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SIMPLIFIED MODELLING OF IN-PLANE BEHAVIOUR OF MASONRY INFILLED RC FRAMES UNDER SEISMIC LOADING: ADVANTAGES AND BARRIERS

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ABSTRACT

Due to the significant contribution of the infill walls on the global seismic response of reinforced concrete (RC) buildings, several studies were carried out over time in order to identify this contribution. These studies led to the development of several models to simulate realistically the interaction between the infill and the adjacent RC frame. Due to the larger variety of masonry properties, several models can be found in the literature and these proposals are seen to be different in terms of their modelling refinement. The various techniques available to model the in-plane behaviour of infills are addressed, highlighting their application feasibility. Due to their low computational effort, the use of strut elements to model the behaviour of infill panels is a popular approach and several proposals are discussed in more detail. The reliability of using a strut model to simulate the infill panel behaviour is discussed based on experimental data from ten different test campaigns. In light of this comparison, several recommendations in terms of the characteristics of the strut models that need to be considered to achieve more realistic results when analysing the behaviour of masonry-infilled RC frame structures were established.

Keywords: masonry infilled RC frames, in plane behaviour, strut model

1. INTRODUCTION

Masonry-infilled reinforced concrete (RC) frames are widely used as a structural system around the world. In such structural system, the structural contribution of the masonry infill walls for the lateral load resisting system is usually ignored. However, the actual behaviour of such structures that has been observed in past earthquakes (e.g. Chania, 2008, Tabanlı (Van), Turkey, 2011, Nepal, 2015) shows that their response is often different from the response of the bare frame only, as shown in Figure 1 and Figure 2. In some cases, the interaction between the masonry infill and the frame can lead to premature failure (e.g. in case of infill configurations that lead to soft-storey mechanisms or short column effects such as those of Figure 2 a) and b)) while in others it can improve the actual performance of the building.

a) Infill shear and minor frame damages, Nepal earthquake, 2015

b) shear failure in the infill and in the RC column, San Antonio earthquake, 2010 (Velasquez et al. 2016)

c) Infill shear and flexural frame damages Wenchuan earthquake, 2008 (Li et al. 2008)

d) Collapsed infill panel with major frame damage Wenchuan earthquake, 2008 (Li et al. 2008)

Figure 1 Damage patterns of masonry infills and RC frames which varies from minor cracks to major failure

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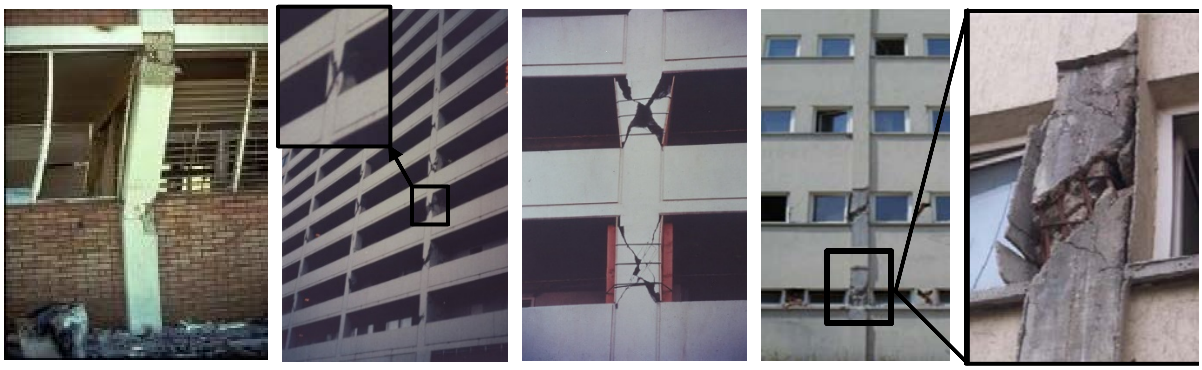
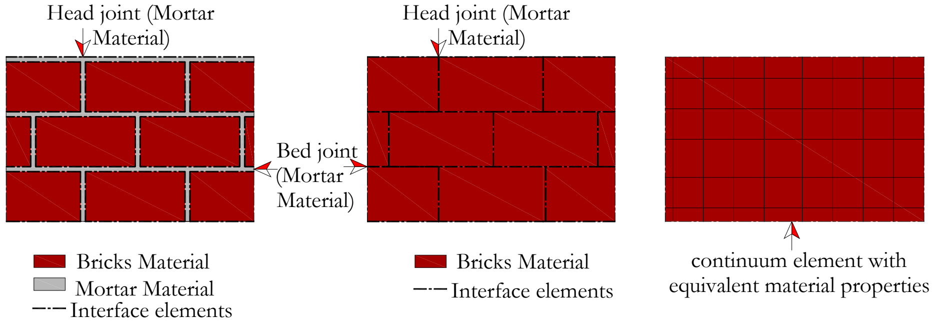


a)Managua earthquake. 1972 (Pradhan et al. 2012) *b) Northridge earthquake, 1994 (Faison et al. 2004)* *c) Düzce earthquake, 1999 (Beyhan and Polat 2011)*

Figure 2 Captive column failure (short column failure)

Early experimental tests in masonry-infilled RC frames carried out in the mid-1950s showed that the infill works as diagonal bracing for the RC frame. Since then, macro modelling approaches (e.g. using strut models) have been used due to their simplicity to represent the behaviour of infill panels under earthquake loading and account for their structural effects in the overall performance of the masonry-infilled system. Generally, these studies can be categorized into two main groups: a) stiffness-based methods that define the geometric cross-section of the strut, which is then combined with an equivalent material representing the masonry (Holmes 1961, Mainstone 1971, Hendry 1990); b) strength-based methods that define a backbone curve for the force-displacement curve of the equivalent strut element (Bertoldi et al. 1993, Panagiotakos and Fardis 1996, Dolšek and Fajfar 2008). With the availability of high computational resources, the use of continuum detailed nonlinear finite element models became feasible for the simulation of the complex behaviour of these structures. Depending on the level of component refinement, these models can be classified into three main groups: Micro, Meso and Macro-models, as shown in Figure 3. Although these models can capture the complex behaviour of infilled structures, they are unable to be implemented in performance-based studies requiring a high number of analysis given their high computational cost. Therefore, simplified and less computationally demanding models such as those based on strut elements are necessary. However, an insufficient number of studies has addressed the differences in accuracy and performance that are obtained when using different types of strut models to represent the infill panels with different configurations and properties. Therefore, the main objective of the current paper is to analyse the performance of strut models to model the structural response of masonry infills when their properties and parameters are defined using different methods (i.e. geometric-based methods, force-displacement approaches and reduction factors to account for openings). The behaviour of several existing strut models was compared with available data from several experimental tests in masonry-infilled RC frames to determine the most realistic and accurate approaches. The selected experimental specimens exhibit different infill configurations (e.g. panels with solid and perforated masonry), and infill material properties. The comparison between the results obtained from the different strut models and experimental data was performed by estimating the response deviation between the numerical and the experimental results.



a) Micro-continuum models *b) Meso-continuum models* *c) Macro-continuum models*

Figure 3 Continuum modelling approaches for masonry infills

3. STRUT MODELS TO SIMULATE MASONRY INFILLS

As referred, from a practical point of view, the use of continuum modelling approaches is not feasible for large systems due to the high computational costs. Due to their inherent simplicity, strut models (i.e. macro-models) are one of the most practical approaches to represent the behaviour of infill panels, especially for design and performance assessment purposes. As shown in Figure 4, these modelling approaches are based on replacing the infill panel by an equivalent pinned diagonal strut system. A large amount of research has been dedicated to determining the main structural properties of the diagonal strut such as the width, stiffness, constitutive behaviour and the number of struts that should be considered. The characteristics of the diagonal strut model vary according to the type of analysis (i.e. linear elastic or nonlinear) and the loading procedure (monotonic, cyclic or transient loading). For example, the required properties for a diagonal strut in case of a linear elastic analysis are the geometric properties of the strut (length and cross-section size) and the modulus of elasticity. However, when the nonlinear behaviour of the material is considered, the complete force-displacement behaviour of the strut is needed instead. Furthermore, the required properties that are required for the diagonal strut become more complex in case of cyclic and dynamic loading.

Tucker (2007) classified available analytical methodologies defining the in-plane properties of strut models into two main approaches: stiffness-based methods and strength-based methods. Both methods replace the infill panel by an equivalent strut but use different approaches to define the necessary properties of the strut. The stiffness-based method estimates the structural contribution of the infill wall based on the development of the compression area along the infill. Therefore, this method focuses on estimating the geometric properties of the strut and associates these properties with equivalent material properties (usually the characteristic compressive strength of the masonry) in order to define the lateral capacity of the infill. Since the strut is a fictitious element and the main objective is to determine the contribution of the infill to the global structural response, the strength-based method, on the other hand, defines the strut properties by quantifying the lateral forces carried by the infill wall. In the following section, the reliability of these methods is discussed by simulating several experimental tests.

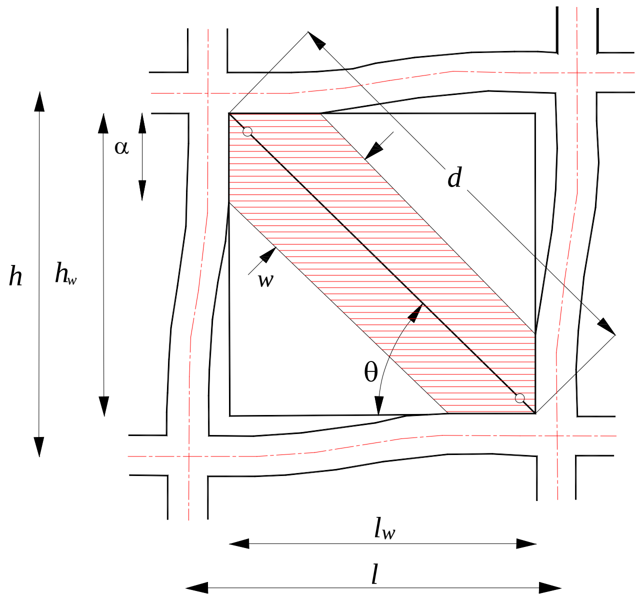


Figure 4 Formulation of the equivalent diagonal strut and its relevant parameters.

3. Reliability of existing strut models

3.1 Stiffness-based approach

As referred before, in this method the infill is modelled using a strut element with properties defined as a function of the geometry of the panel. The structural behaviour of the strut is then defined using a constitutive model that depends on the strength of the masonry. Since the first proposal for a value of the strut width by Holmes (1961) who defined it as one third of the diagonal length of the panel, multiple proposals have been defined. In this section, eight different proposals presented in Table 1 are considered to define the properties of the strut and simulate the experimental behaviour of sixteen fully infilled frame specimens that represent a wide range of structures in terms of the material and type of the masonry. The specimens are termed Specimen 2 (Zhai et al. 2016), Specimen M2 (Pires 1990), Specimen III/2 (Sigmund and Penava 2013), Specimen S and IS(Kakaletsis 2009), Specimen FT1 (Bergami 2007, Bergami and Nuti 2015), Specimen DFS (Basha and Kaushik 2016), Specimen F1 (Stylianidis 2012), Specimen SBF (Misir 2015), Specimen 4,5,6,7, 11 and 12 (Mehrabi and Shing 1996) and Specimen unit1 (Crisafulli 1997).

Table 1 Summary of the expressions considered herein to define the strut width

Model	Expression	Notation and variables
a) Holmes (1961)	$w=\frac{1}{3}d$	d is the diagonal length of the infill panel
b) Mainstone (1971)	$w=0.175\,d\,\lambda_h^{-0.4}$	$\lambda_h=\lambda h_w$, h_w is the infill height and $\lambda=\sqrt[4]{\frac{E_I t\sin 2\theta}{4EIh_w}}$, t is the wall thickness, E_I is the modulus of elasticity of the infill, E is modulus of elasticity of column material and I is the moment of inertia of the column
c) Te-Chang and Kwok-Hung (1984)	$w=\frac{0.95h_w\cos\theta}{\sqrt{\lambda_h}}$	θ is the diagonal strut inclination angle on the horizontal plane
d) Decanini and Fantin (1987)	$w=\begin{cases}\left(\frac{0.748}{\lambda_h}+0.085\right)d & if\ \lambda_h\le 7.85\\ \left(\frac{0.393}{\lambda_h}+0.130\right)d & if\ \lambda_h>7.85\end{cases}$	
e)Moghaddam and Dowling (1988)	$w=\frac{1}{6}d$	
f)Hendry (1990)	$w=0.5\sqrt{\alpha_l^2+\alpha_h^2}$ where α_h and α_l are the horizontal and vertical contact length respectively	$\alpha_h=\frac{\pi}{2}\sqrt[4]{\frac{4(EI)_{beam}h_w}{E_I t\sin 2\theta}}$ $\alpha_l=\frac{\pi}{2}\sqrt[4]{\frac{4(EI)_{column}l_w}{E_I t\sin 2\theta}}$
g) Paulay and Priestley (1992)	$w=\frac{1}{4}d$	
h) Durrani and Luo (1994)	$w=\gamma\,d\sin(2\theta)$ where γ is a variable	$m=6\left(1+\frac{6E_bI_bh}{\pi E_cI_cL}\right)$ $\gamma=0.32\left[\frac{h^4E_wt_w}{mE_cI_ch_w}\right]^{-0.1}\left[\sqrt{\sin 2\theta}\right]$

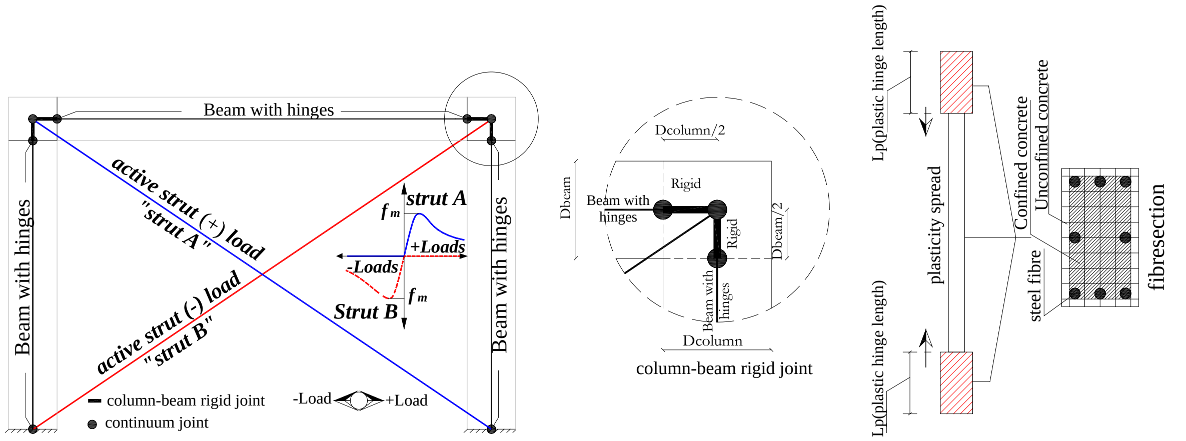
In order to assess the reliability of the several stiffness-based procedures, the numerical simulation of the experimental tests corresponding to the selected sixteen specimens was performed using the software OpenSees (McKenna et al. 2000). The RC frame elements (i.e. beams and columns) were modelled using force-based elements considering fibre-sections (also known as the *Beam with Hinges* element). The Modified Radau Hinge Integration method (Fenves and Scott 2006, Scott and Ryan 2013) was the selected plastic hinge integration method to assign inelastic actions at the end regions of the element with a specified length. Still, additional fibre sections were also considered in the central part of the element to model its possible nonlinearity since recent modifications in this element (Scott, *et al.*, 2013) allow plasticity to be extended beyond the length of the plastic hinges.

The fibre discretization of the RC cross sections considered the concrete cover modelled using the concrete model termed Concrete01 in OpenSees representing the uniaxial concrete material with degraded linear unloading/reloading stiffness in compression and no tensile strength. Confined concrete was modelled using a confinement factor determined based on the expression proposed by Kent and

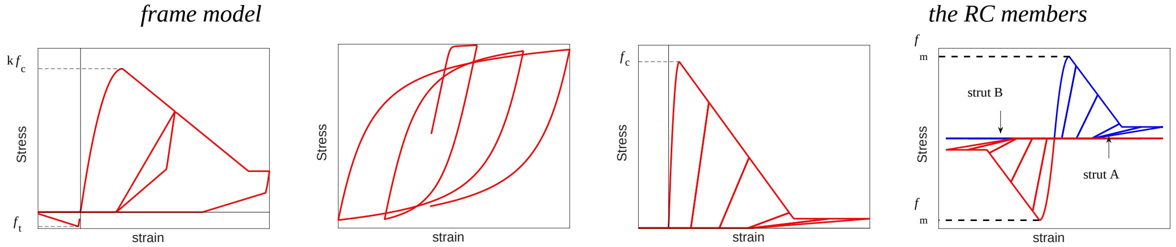
Park (1971) associated with the Concrete02 model. The Concrete02 concrete model is similar to the Concrete01 but considers the tensile strength of the concrete. Steel reinforcing bars were modelled using the uniaxial Giuffre-Menegotto-Pinto model (Menegotto and Pinto 1973) with isotropic hardening, termed Steel02 in OpenSees, with the default parameters proposed by the software. For the beam-column joints, a rigid end-offset joint model was used (Mondal and Jain 2008). The lengths of the rigid parts were considered to be half of the depth of the perpendicular element. The infills were modelled using a single compressive strut element with an area evaluated based on the previous expressions and the constitutive model for masonry was defined based on the model proposed by Hendry (1990) which matches the shape of the Concrete01 constitutive model. The constitutive model proposed by Hendry (1990) is given by the following expression:

$$\sigma_m = f_m^{'}\left[2\frac{\varepsilon_m}{\varepsilon_{crm}} - \left(\frac{\varepsilon_m}{\varepsilon_{crm}}\right)^2\right] \tag{1}$$

where ε_m and σ_m are the compressive strain and the corresponding compressive stress of the masonry, respectively, $f_m^{'}$ is the maximum compressive strength of the masonry and ε_{crm} is the compressive strain at the onset of failure, which according to (Dolšek and Fajfar 2008) ranges from 0.0015 to 0.002. In these analyses, the value of ε_{crm} was considered to be 0.002 in all models. Figure 5 shows the general description of the model implemented in OpenSees for the RC frame and the infill panel in addition to the detailed description of the RC element model. To be consistent with the experimental tests, all models were first analysed for a preliminary vertical loading and then followed by a cyclic lateral loading according to the loading protocol of each experimental campaign.



a) General description of the infilled frame model *b) Beam-column joints* *c) Beam with hinges element used for the RC members*



Confined concrete material (Concrete02) Steel material (Steel02) Unconfined concrete (Concrete01) Strut material

d) Material models

Figure 5 Description of the implemented model for the infill panel using a stiffness-based approach.

Figure 6 shows the values of *w/d* (width of the strut over the length of the diagonal strut) that were obtained using the eight expressions of Table 1 for the sixteen experimental specimens. The considered procedures are denoted as: (a) Holmes (1961), (b) Mainstone (1971) (c) Te-Chang and Kwok-Hung (1984), (d) Decanini and Fantin (1987), (e) Moghaddam and Dowling (1988), (f) Hendry (1990), (g)

Paulay and Priestley (1992) and (h) Durrani and Luo (1994) in all the following figures. It is can be seen that, for most specimens, the *w/d* ratio ranges between 0.1 and 0.4. Exceptions are found for specimens S.S, S.F1, S.III/2 and S.SBF which present higher *w/d* ratios when using the expression denoted as (d). The selected stiffness-based procedures are also seen to lead to a wide range of diagonal strut widths.

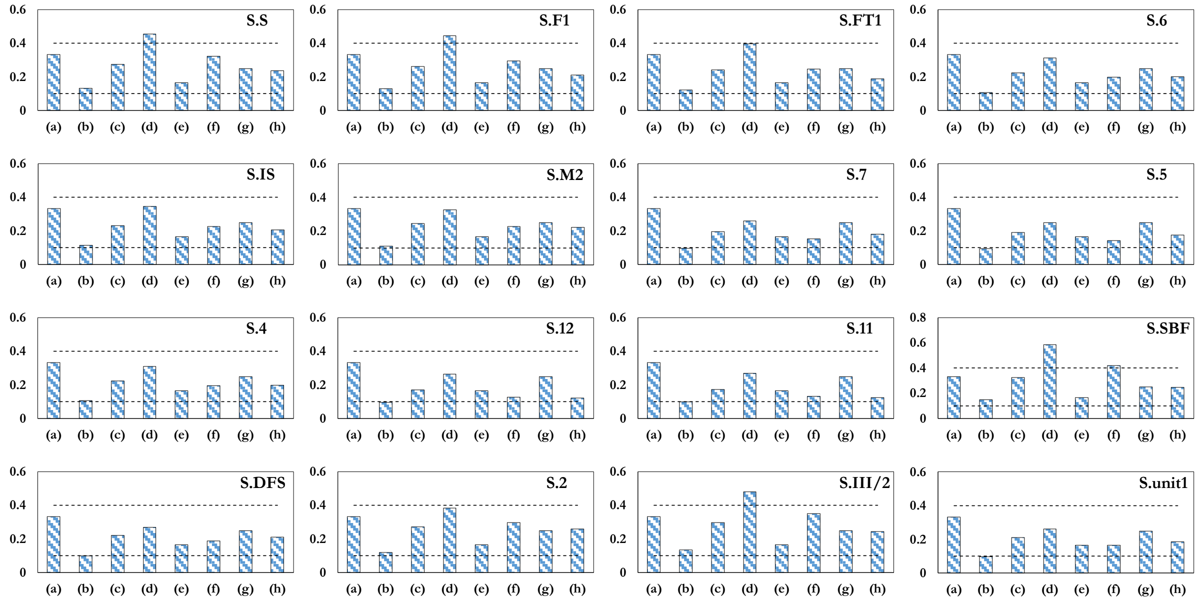


Figure 6 The *w/d* ratios obtained from the eight-different stiffness-based procedures for the sixteen specimens

From the numerical simulation of the experimental tests, the initial stiffness and maximum strength of each specimen were obtained from the cyclic responses. Ratios of numerical over experimental initial stiffnesses and maximum strengths were then determined for all the specimens involving all the selected stiffness-based procedures. Figure 7 and Figure 8, show the ratios between the numerical and experimental data of initial stiffness and numerical maximum strength, respectively, with a reference line corresponding to a unit value ratio. For the initial stiffness results, a large variability was found. This variability of the ratios means that each procedure may significantly underestimate or overestimate the initial stiffness for any specimen which is a direct reflection of the large variability of the estimated strut widths (see Figure 6). In terms of the maximum strength, all the selected stiffness-based procedures overestimated significantly the maximum experimental strength value. This result can be interpreted as corresponding to a significant overestimation of the strut area at the strain level leading to the maximum lateral force. By comparing the performance of the same formula in both parameters i.e. stiffness and strength, it can be seen that procedures with better performance when estimating the initial stiffness will have a worse performance when estimating the maximum strength, and vice-versa. For example, the procedure by Mainstone (1971) has the best performance when estimating maximum strength among all the procedures, but exhibits one of the worst performances when estimating initial stiffness. Based on these observations, it can be deduced that the effective area of the equivalent strut decreases as the lateral displacement of the structure increases due to two main factors: the reduction of the contact length between the panel and the frame, and due to the cracking of the masonry infill. Therefore, it can be concluded that using a fixed geometry-based definition for the strut element can only be useful for elastic analysis. In this case, the model denoted as (c) provides a reasonable estimate of the initial stiffness (the average ratio between estimated and experimental initial stiffnesses is 1.06 with a coefficient of variation of 0.87). Thus, for performance-based analyses requiring a nonlinear model, these proposals should be combined with another model that would simulate a change in the strut geometry during the loading history, either by defining an area reduction factor function of lateral displacements (e.g. see (Crisafulli and Carr 2007)) or by using a constitutive material reflecting this phenomenon, as presented next.

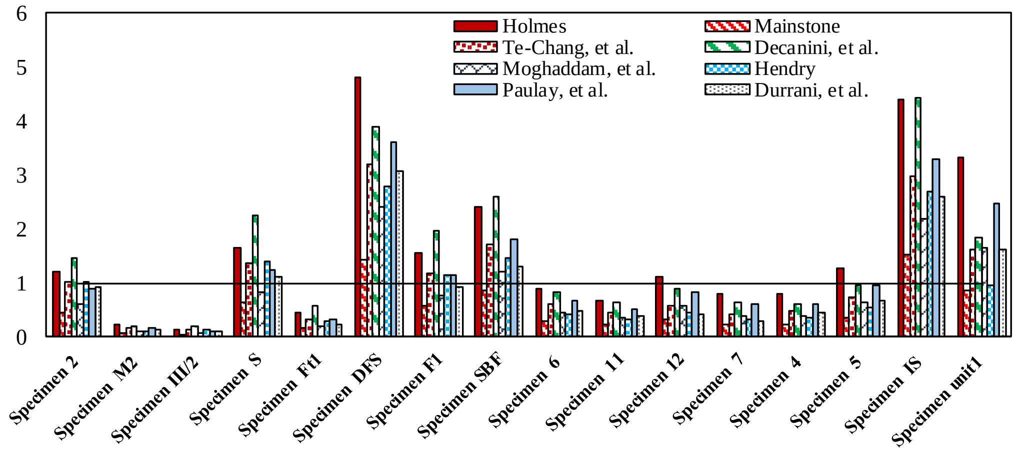


Figure 7 Ratios between the numerical initial stiffness and the initial stiffness obtained from the experimental data for sixteen specimens

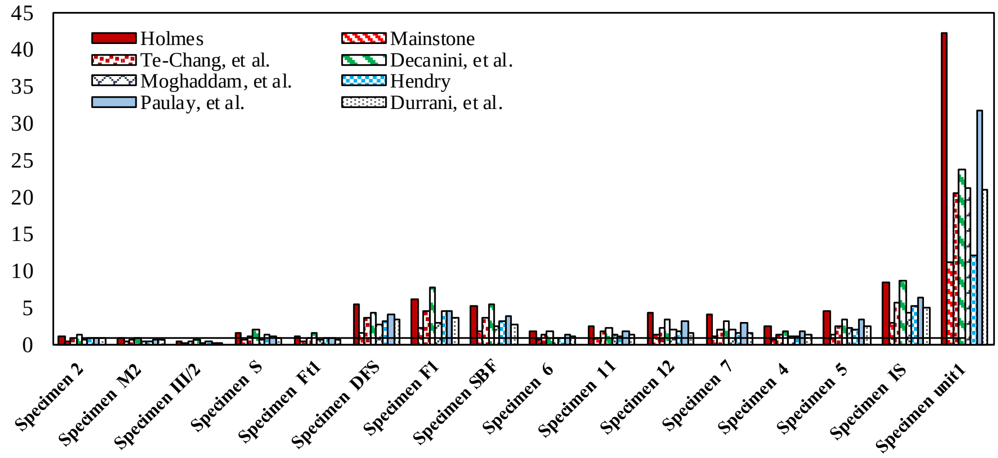
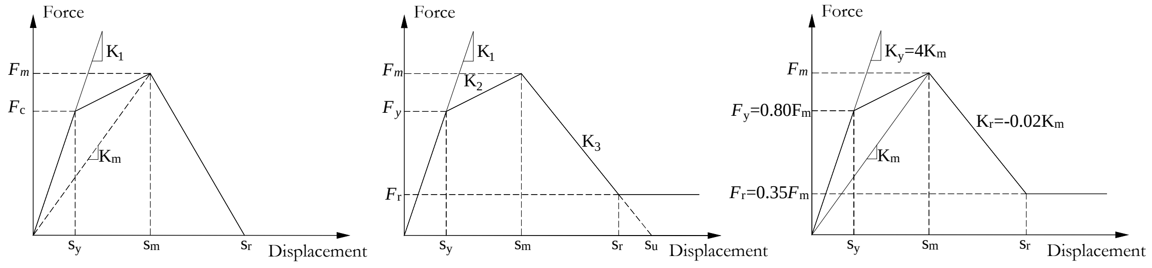


Figure 8 Ratios between the numerical maximum strength and the maximum strength obtained from the experimental data for sixteen specimens

3.2 Strength-based approach

Based on the results that were obtained from the performance analysis of the selected stiffness-based procedures, it can be seen that none of the procedures is able to globally represent the behaviour of the infills. In particular, all the procedures provide initial stiffness estimates that may significantly underestimate or overestimate the real initial stiffness while providing, in most cases, maximum strength estimates that significantly overestimate the real maximum strength. The fact that these procedures are unable to account for the reduction in the effective strut area as the lateral displacement increases was also seen to be an important factor in their lack of accuracy. In addition, these procedures also assume that the infill panel does not exhibit any failure mechanism other than crushing by excessive compression (e.g. such as shear failure in the mortar joints or diagonal tensile failure).

Strength-based procedures are alternative methods that define a behaviour model for an infill wall. These procedures directly establish a force-displacement relation that represents the behaviour of the infill under lateral loading. As carried out for the stiffness-based procedures, the performance of three empirical methods is analysed herein. Defining the force-displacement relation representing the behaviour of the strut element involves determining the evolution of forces transferred through the infill panel based on the (expected) governing failure mechanism. In this section, the three different procedures shown in Figure 9 were considered to define the force-displacement relation of the strut element. These procedures are those proposed by a) Dolšek and Fajfar (2008), b) Panagiotakos and Fardis (1994) and c) Bertoldi et al. (1993). These models were selected among the several proposals found in the literature to define the force-displacement relation. In particular, these models were selected because they provide a complete description of the force-displacement relation using explicit expressions, a fact that led several researchers to use these models (e.g. see (Sattar and Liel 2010, Celarec et al. 2012, Ricci et al. 2013, Furtado et al. 2016, Ricci et al. 2016) among others).



a) b) c)

$$F_m = 0.818\frac{L_w t_w f_{tp}}{C_I}\left(1+\sqrt{C_I^2+1}\right),$$

$$C_I = 1.925\frac{L_w}{h_w}, K_1 = \frac{G_w L_w t_w}{h_w}$$

$$f_{tp} = 0.17 \sim 0.38\sqrt{f_m^{'}}, G_w = 0.40E_m$$

$$F_y = f_{tp} t_w L_w \quad , \quad K_2 = \frac{E_m b_w t_w}{d},$$

$$b_w = 0.175\, d\, \lambda_h^{-0.40}$$

$$K_m = \frac{E_m b_w t_w}{d}\cos^2\theta ,$$

$$F_m = \min(\sigma_{centrer}, \sigma_{corner}, \sigma_{sliding}, \sigma_{diagonal}) t_w b_w$$

where $f_m^{'}$ is the masonry compressive strength in MPa, d is the diagonal length of the infill and b_w is the width of the strut, σ_{centre} crushing stress at the centre of the panel, σ_{corner} crushing stress of the panel corner, $\sigma_{silding}$ sliding stress at the horizontal mortar joints and diagonal tensile stress $\sigma_{diagonal}$

Figure 9 Force-displacement trilinear curve and their parameters for the strut element according to three proposals: a) Dolšek and Fajfar (2008), b) Panagiotakos and Fardis (1994) and c) Bertoldi et al. (1993)

The selected procedure for modelling the RC elements is similar to the one presented previously. However, implementing these trilinear force-displacements for a compression-only strut was only possible using two active struts since using zero branches for tensile behaviour (that may lead to numerical issues) or by adding a zero-length element to deactivate the strut for tensile loading. The use of two active struts may, however, affect the local response that is obtained, especially for the edge column as shown in Figure 10. On other hand, using extra elements (i.e. using extra zero-length elements) increases the computational cost of the model. Since the main scope of this paper is to assess the reliability of using a strut element to capture the global structural response, the struts were modelled using truss elements associated with the Pinching4 material of OpenSees. The ratios between the obtained numerical initial stiffness and maximum strength to the corresponding experimental values are plotted in Figure 11 and Figure 12, respectively. It is worth noting that the initial stiffness was computed based on the effective panel width (i.e. subtracting the perforated area) to obtain a better result for that parameter. Based on the presented results, it is seen that strength-based procedures provide alternative approaches to establish the parameters of the strut model and simulate the behaviour of the infill with a performance that is generally better than that of the stiffness-based procedures previously analysed, especially for the maximum strength. However, strength-based procedures also establish the infill strength based on an (assumed) governing behaviour mechanism. Even though a large part of the global behaviour of the infill may be governed by one behaviour mechanism, infill panels experience several behaviour mechanisms that are globally interconnected and responsible for transferring forces in different ways. Therefore, assuming that only one mechanism controls the behaviour of the infill panel inevitably leads to differences between the numerical prediction and the real behaviour of the infill which is revealed by the presented figures.

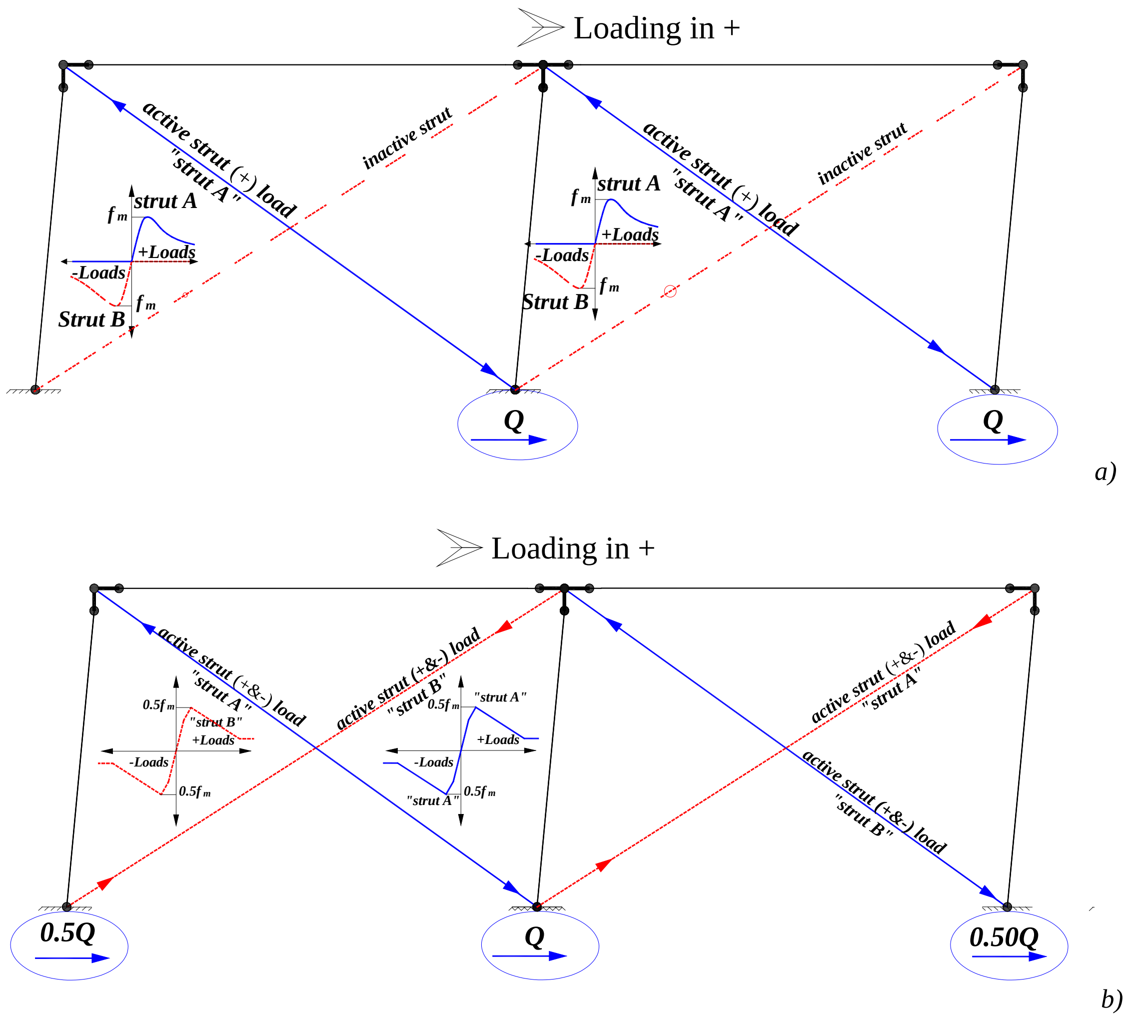


Figure 10 Shear force at the base due to the infill when using a compression-only strut (a) and when using a compression-tension strut (b)

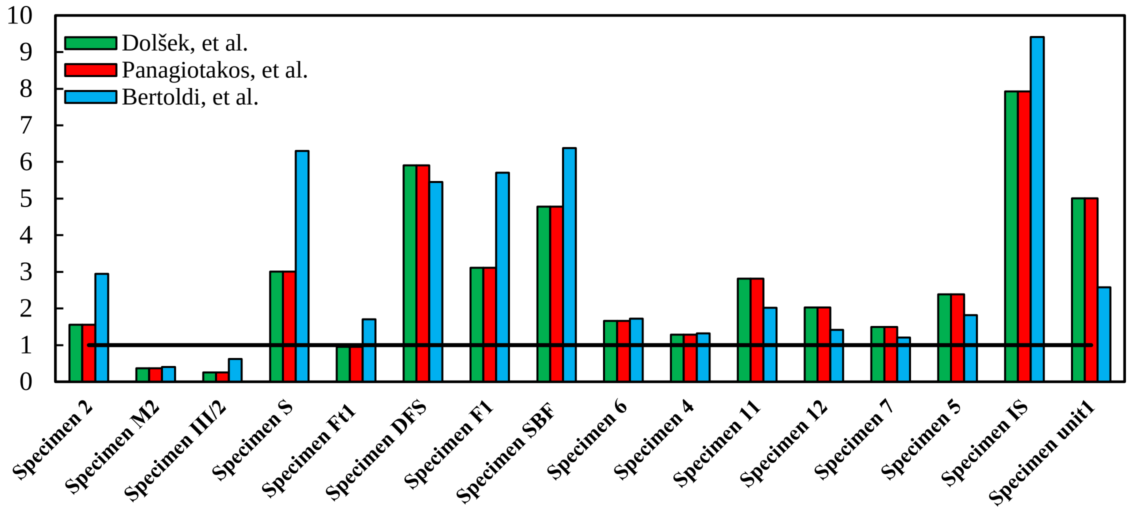


Figure 11 Ratios between the numerical initial stiffness and the initial stiffness obtained from the experimental data for sixteen specimens using the effective wall thickness

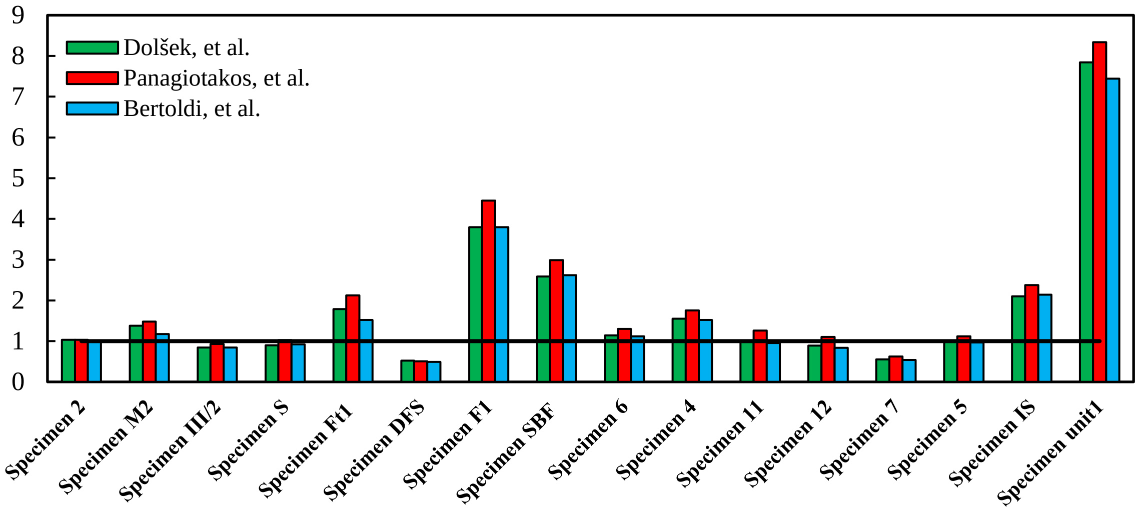


Figure 12 Ratios between the numerical maximum strength and the maximum strength obtained from the experimental data for sixteen specimens

4. CONCLUSION

The current paper reviewed procedures to model the behaviour of masonry infill panels. The use of simplified strut models was found to be more feasible for performance-based studies which require a high number of analyses. The performance of existing models involving stiffness-based and strength-based procedures to establish adequate values for the parameters needed to simulate the behaviour of masonry infills using the single strut modelling approach was reviewed. Regarding the eight selected stiffness-based procedures, their main hypothesis is that it assumes that an infill panel works as a constant area member under compression loads throughout the entire loading history. This assumption was seen to lead to large errors in predicting both the maximum lateral strength and the initial stiffness of the infill. As such, accounting directly or indirectly for the change in geometry of the actively loaded area of the masonry panel throughout the loading history is fundamental to obtain an adequate representation of the nonlinear behaviour of masonry infills. Regarding the performance of the three selected strength-based procedures, the results obtained were seen to involve better predictions of the maximum lateral strength and of the initial stiffness of the infill. However, to obtain more realistic predictions, it is recommended to compute the infill stiffness using the infill's effective thickness instead of using the wall's gross (real) thickness.

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