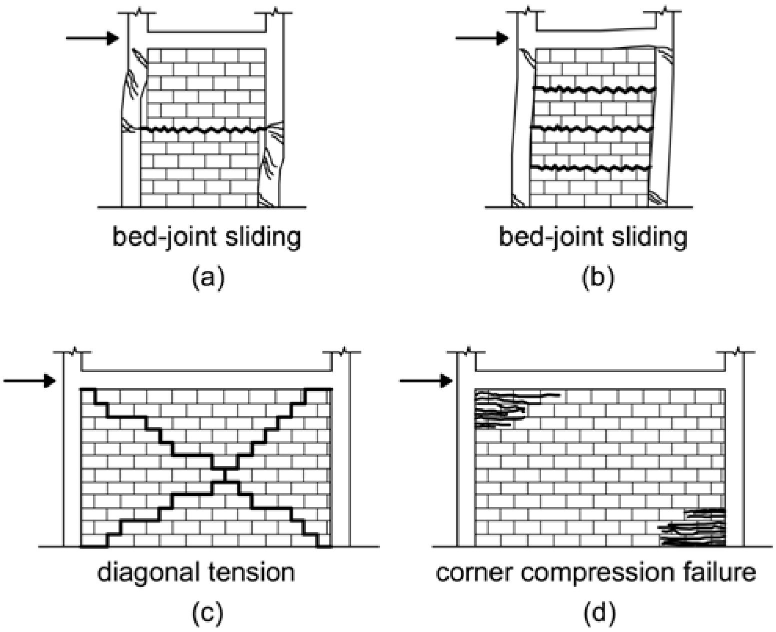


Fig. 18. Typical failure modes of masonry infilled frames: (a) and (b) sliding shear failure of infill possibly leading to failure of the adjacent frame columns (c) diagonal tension failure (d) compression failure of infill corners (reported by Anderson and Brzev, 2009)

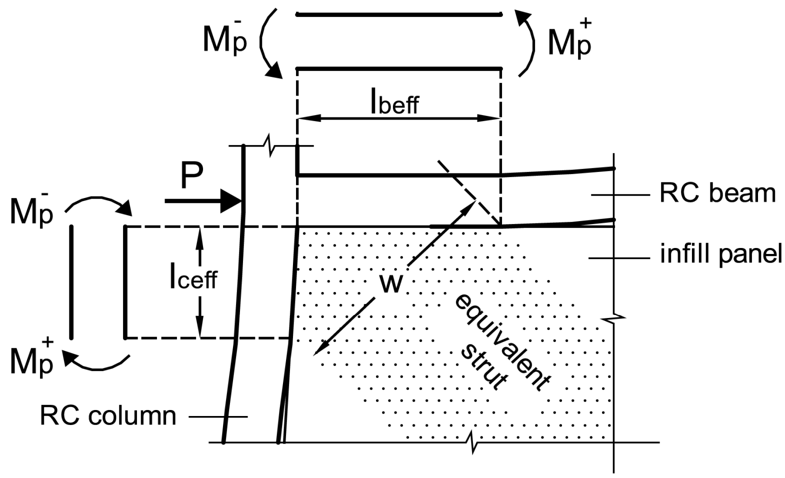


Fig. 19. Potential formation of captive elements along the lengths of the frame components in contact to the equivalent compression strut, according to ASCE (2007) and FEMA 306 (1999)

## *Reinforced Concrete Infilled Frames*

Reinforced Concrete (RC) infilled frames (i.e., when the infill is from reinforced concrete) possess considerably higher tensile strength, even in the absence of web reinforcement in the infill panel, compared to masonry panels in which the weak link is the low resistance of mortar along the bed-joints. Furthermore, the stiffness and the resistance of the RC infill are generally significantly higher than the respective characteristics of the frame members. Therefore, in the region of the frame joint and the corner of the infill panel the weaker link is usually the frame component rather than the infill. Hence, none of the typical failure modes depicted in Fig. 18 are likely to occur in RC infilled frames.

Degradation of strength and stiffness in RC infilled frames initiate when slip and detachment along the frame/infill interfaces occur (Oesterle *et al.*, 1976; Synge *et al.*, 1980).

The characteristics of the frame/infill interface and the type of connection between the two components have a major impact upon the load transfer mechanisms, the mode of failure, the collapse load and the deformation at which failure of the infilled frame occurs (Kahn and Hanson, 1979; Koutromanos *et al*., 2013; Moretti *et al*., 2014). In RC infills the influence of the frame/infill connection is more decisive than in masonry infills, because (a) there are more options for connecting a RC infill with the frame and (b) the inherent higher stiffness of a RC infill affects more the infilled frame, compared to a masonry infill with similar geometry.

A variety of different types of connection between RC infills and RC frame have been tested, e.g., concrete keys, mechanical or adhesive dowels, with or without roughening of the infill/frame interfaces (Hayashi *et al*., 1980; Sugano and Fujimura, 1980). The embedment of adhesive anchors was proved to be the most effective method (Altin *et al*., 1992; Aoyama *et al*., 1984) and therefore it is the most commonly applied method for the connection between the infill and the frame.

The RC infill can be connected to the frame either along all four sides, or only along the horizontal interfaces, or connected with the frame by simple bearing (Fig. 20). The option of connecting the frame to the infill by simple bearing leads to increased slip in the frame/infill interface and to premature failure of the infilled frame. The connection of the infill to the frame in all four sides, compared to connection only along the horizontal interfaces, leads to higher stiffness, yet to reduced displacement ductility and increased response degradation (Hayashi *et al*., 1980; Moretti *et al*., 2014). The magnitude of relative slip at the frame/infill interfaces decreases considerably in the presence of dowels, as compared to the infill that is not connected to the frame (Fig. 20a). However, the presence of stiff boundary elements (frame columns), is the most decisive factor in reducing the slippage between frame/infill (Oesterle *et al*., 1976; Moretti *et al*., 2014). Artificial roughening of the interfaces between frame/infill leads to reduced slip at low levels of horizontal load, but does not significantly influence slippage at failure load.

Many tests have been performed investigating the characteristic parameters of the dowels (Fig. 21): The rebar diameter, $\varnothing_d$, the spacing, *s*, between dowels, the embedment length $L_{b1}$ in the frame component (usually fixed with epoxy in holes drilled in the existing frame members) and $L_{b2}$ the embedment length of the dowel rebar in the infill.

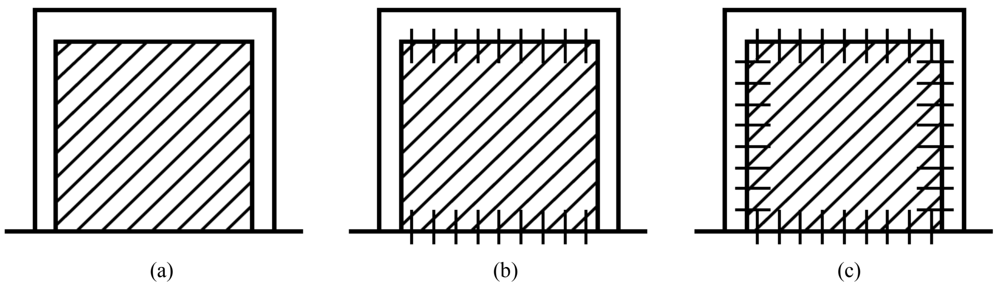


Fig. 20. Different types of connection of a RC infill to the surrounding frame: (a) simple bearing (b) only along horizontal interfaces (c) along all the interfaces

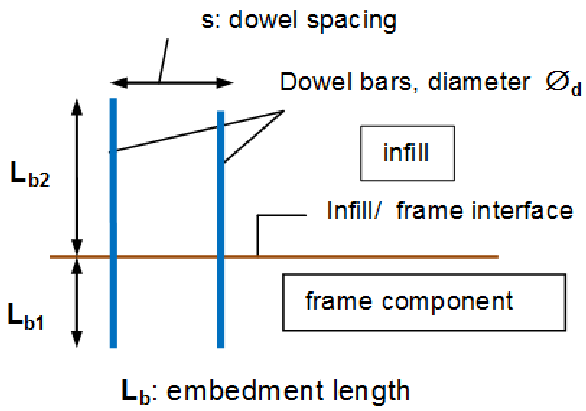


Fig. 21. Characteristic parameters of steel dowel rebars for the connection along frame/infill interfaces

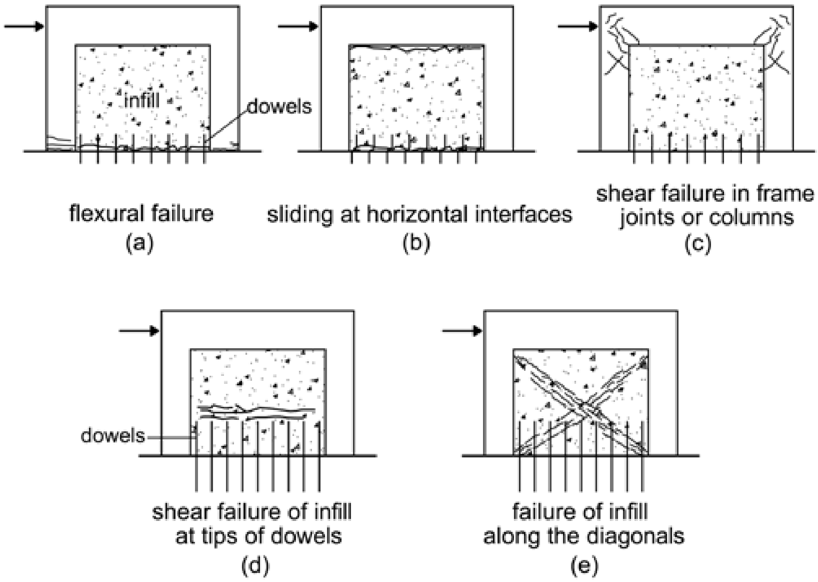


Fig. 22. Typical failure modes of RC infilled frames. Failure modes (d) and (e) correspond to very strong connection between infill/frame. (In all cases only the dowels in the lower horizontal infill/frame interface have been depicted)

According to test results, a minimum embedment length of 6 $d_b$ (where $d_b$ is the dowel rebar) is required for an epoxy grouted dowel to achieve its full shear strength. Codes specify, more conservatively, an increased minimum embedment length, e.g., 10 $d_b$ in ASCE Standard (ASCE, 2007), or 8$d_b$ in KAN.EPE (2013). When longer embedment lengths are used for dowels, the dowel rebars may participate also in carrying the bending moment acting along the horizontal interfaces. The presence of dowels with increased embedment length, larger diameters (in correspondence to the scaling factor of the specimens) and with high geometrical steel content may lead to shear failure of the infill at tips of the dowels (Altin *et al*., 1992), or failure of the infill along the diagonals (Altin *et al*., 2008; Kara and Altin, 2006), depicted in Fig. 22d and 22e, respectively. Furthermore, higher embedment lengths than the minimum required (8 $d_b$ to 10 $d_b$) result in higher initial stiffness and shear resistance to a certain extent, but also lead to reduced ductility (Moretti *et al*., 2014; Perdikaris *et al*., 2012).

Typical failure modes likely to occur in RC infilled frames are illustrated in Fig. 22.

## Conclusion

Infill walls increase significantly the stiffness of a RC frame building. However, their presence may also be detrimental to the overall earthquake performance of the structure, especially when the building is not designed according to modern principles. Modeling of infilled frames is complex because of their highly non-linear behavior when subjected to horizontal loading.

The equivalent diagonal strut model is a practical engineering tool for the design of infilled frames. The type of strut model adopted, however, can alter significantly the results. Therefore, the strut model characteristics should be selected according to the objective of the analysis. In case of RC infills, the connection between infill and frame strongly affects seismic behavior and should be appositely modeled.

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## Ethics

This article is original and contains unpublished material. References are provided when material from other researchers is cited. The author confirms that no ethical issues arise from the content of this work.

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