Earthquakes and Structures, *Vol. 6, No. 4 (2014) 409-436*
DOI: http://dx.doi.org/10.12989/eas.2014.6.4.409

Seismic response of masonry infilled RC frames: practice-oriented models and open issues

Carmine Lima[a], Gaetano De Stefano[b] and Enzo Martinelli[*]

Department of Civil Engineering, University of Salerno, 84084 Fisciano (SA), Italy

(Received January 31, 2013, Revised December 16, 2013, Accepted January 13, 2014)

Abstract. Although it is widely accepted that the interaction -between masonry infill and structural members significantly affects the seismic response of reinforced concrete (RC) frames, this interaction is generally neglected in current design-oriented seismic analyses of structures. Moreover, the role of masonry infill is expected to be even more relevant in the case of existing frames designed only for gravitational loads, as infill walls can significantly modify both lateral strength and stiffness. However, the additional contribution to both strength and stiffness is often coupled to a modification of the global collapse mechanisms possibly resulting in brittle failure modes, generally related to irregular distributions of masonry walls throughout the frame. As a matter of principle, accurate modelling of masonry infill should be at least carried out by adopting nonlinear 2D elements. However, several practice-oriented proposals are currently available for modelling masonry infill through equivalent (nonlinear) strut elements. The present paper firstly outlines some of the well-established models currently available in the scientific literature for modelling infill panels in seismic analyses of RC frames. Then, a parametric analysis is carried out in order to demonstrate the consequences of considering such models in nonlinear static and dynamic analyses of existing RC structures. Two bay-frames with two-, three- and four-storeys are considered for performing nonlinear analyses aimed at investigating some critical aspects of modelling masonry infill and their effects on the structural response. Particularly, sensitivity analyses about specific parameters involved in the definition of the equivalent strut models, such as the constitutive force-displacement law of the panel, are proposed.

Keywords: masonry infill; nonlinear analysis; existing structures; reinforced concrete; strut models

1. Introduction

A significant part of the overall existing building stock in earthquake-prone areas, such as Italy and the Mediterranean basin, was constructed in the first two or three decades after World War II (Ricci *et al.* 2011). Although reinforced concrete (RC) frames are recognised as one of the most common structural form for low- to medium-rise buildings in the world, such buildings were generally designed and constructed according to the code provisions of that time and without considering any possible seismic-induced effects (Asteris 2003). Moreover, such structures are

[*]Corresponding author, Assistant Professor, E-mail: e.martinelli@unisa.it
[a]Research Assistant, E-mail: clima@unisa.it
[b]Ph. D., E-mail: gaetano.destefano@gmail.com

http://www.techno-press.org/?journal=eas&subpage=7 ISSN: 2092-7614 (Print), 2092-7622 (Online)

often characterised by the presence of masonry infill panels that, according to the general practice of the time, are in contact with the frame and actually participate with the RC frame elements in the global seismic response of the structure. In fact, reports on the failure of infilled RC frame structures observed under devastating earthquakes occurred in various regions of the world (such as Chi-Chi, Taiwan, in 1999, Kocaeli, Turkey, in 1999, Central Peru in 2007, and Sichuan, China, in 2008) revealed that ignoring the infill effects in structural analyses may result in significant underestimation of the effects induced by the expected seismic shakings (Kuang and Yuen 2010). Furthermore, the seismic events recently occurred in Italy sadly emphasised the significant levels of vulnerability of such a part of the existing building stock and pointed out several critical aspects of their seismic response characterised by partial or total collapses, brittle failures of critical members and significant damage of non-structural components (Uva *et al.* 2012). Particularly, one of the most important lessons learned by the observed damage due to the 2009 L'Aquila Earthquake is the significant role often played by masonry infill in influencing the seismic response of RC frames and the actual level of safety of buildings (Verderame *et al.* 2011).

Although several capacity models aimed at simulating the behaviour of RC members (Paulay and Priestley 1992) and beam-to-column joints (Lima *et al.* 2012a, 2012b) allow for accurate predictions of the seismic response of bare RC frames, the influence of masonry infill is still a subject under investigation (Kakaletsis and Karayannis 2008). Particularly, in spite of the different proposals currently available in the scientific literature for simulating the in-plane behaviour of masonry infilled RC structures, none of such models can be actually considered as a well-established. Moreover, few studies are actually available about the behaviour of masonry infilled frames under seismic actions (Yuen and Kuang 2012). As a matter of fact, the European Standards (EN 1998-3 2005) and the Italian Code (Ministerial Decree 2008) generally account for masonry infill panels as non-structural elements and only provide general criteria for taking into account infill in the structural model if they significantly contribute to the lateral stiffness and strength of the structure (section 4.3 EN 1998-3 (2005) and section 7.2.6 NTC 2008 (Ministerial Decree 2008)). However, no specific rules about how infill should be modelled in global seismic analysis of frames are outlined. Recently, some modelling or design methods of infilled frames were implemented in codes of standards such as ASCE41 (ASCE 2006).

This paper firstly presents a review of two among the most recent proposals available in the scientific literature for modelling the in-plane response of masonry infill and its interaction with the global seismic behaviour of RC frames. Particularly, Section 2 reports a critical State-of-the-Art review of the above mentioned models for nonlinear seismic analysis of RC infilled frames, with emphasis on those based on the "equivalent strut" concept and conceived for practice-oriented analyses. Then, Section 3 describes the structures considered as case studies for comparing the response simulated by the various models and assessing the effect of masonry infill on the seismic response of RC frames. Such structures are characterised by both variable distribution of masonry walls and variable number of stories. The results of nonlinear analyses are considered for understanding the actual role of masonry infill on the seismic response in terms of both displacements and forces. Section 4 reports such results obtained in static analyses, whereas Section 5 provides Readers with the key observations arising from time-history analyses. The key findings of such analyses, highlighted in Section 6, point out the role of masonry infill in influencing the seismic response of existing RC frames and emphasise some critical aspects about the result sensitivity to the choice of the modelling approach.

2. State-of-the-art

In the past decades, plenty of researches were conducted to investigate the behaviour of RC structures infilled by masonry walls. They pointed out some open issues related to the definition of relevant parameters and the complexity of the models that are needed to simulate the behaviour of masonry infills, generally neglected in the design of a seismic structures. Basically, two main classes of models were proposed by various authors for modelling masonry infill in RC frames: 2D models, generally based on Finite Element (FE) simulations carried out at different scales, and more simplified ones, based on the concept of "equivalent strut".

It should be noted that, in order to perform nonlinear seismic analyses, 2D models of infill panels require high computational efforts. In principle, in such models the actual behaviour of masonry infill in RC frames can be simulated by looking after both geometric the properties of masonry walls (possibly considering openings) and the cyclic behaviour of materials through 2D FE models (Fiore *et al*. 2012). However, the complex interaction between masonry walls and RC members, and the even more complex behaviour of masonry components (i.e. bricks and mortar) lead to a wide variety of possible modelling assumptions characterised by variable computational effort and accuracy ambitions (Biondi *et al*. 2000). One of the most radical choices for simulating the behaviour of masonry and its connection with the RC members can be possibly based on the so-called "micro-modelling" (Hao *et al*. 2002). In this case, the behaviour of the single components (i.e., brick element, mortar layers and RC members) is modelled along with the explicit simulation of the interfaces between them, where the possible fracture phenomena is supposed to develop (Ali and Page 1998). However, this approach is generally feasible for analysing the local behaviour of small substructures. On the contrary, the use of such models in global seismic analyses of entire frames would be computationally demanding and time-consuming.

Alternative models based on nonlinear plasticity-based 2D finite elements are generally characterised by higher efficiency, though losing the direct and explicit interpretation of phenomena specifically related to the masonry behaviour and its interaction with the framing RC members (Page 1978). Although this approach is certainly more feasible and efficient than the above mentioned micro-models, its use in nonlinear dynamic analysis is still problematic and computationally intensive, as the full description of the cyclic behaviour of 2D plasticity- or fracture-based elements is certainly a challenging task.

Since 2D FE models are generally unfit for practice-oriented analyses of the seismic response of RC frames, several macro-models were proposed and are currently available in the scientific literature to simulate the global seismic response of masonry infilled RC frames. Such models are generally based on the assumption of two diagonal equivalent struts connecting the two opposite corners of the structural frame cell (Fig. 1). Although, in principle, such macro-modelling approaches generally result in effective simplifications, the calibration of equivalent struts is not an easy task. Particularly, their equivalent width and depth are the main geometric parameters, and the definition of a nonlinear stress-strain (or force-displacement) relationship, possibly accounting for stiffness and strength degradations under cyclic actions, is a challenging task.

Therefore, the description of the mechanical response of the equivalent strut is not straightforward, though several macro-models were proposed and are currently available in the scientific literature to be employed for the global seismic analysis of masonry infilled RC frames. Alternative and rather diverse proposals are currently available in the scientific literature to define the width w of the equivalent strut. The width of the strut is often defined in terms of ratio between *w* and the diagonal length d of the infill panel (Holmes 1961, Paulay and Priestley 1992, Penelis and Kappos 1997) defined by the actual geometric configuration of the RC frame (Fig. 2).

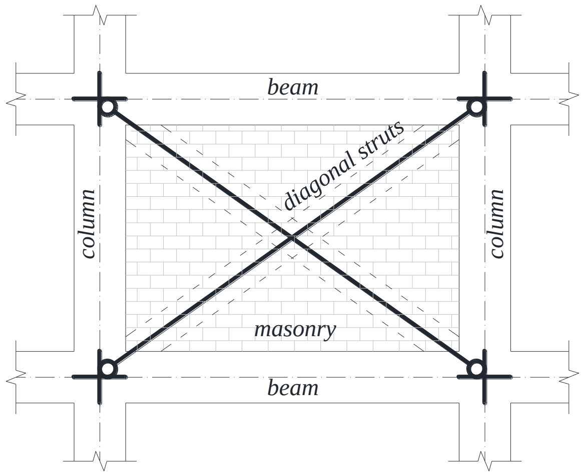


Fig. 1 Macro-modelling of masonry infill through equivalent struts

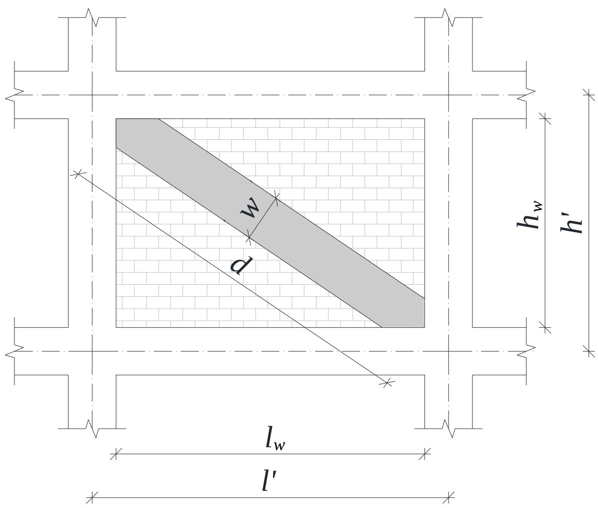


Fig. 2 Relevant geometric parameters of the equivalent strut

Less simple theoretical proposals (Smith 1966, Mainstone 1971, Klingner and Bertero 1978, Liauw and Kwan 1984, Durrani and Luo 1994) available in the scientific literature take into account the level of degradation and cracking of the infill. Such models are often based upon the relative masonry-column stiffness parameter λ

$$\lambda = \sqrt[4]{\frac{E_w t_w \sin 2\theta}{4 E_c I_c h_w}} \tag{1}$$

in which θ is the slope angle of the infill's diagonal defined as follows

$$\theta = \tan^{-1}\left(\frac{h_w}{l_w}\right) \tag{2}$$

and where E_w and E_c are the elastic moduli of masonry and concrete, respectively, t_w is the infill thickness, I_c is the moment of inertia of the column, and h_w and l_w are the depth and the length of the masonry wall, respectively.

Moreover, Papia and Cavaleri (2001) gave a significantly innovative contribution to the issue of simulating the effect of masonry on the response of RC frames under horizontal seismic-induced actions. Their model led to define the "equivalent strut" width which is not only based on the flexural stiffness of RC and masonry members, but takes into account the axial stiffness of RC members and the shear stiffness of the masonry wall (the latter being considered through the Poisson ratio ν which is needed, in principle, to express the elastic relationship between Young modulus E_w and shear modulus G_w of masonry). In fact, experimental results reported in Papia and Cavaleri (2001) led to the following definition of the width-to-length ratio *w/d* of the equivalent strut

$$\frac{w}{d} = \frac{c}{z}\frac{1}{\lambda_c^{\beta}} \tag{3}$$

whose key parameters β, λ_c, c and z were expressed as follows

$$\lambda_c = \frac{E_w}{E_f}\frac{t_w h'}{A_c}\left(\frac{h'^2}{l'^2} + \frac{1}{4}\frac{A_c}{A_b}\frac{l'}{h'}\right) \tag{4}$$

$$c = 0.249 - 0.0116 + 0.567\nu^2 \tag{5}$$

$$\beta = 0.146 + 0.0073 + 0.126\nu^2 \tag{6}$$

$$z = \begin{cases} 1 & if \quad \dfrac{l'}{h'} = 1 \\ 1.125 & if \quad \dfrac{l'}{h'} = 1.5 \end{cases} \tag{7}$$

Further detailed models were proposed for taking into account openings (windows or doors) in masonry infills. In particular, the model by Papia and Cavaleri (2001) described in Eq. (3) to Eq. (7) provides specific rules for simulating the effects of openings. As a matter of fact, openings results in a significant reduction of both lateral stiffness and strength which was measured by means of both numerical and experimental studies. Such investigations led to defining a reduction factor $r=w_v/w$ between the width w_v of equivalent strut representing the masonry wall with openings and the corresponding one w possibly derived through Eq. (3) for a full masonry infill. Then, a simple analytical relationship was calibrated to express r as a function of the geometric parameter a, actually defined as the ratio between the opening dimensions h_v and l_v and the corresponding dimensions h_w and l_w of the masonry wall under consideration (Fig. 3)

$$a = {h_v}/{h_w} = {l_v}/{l_w} \tag{8}$$

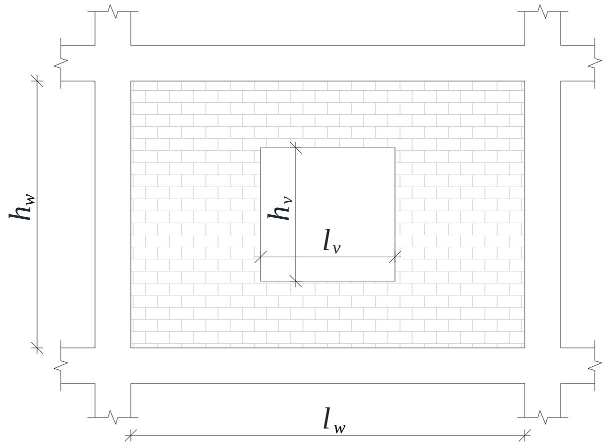


Fig. 3 System frame/panel with opening

$$r=1.24-1.7a \tag{9}$$

2.1 Modelling nonlinear response of RC infilled frames

Full nonlinear force-displacement relationships need to be defined for the equivalent struts to be possibly employed in RC frame models, potentially fit for performing nonlinear (either static or dynamic) seismic analyses. Particularly, the progressive degradation of the stiffness and strength during cyclic loading processes should be accurately taken into account. Thus, besides the various alternative definitions of *w/d* ratio listed in the previous subsection to describe, in principle, the lateral stiffness of equivalent struts, further formulations are needed to describe the response of equivalent struts in compression and possibly simulate their hysteretic response under cyclic actions by taking into account both inelastic displacement and damage. The following subsections outline two approaches currently available in the scientific literature to describe the nonlinear response of infilled RC frames: the model by Panagiotakos and Fardis (1996) and the one by Dolsek and Fajfar (2008). Those models will be used in the present paper for developing the analyses on the case study.

2.1.1 The model by Panagiotakos and Fardis (1996)

On the basis of a previous experimental parametric study on the cyclic behaviour of four-storey masonry infilled RC frames tested at Elsa laboratory in Ispra (Panagiotakos and Fardis 1994), Panagiotakos and Fardis (1996) proposed a nonlinear force-displacement relationship to describe the response of equivalent struts. The proposed skeleton curve is step-wise linear in shape to describe the behaviour of the strut in compression. It was directly derived by comparing the experimental results on masonry walls, bare RC structures and the corresponding infilled frames. The ultimate strength of the infills was obtained as 1.3 times the cracking force estimated from test results on walls in diagonal compression and corresponded to a base shear coefficient of 0.08, while the contribution of their elastic uncracked stiffness to the overall lateral stiffness of the structure was about 24 times the lateral stiffness of the fully cracked bare frame, with all its members considered with their secant stiffness at yielding. Particularly, the proposal by

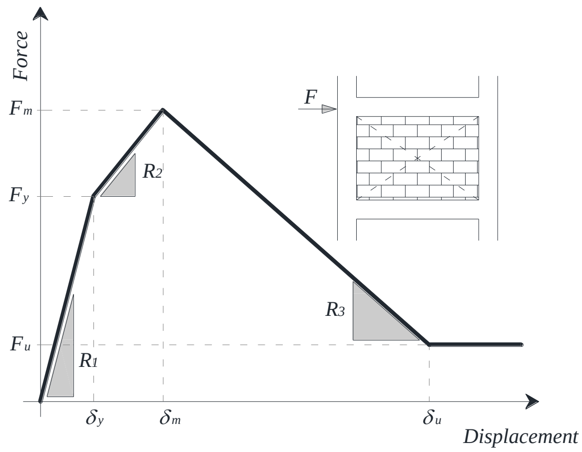


Fig. 4 Force-displacement curve according to the model by Panagiotakos and Fardis (1996)

Panagiotakos and Fardis (1996) is characterised by the following stress states (Fig. 4)

- initial elastic behaviour of the uncracked masonry wall;
- post-elastic linear response, characterised by a reduced value of stiffness;
- softening response of the panel after the maximum force;
- residual axial strength after a given value of displacement.

Thus, six parameters are strictly needed to describe the non-linear skeleton curve according to the proposal under consideration. In fact, the initial shear stiffness R_1 of the uncracked panel can be defined as follows

$$R_1 = \frac{G_w t_w l_w}{h_w} \tag{10}$$

where G_w is the shear modulus of the masonry infill obtained in diagonal-compression tests while t_w, l_w and h_w are the thickness, the length and the height of the masonry wall. Then, the load value at the onset of cracking, namely yielding force F_y, is defined as follows

$$F_y = f_{ws} t_w l_w \tag{11}$$

in which f_{ws} is the tensile strength evaluated by diagonal-compression tests.
Moreover, the axial post-cracking stiffness R_2 can be defined according to the following equation

$$R_2 = \frac{E_w t_w w}{d} \tag{12}$$

in which E_w is the Young modulus of masonry and the following equation defined by Klingner and Bertero (1978) can be adopted for evaluating the width w of the equivalent strut

$$w = 0.175(\lambda h_w)^{-0.4} d \tag{13}$$

where λ is defined through Eq. (1) and

$$d = \sqrt{h_w^2 + l_w^2} \tag{14}$$

The post-cracking branch keeps growing up to the maximum force F_m

$$F_m = 1.3F_y \tag{15}$$

Then, a post-peak softening branch follows, whose (negative) stiffness R_3 can be assumed within the range 0.005 $R_1 < R_3 < 0.1\ R_1$ and the residual strength F_u can range between $0 < F_u < 0.1F_y$. In order to guarantee numerical stability in nonlinear simulation, values greater than zero are recommended for F_u. The combination of infill parameters reported above resulted in the best agreement with the available monotonic and cyclic test results obtained on infill panels. However, it is worth highlighting that the use of the model by Panagiotakos and Fardis (1996) does not take into account infills with opening, thus its application is limited to solid walls.

2.1.2 The model by Dolsek and Fajfar (2008)

More recently, Dolsek and Fajfar (2008) proposed an alternative model to simulate the nonlinear behaviour of masonry infill in RC frames. In fact, the model is formally similar to the one by Panagiotakos and Fardis (1996) with the only key difference of assuming zero residual strength ($F_u = 0$). Alternative analytical expressions were assumed to describe the nonlinear force-displacement curve (Fig. 5). Particularly, the initial stiffness is defined as R_1 in Eq. (10), while the maximum force F_m is defined according to a proposal by Zarnic and Gostic (1997)

$$F_m = 0.818\,\frac{f_{ws} t_w l_w}{C_I}\left(1+\sqrt{C_I^2+1}\right) \tag{16}$$

$$C_I = 1.925\,\frac{l_w}{h_w} \tag{17}$$

where the meaning of the symbols is reported in the notation section.

The other parameters required to define the curve in Fig. 5 can be determined through simple analytical relationships. Then, the force F_y at the onset of cracking is assumed to be equal to 0.6 F_m whereas the maximum horizontal displacement δ_m is defined as a percentage of the wall height (0.2% for plain walls, 0.15% for walls with a window and 0.10% for walls with a door opening).

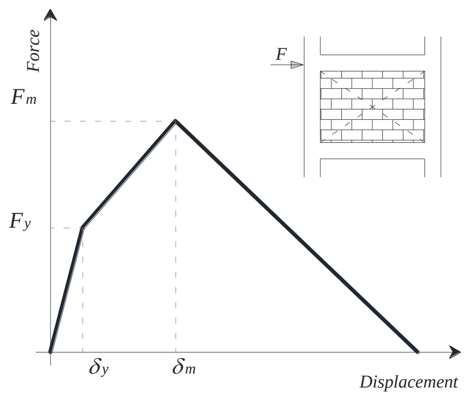


Fig. 5 Force-displacement curve according to the model by Dolsek and Fajfar (2008)

Furthermore, the role of openings on the whole nonlinear response was taken into account by introducing the following λ_0 parameter

$$\lambda_0 = 1 - \frac{1.5L_0}{l_w} \geq 0 \tag{18}$$

where L_0 is the total horizontal projection of openings in the wall under consideration. Then, the model by Dolsek and Fajfar (2008) foresees a reduction in both the initial stiffness and the yielding force which can be easily quantified by reducing Eq. (10) and Eq. (11) as follows

$$R_1 = \lambda_0 \frac{G_w t_w l_w}{h_w} \tag{19}$$

$$F_y = 1.3\lambda_0 f_{ws} t_w l_w \tag{20}$$

Finally, the slope of the softening response R_3 is evaluated through the model by Panagiotakos and Fardis (1996) outlined in the previous subsection and the residual force F_u is assumed to be 0 (Fig. 5). As a limitation, this approach does not distinguish between window and door openings, as well as it does not allow taking into account additional concrete elements around openings.

3. Description of the structures considered as case-studies

A two-storey two-bay RC frame described in the scientific literature (Di Sarno *et al*. 2008) is considered in this paper as a starting point of a wide numerical analysis. The structure under consideration was only designed for gravitational loads, according to the codes and general practice in use in Italy in ’70 of the past century. Typical details for gravity load design, i.e., smooth bars, intermediate concrete compression strength, hooks and large spacing stirrups, were taken into account. The key geometric properties of the structure under consideration are outlined in Fig. 6. Further information are available in Di Sarno *et al*. (2008).

No masonry infill are actually present within the structure. Thus, it was firstly analysed in its “as-built” configuration (denoted as “bare structure” in the following) with the aim of calibrating the numerical models employed for structural members. Then, the possible influence of masonry infill was investigated by considering the three alternative configurations depicted in Fig. 7. Particularly, the configuration called “Type A” corresponds to accounts for fully infilled frames, the “Type B” refers to buildings in which one bay is infilled, while the configuration referred to as “Type C” has not infills at the first storey. In detail, the masses corresponding to masonry infill were directly applied to the supporting beams and kept unchanged for all the structures under investigation. The infill panels were considered as made of clay bricks and two different thickness were taken into account for developing seismic analysis: t_w = 14 cm and t_w = 20 cm, respectively.

Finally, three- and four-storey frames were also designed to explore the influence of masonry infill on taller buildings virtually designed against gravitational loads only (Fig. 8). In fact, the simulated design under gravitational loads only for the three- and four-storey frames led to adopting for all columns square 30x30 cm^2 transverse sections with four 14 mm diameter longitudinal bars, which is the same section actually adopted for the two-storey building.

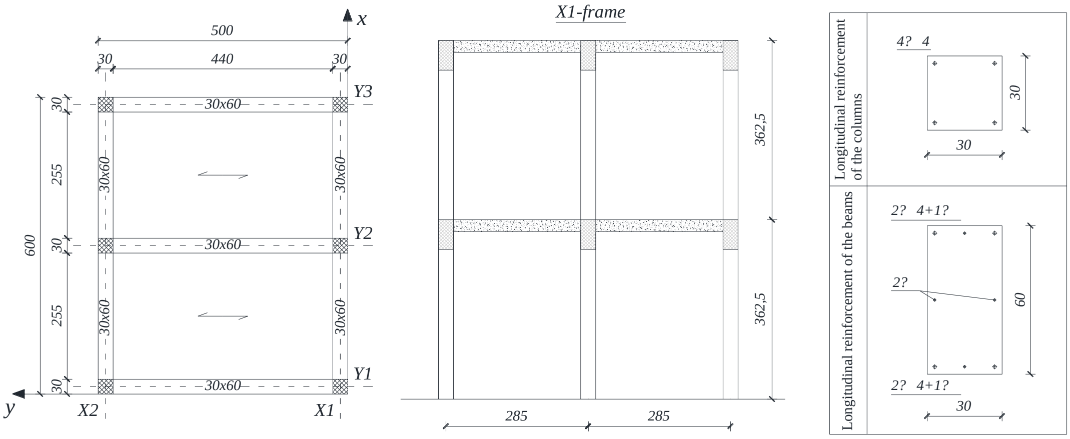


Fig. 6 The case-study: structural plan, section and longitudinal reinforcements (units in cm)

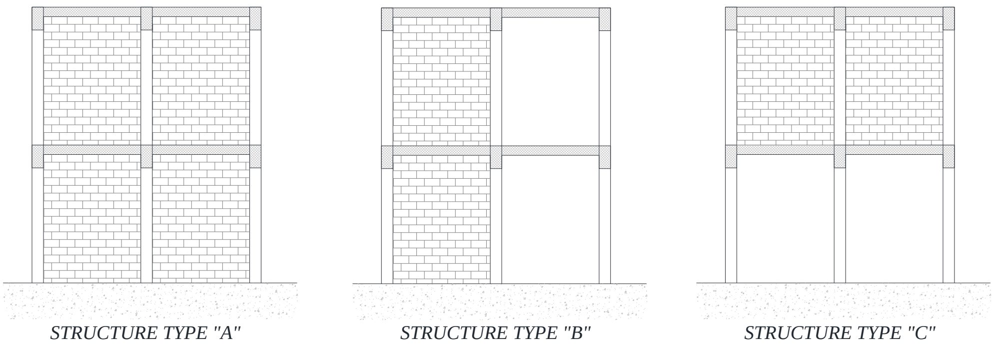


Fig. 7 Alternative distributions of masonry infill within the RC frames considered as case study

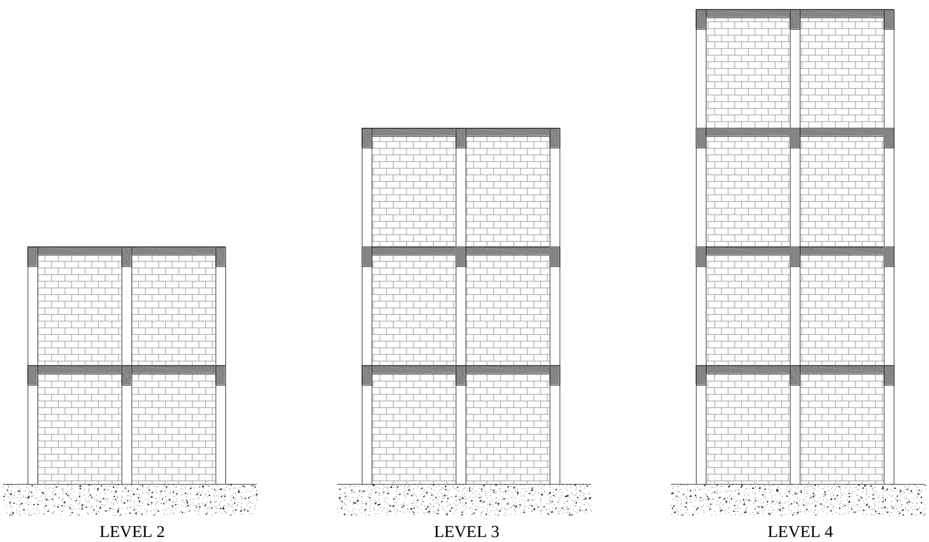


Fig. 8 Two-, three- and four-storey structures under investigation

3.1 Materials

The following mechanical properties were adopted to simulate the behaviour of structural materials in typical existing buildings realised in Europe in '50s - '60s of the past century:

- Concrete R_{cm} = 22.5 MPa (f_{cm} = 19 MPa);
- AQ50-type reinforcement steel with f_{sm} = 330 MPa (smooth bars).

Masonry infill were supposed to be made out of artificial blocks of expanded clay. They were modelled by considering the mechanical properties listed below:

- average compression strength f_k = 2,00 MPa;
- average shear strength f_{vk0} = 0,048 MPa;
- normal elastic modulus E_w = 1600 MPa;
- shear modulus G_w = 152.83 MPa.

3.2 Structural modelling

The structures were modelled in OpenSEES (Mazzoni *et al*. 2007) with the aim of performing nonlinear analyses to simulate the behaviour observed for the bare structure and to predict the response of the same structure with different arrangement of masonry infill. Particularly, a nonlinear finite element (FE) model was developed to simulate the structural response of the structure described above subjected to seismic excitations.

Actually, the nonlinear behaviour of beams and columns was simulated by employing force-based distributed plasticity elements (nonlinearBeamColumn, Mazzoni *et al*. 2007). Transverse sections of such members were subdivided in 30x30 fibres, whose number was defined via a thorough sensitivity analysis. Nonlinear stress-strain laws were considered to simulate the behaviour of concrete and reinforcing steel of longitudinal bars. Particularly, the Concrete01 model (Fig. 9(a)) was employed for modelling both cover and core concrete. Moreover, an elastic-plastic behaviour with 1% symmetric hardening (factor *b* in Table 1 represents the ratio between the stiffness of the plastic branch and the elastic one) was adopted for rebars through the so-called Steel01 (Mazzoni *et al*. 2007) stress-strain law (Fig. 9(b)). The mechanical properties adopted for structural materials are summarized in Table 1.

Truss elements were employed for simulating the behaviour of masonry infill in terms of both strength and stiffness: they were supposed to sustain only compressive (negative) axial forces. Stiffness degradation in both loading and unloading phases was implemented through the so-called "Pinching 4" model available in OpenSEES (Mazzoni *et al.* 2007) depicted in Fig. 10.

Table 1 Relevant parameters for describing the material behaviour

Material	f_{cm} [MPa]	$f_{cm,u}$ [MPa]	ε_{c0}	ε_{cu}
Concrete01	19.00	15.77	0.0020	0.0040
	f_{sm} [MPa]	E_s [GPa]	b	
Steel01	330.00	210.00	0.01	

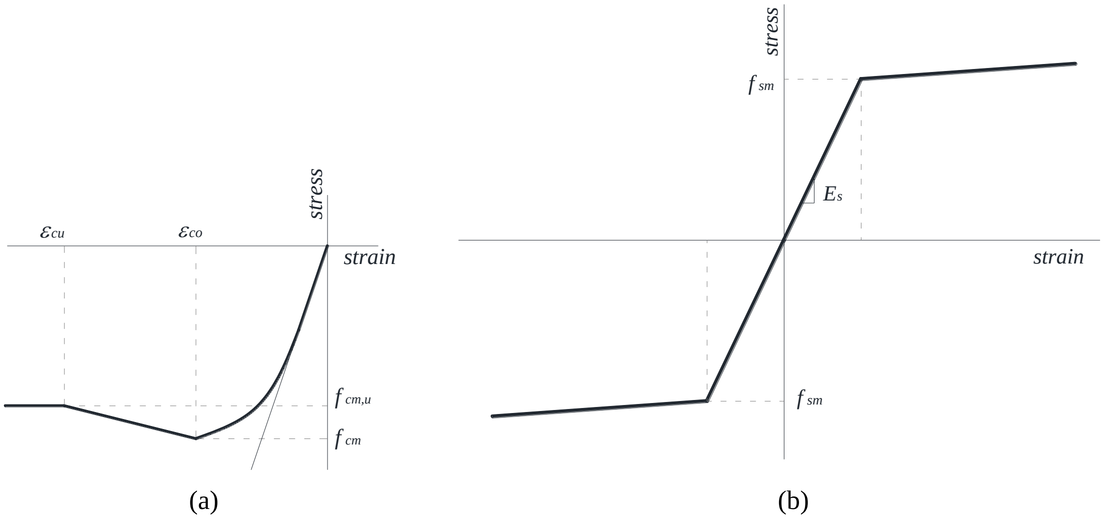


Fig. 9 Stress-strain relationships adopted for concrete (a) and steel (b)

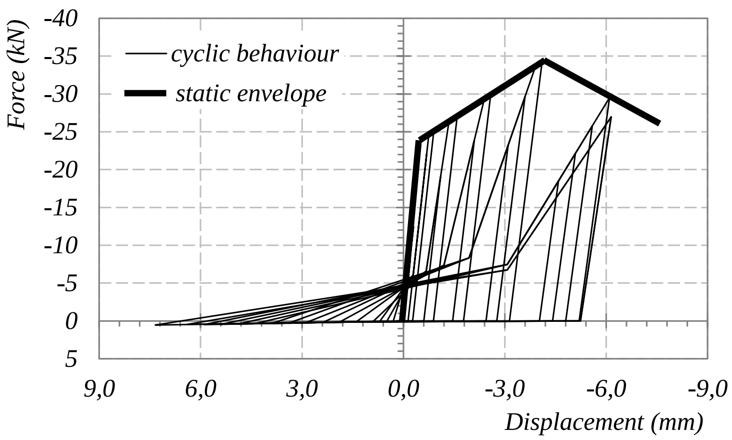


Fig. 10 Skeleton curve and hysteretic behaviour assumed for the equivalent struts

In detail, the values defining force and deformation points on the skeleton compressive response were evaluated from the geometric and mechanical properties of the masonry infills through the model by Dolsek and Fajfar (2008), while the ones on the response envelope in tension were assumed equal to zero. The following values were assumed for the parameters defining the cycling degradation (Mazzoni *et al.* 2007) through a previous calibration performed by comparing numerical analyses and experimental tests on masonry walls available in the scientific literature (Koutromanos *et al.* 2011):

- $rDispP=$rDispN=0.50 (floating point value defining the ratio between the deformation at which reloading occurs to the maximum and the minimum deformation demand);
- $fForceP=$fForceN=0.25 (floating point value defining the ratio between the force at which reloading begins to force corresponding to the maximum and the minimum

deformation demand);

- $uForceP=$uForceN=0.05 (floating point value defining the ratio between the strength developed upon unloading from negative load to the maximum and the minimum strength under monotonic loading);
- $gK1=1.0, $gK2=0.2, $gK3=0.3, $gK4=0.2, $gKLim=0.9 (floating point values controlling the unloading stiffness degradation);
- $gD1=0.5, $gD2=0.5, $gD3=2.0, $gD4=2.0, $gDLim=0.5 (floating point values controlling the reloading stiffness degradation);
- $gF1=1.0, $gF2=0.0, $gF3=1.0, $gF4=1.0, $gFLim=0.9 (floating point values controlling the strength degradation)
- $gE=10 (floating point value related to the maximum energy dissipation under cyclic loading);
- $dmgType=cycle (string which indicates the type of damage: option= "cycle" or "energy").

Fig. 11 describes the geometry of the structural models implemented for the masonry infilled structure while the model of the bare one was obtained accounting the geometry of Fig. 11 without diagonal trusses.

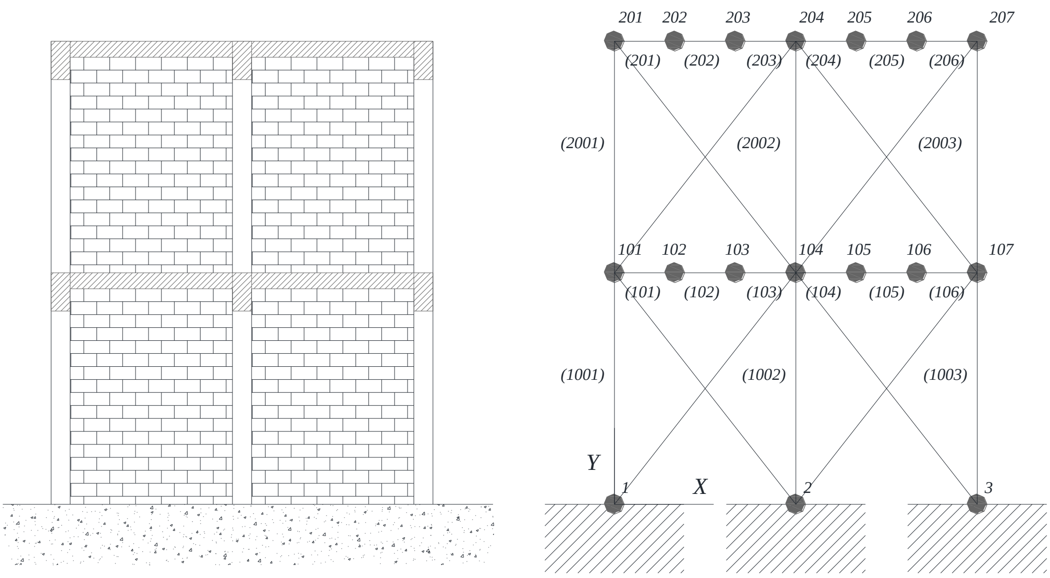


Fig. 11 Geometry of structural models for either bare and masonry infilled RC structure under consideration

4. Results of nonlinear static analyses

The first results reported in this section refer to the two-storey frame described in section 3. Particularly, the comparison between the capacity curves derived for the bare and the three infilled structures depicted in Fig. 7 will be carried out. Then, such a comparison will be extended to the expected values of displacement demand obtained through the pushover analysis and the following nonlinear static analysis methods for bare and infilled RC frames, respectively:

- the well-known N2-method (Fajfar, 1999), according to the EN 1998-1 (2005) provisions;
- the more recent procedure proposed by Dolsek and Fajfar (2005) for evaluating the performance point of infilled structures.

Further details about the mentioned method for infilled structures are herein omitted for sake of brevity and can be found in Dolsek and Fajfar (2005).

The analyses proposed in this paper are mainly intended at highlighting possible sensitivity of the structural response of the infilled frame with respect to:

- the choice of the constitutive law used in modelling masonry infill;
- the arrangement of infill within the RC frame;
- the amount of openings in the masonry panel.

4.1 Nonlinear Static Analysis (NLS)

Based on the mechanical models outlined in section 2 and the geometric and mechanical properties of the structures described in section 3, the nonlinear response of struts simulating the behaviour of masonry infill for the structures under consideration can be easily simulated. Particularly, Fig. 12 depicts the axial force-displacement curves of the equivalent strut obtained through the models by Panagiotakos and Fardis (1996) and Dolsek and Fajfar (2008) for a masonry panel with thickness t_w=14 cm (Fig. 12(a)) and t_w=20 cm (Fig. 12(b)). The two compared models, respectively described in subsections 2.1.1 and 2.1.2, lead to a slight difference in terms of post-cracking behaviour: the latter leads to slightly higher values of displacements, while the model by Panagiotakos and Fardis (1996) leads to values of the stiffness of the second branch R_2 very close to the initial stiffness R_1.

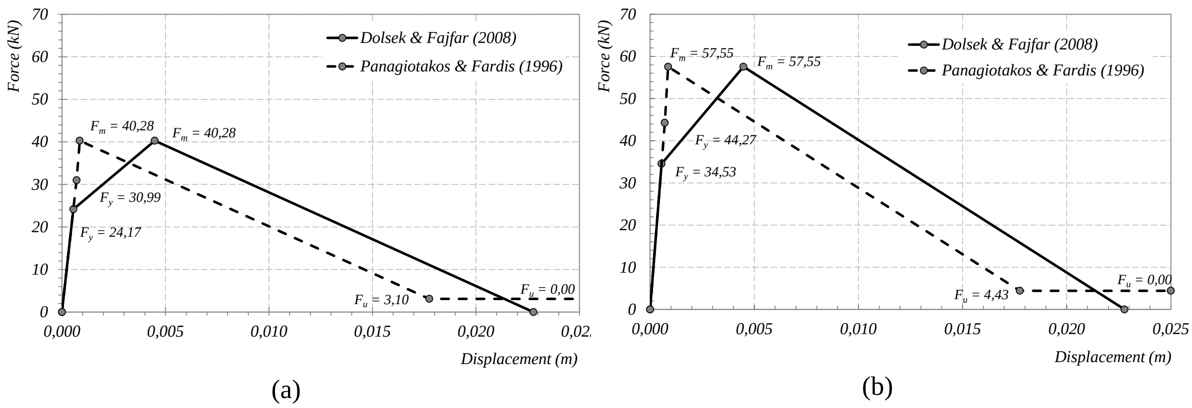
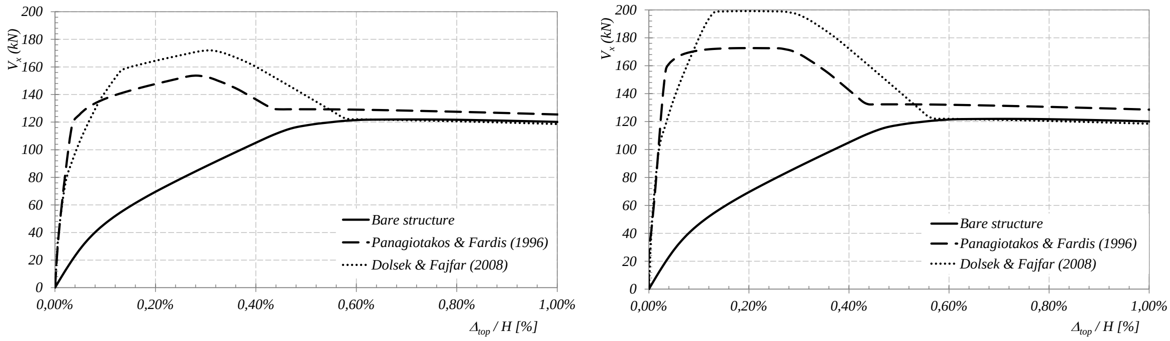


Fig. 12 Comparison between the models by Panagiotakos and Fardis (1996) and Dolsek and Fajfar (2008) in terms of force-displacement curves for the equivalent strut: wall thickness (a) t_w=14 cm and (b) t_w=20 cm

Performing Nonlinear Static (NLS) analyses, the capacity curves can be obtained in terms of top displacement (on the x-axis) and base shear (on the y-axis). Fig. 13 depicts the capacity curves derived for the two-storey bare structure and the fully infilled ones (Type A in Fig. 7) in which a first comparison between the results obtained by assuming the models by Panagiotakos and Fardis (1996) and Dolsek and Fajafar (2008) is carried out. Particularly, Fig. 13(a) and Fig. 13(b) depict



(a) (b)

Fig. 13 Capacity curves for the fully infilled RC frames according to the models by Panagiotakos and Fardis (1994) and Dolsek and Fajfar (2008): (a) wall thickness t_w=14 cm; (b) wall thickness t_w=20 cm

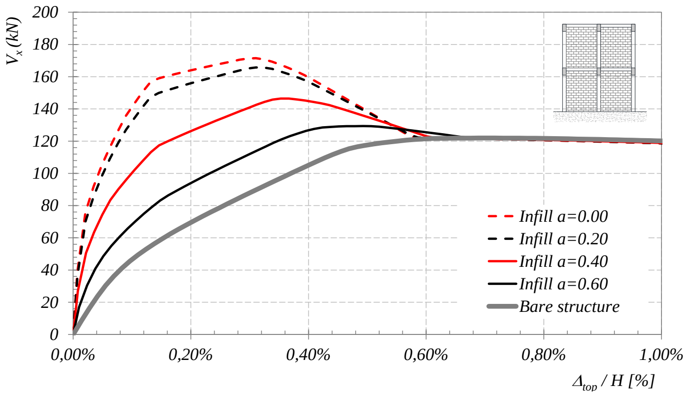


Fig. 14 Capacity curves for masonry infill with openings (Type A frame)

the capacity curves of the fully infilled structure with wall thickness t_w equal to 14 cm and 20 cm, respectively.

Since the influence of infills on the global response of structures can be significantly affected by the presence of openings (e.g., windows or doors), the model by Papia and Cavaleri (2001) was considered for simulating the response of structures with such infills. Particularly, opening ratios [see Eq. (8) in section 2] ranging between 0 and 0.60 were considered and all the three types of two-storey masonry distributions represented in Fig. 7 were considered. Openings centred in the masonry panel without any additional concrete element around they were taken into account. Fig. 14 depicts the results of the fully infilled frame (Type A) and shows the significant variation in terms of lateral stiffness and maximum strength induced by opening of increasing dimension. This effect points out the importance of masonry infills, especially under seismic actions corresponding to low intensity (and high frequency) earthquake, which are generally relevant for Serviceability Limit State (SLS) checks (EN 1998-1, 2005). A low influence of masonry openings can be observed for the two-storey Type B infilled structures (Fig. 15), as the contribution of infills to the resulting structural capacity is clearly lower than in the case of fully infilled (Type A) ones (Fig. 14). Moreover, as expected, Fig. 16 shows an almost negligible influence of openings in the case of the so-called “pilotis” frames (Type C in Fig. 7). In fact, in such a structural configuration the

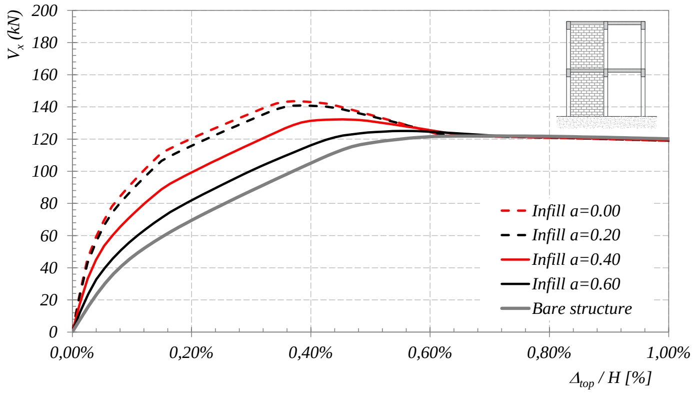


Fig. 15 Capacity curves for masonry infill with openings (Type B frame)

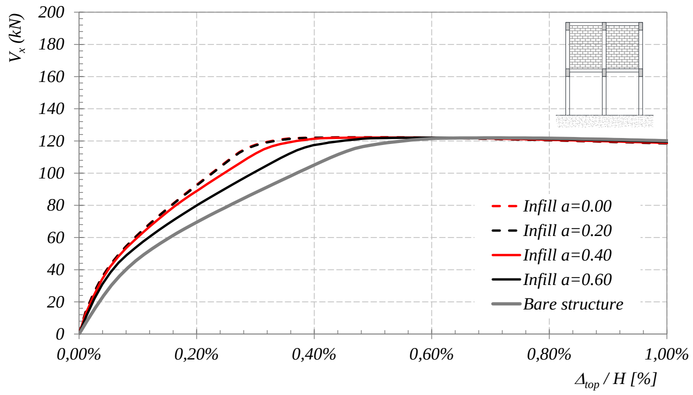


Fig. 16 Capacity curves for masonry infill with openings (Type C frame)

global response is characterised by a soft-storey mechanism at the first storey in which there are not masonry infills.

4.2 The influence of infills on the seismic response of RC frames

Further understanding of the role of infills in the seismic response of structures can be achieved by analysing the difference in terms of seismic demand evaluated on bare and infilled structures through Nonlinear Static (NLS) analyses as mentioned at the beginning of section 4. For all the following analyses the model by Dolsek and Fajfar (2008) was taken into account and the possible effect of openings was considered by modifying the width *w* of the equivalent strut according to Papia and Cavaleri (2001).

Since a different importance of masonry infill is expected, depending on the actual seismic intensity values, two Linear Elastic Design Spectra (LEDS) are considered in the following. Fig. 17 depicts such elastic spectra (with 5% damping) and emphasises their Peak Ground Acceleration values of 0.334g (Earthquake #1) and 0.261g (Earthquake #2). As a matter of principle, they could be intended as referring to two seismic events characterised by different return periods. In fact, they were derived for a site in L'Aquila (Italy), for a Soil Category A and return periods of 475

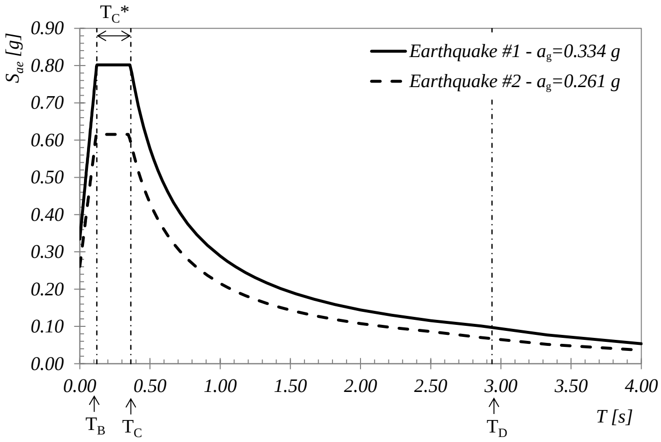


Fig. 17 Elastic design spectra of the seismic excitations considered in the present study

Table 2 Key parameters of the target elastic spectra considered in this study (EN 1998-1 2005)

	(NC) Earthquake #1	(LS) Earthquake #2
a_g [g]	0.334	0.261
F_0	2.400	2.364
T_C* [s]	0.364	0.346
T_B [s]	0.121	0.115
T_C [s]	0.364	0.346
T_D [s]	2.936	2.642

Table 3 Displacement demand for infilled and bare frames (NLSA)

	Earthquake #1 - Δ_{top} (cm)	Earthquake #2 - Δ_{top} (cm)
Bare structure	4.87	3.57
Infill Type A	2 64	1.49
Infill Type B	3.96	2.72
Infill Type C	4.74	3.27

and 945 years as required by EN 1998-1 (2005) for Life Safety (LS) and Near Collapse (NC) Limit States, respectively (Table 2).

The first comparisons can be carried out for the two-storey structures in terms of displacement demand evaluated through the well-known N2 Method (Fajfar 1999) for the bare structure and by applying the recent procedure proposed by Dolsek and Fajfar (2005) for the masonry infilled RC frames. As expected, the infilled frame results in lower top-displacement demand (Table 3 and Fig. 18). Such a difference is clearly higher in the case of fully infilled structures (Type A in Fig. 7) and for lower intensity seismic event (Earthquake #2).

A further aspect arises from comparing the results of NLS analyses in terms of interstory drifts (Fig. 19). In this case, the expected effect of masonry infills to reduce seismic demand is not generally confirmed, and an even significant increase in interstory drift is determined, as expected, for the "pilotis" pattern (Structure Type C in Fig. 7) resulting in a soft-storey mechanism at the

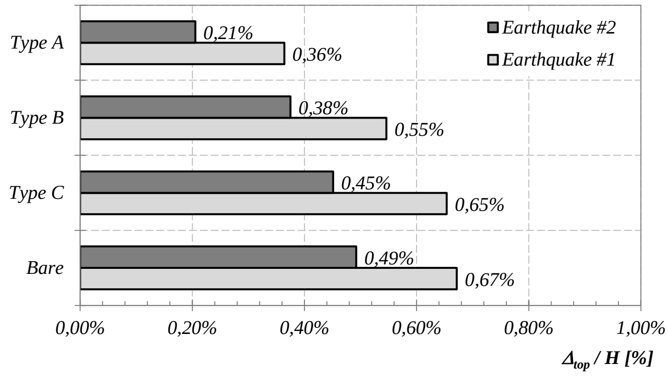
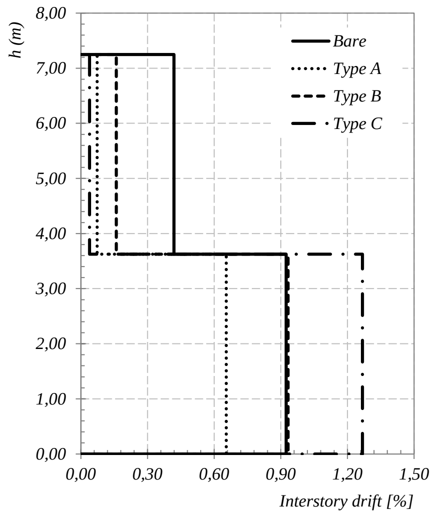
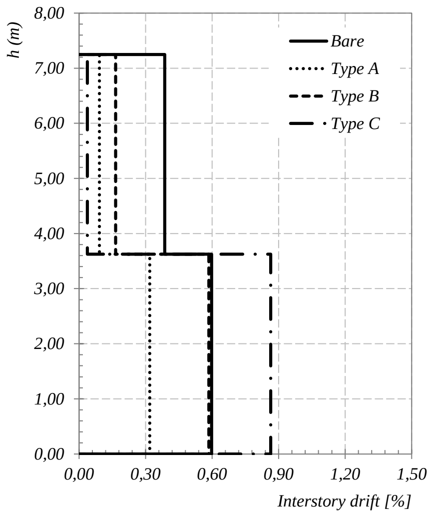


Fig. 18 Influence of masonry infills in terms of top displacements (NLS analysis)



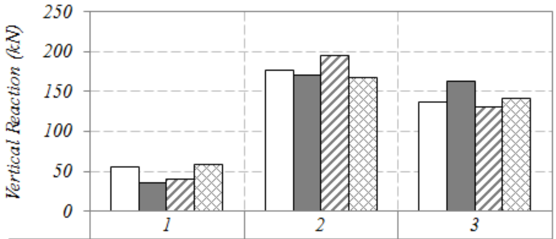
(a) Earthquake #1



(b) Earthquake #2

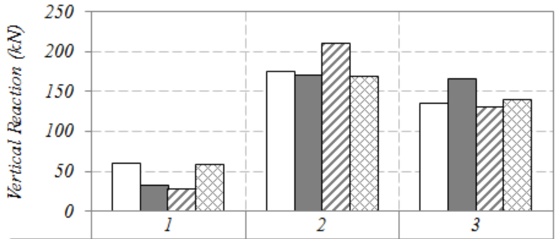
Fig. 19 Influence of masonry infills in terms of interstory drifts (NLS analysis)

first storey. Then, the results reported in Fig. 18 and Fig. 19 confirm the commonly accepted idea that masonry infill generally (but not always) reduces the seismic-induced displacement demand on structures and, then, analyses based on neglecting their contribution can be performed for reasonably conservative predictions of the seismic demand of RC structures. However, a slightly deeper examination of the results should point out that the above deduction is highly simplistic and could even be dangerously wrong. In fact, seismic demand should not only be analysed in terms of displacements, but also in terms of forces (brittle mechanisms). Under this standpoint, the aforementioned simplistic conclusions about the role of infill walls could often be reversed, as significantly higher forces can be expected on both RC members and foundations if the influence of masonry infills is duly taken into account (Fig. 20). Particularly, Fig. 20 reports the vertical reactions at the bases of the three columns at the foundation level: the number 1 indicates the left column, the number 2 represents the central one and the number 3 refers to the right one. Values of



	1	2	3
Bare	55.35	176.37	137.23
Type A	36.01	170.95	162.00
Type B	40.74	196.38	131.83
Type C	59.02	167.52	142.41

(a) Earthquake #1



	1	2	3
Bare	59.50	174.77	134.71
Type A	33.16	170.25	165.54
Type B	28.23	210.20	130.52
Type C	58.93	168.82	141.20

(b) Earthquake #2

Fig. 20 Vertical reactions (NLS analysis)

the axial forces achieved in columns of infilled structures are even significantly higher than the ones obtained in bare frames. Therefore, the presence of masonry infill can result in lower ductility capacity in such columns and, then, neglecting such columns can lead to unsafe seismic analysis of buildings.

4.3 Final comments

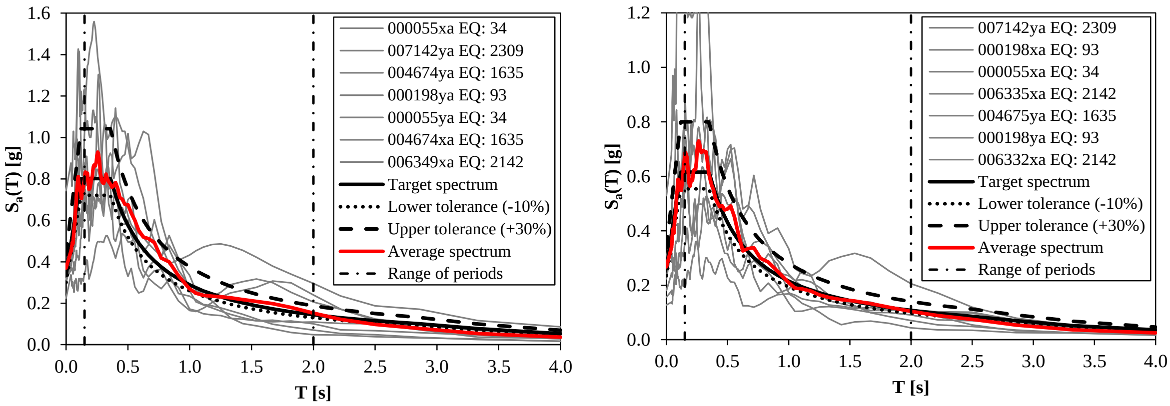
The above subsections pointed out the significant difference which can arise by applying the various models currently available in the literature for simulating the contribution of masonry infills on the seismic response of structures. The results of NLS analyses carried out to simulate such a response emphasised that neglecting such a contribution generally leads to a conservative evaluation of displacement demand, but conversely results in a significant underestimation of the actual levels of stress on the RC members.

5. Results of Nonlinear Time-History Analyses (NLTH)

Since NLS analyses carried out on the two-storey building shed a concerning light on the influence of masonry infills on the seismic demand of RC frames, more accurate Nonlinear Time-History (NLTH) analyses were also carried out to further investigate such an influence. To this end, two sets of seven natural accelerograms were selected from the European Strong Motion Database by considering the two target LEDS whose key parameters are reported in Table 2 (Iervolino *et al*. 2010). Fig. 21 depicts both target and natural spectra for the two sets of accelerograms considered in this study. Moreover, the investigation was extended to three- and four-storey frames (Fig. 8) with the same distribution of masonry infill reported in Fig. 7.

As already done for NLS analyses, masonry infill were simulated by the model by Dolsek and Fajfar (2008) with the width w of the equivalent struts evaluated according Papia and Cavaleri (2001). The behaviour of the equivalent strut under cyclic loads was simulated by the Pinching04 model available in OpenSEES (Mazzoni *et al*. 2007) shown in Fig. 10.

The results in terms of maximum absolute displacement demands obtained for the two-storey frames are firstly examined. Particularly, Table 4 reports the numerical values obtained for the two

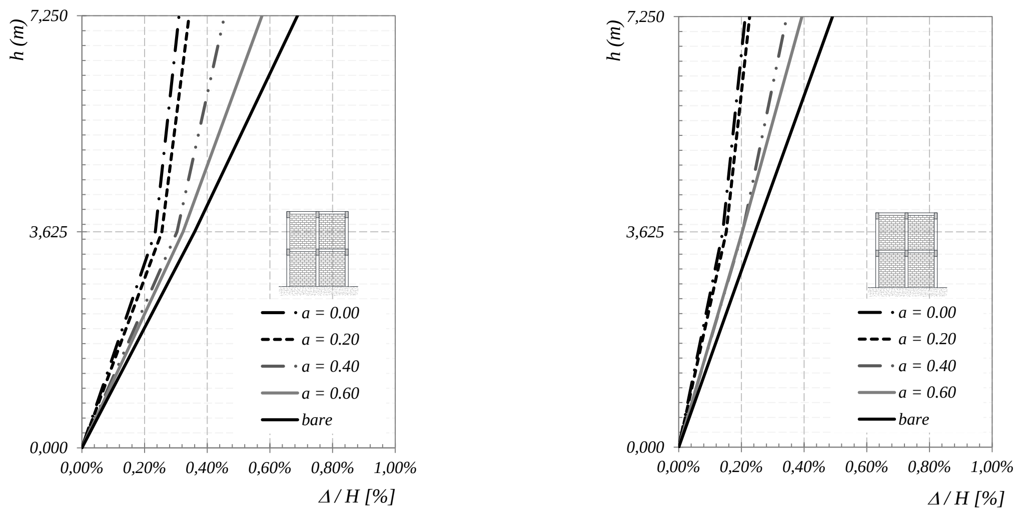


(a) Earthquake #1 (b) Earthquake #2

Fig. 21 Response spectra corresponding to the two sets of natural records considered in this study

Table 4 Displacement demand for the infilled and bare frames (NLTH)

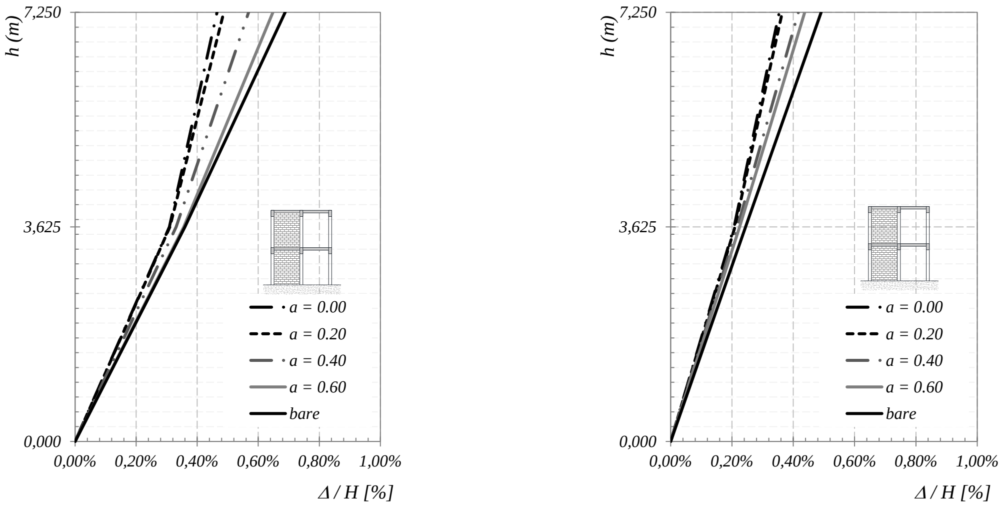
	Earthquake #1 - Δ_{top} (cm)	Earthquake #2 - Δ_{top} (cm)
Bare structure	4.99	3.56
Infill Type A	2.25	1.54
Infill Type B	3.37	2.57
Infill Type C	3.25	2.44



(a) Earthquake #1 (b) Earthquake #2

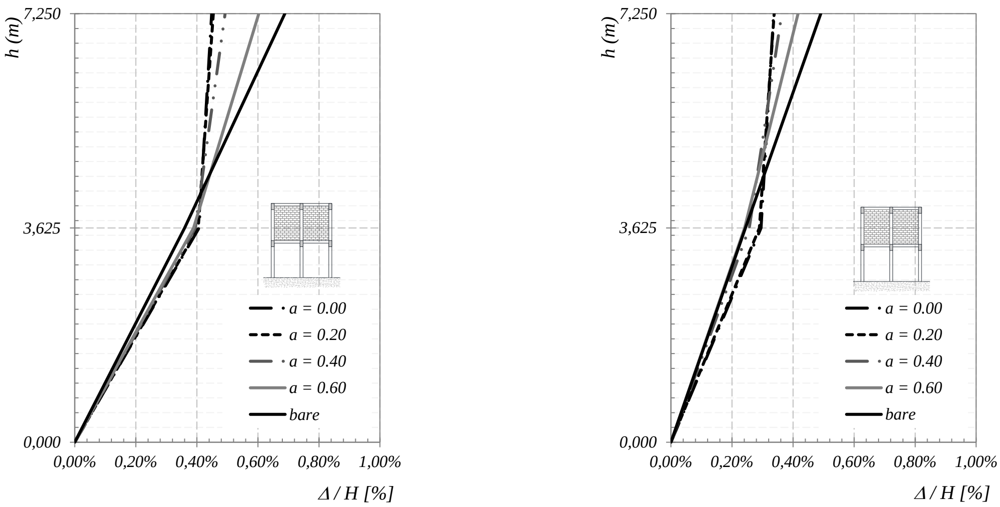
Fig. 22 Average values of interstory displacements-total height ratio (Type A frame)

seismic intensities. The comparison between the results obtained through NLS analysis (Table 3) and the corresponding ones deriving by NLTH (Table 4) shows that they are in good agreement for the bare structure, while NLS analysis leads generally to significant overestimation of seismic demand for infilled structures. The assessment of NLS analysis for infilled structures is beyond the



(a) Earthquake #1 (b) Earthquake #2

Fig. 23 Average values of interstory displacements-total height ratio (Type B frame)

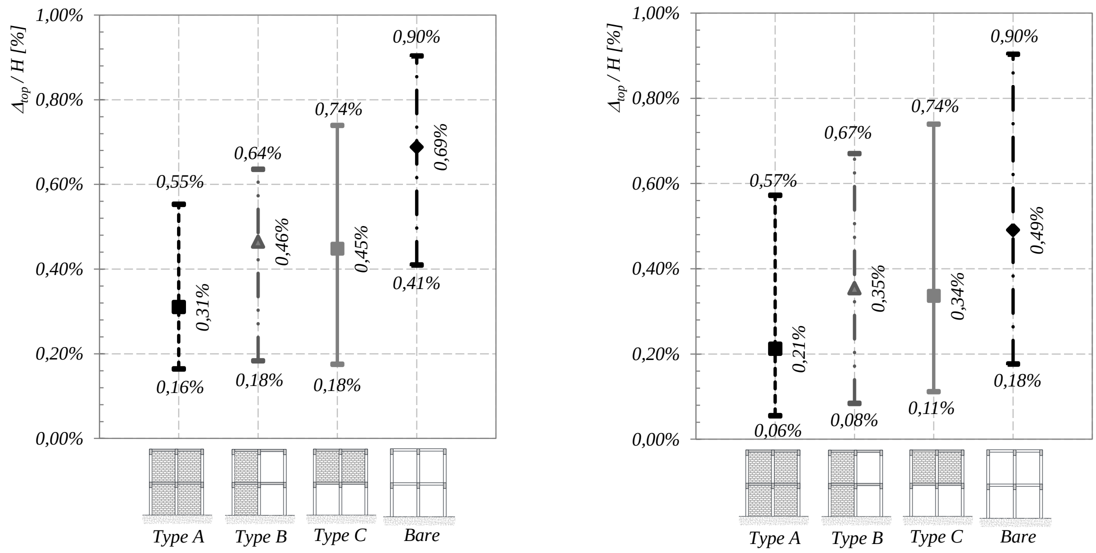


(a) Earthquake #1 (b) Earthquake #2

Fig. 24 Average values of interstory displacements-total height ratio (Type C frame)

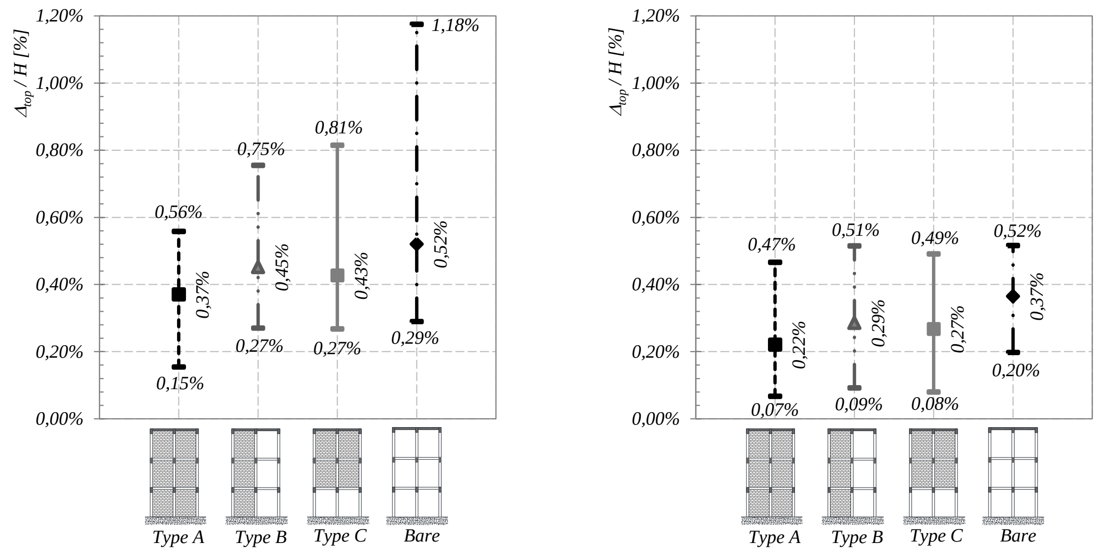
scopes of this paper (and will be specifically addressed by the authors in a future work). However, this result puts some concerns on the general validity and the actual predictive capacity of the method proposed by Dolsek and Fajfar (2005).

This paper specifically aims at emphasising the influence of masonry infill on the seismic response of structures. NLTH analyses generally confirm the trend observed through NLSA in terms of reduction of top displacement of infilled structures with respect to the corresponding values deriving on the bare RC frames. Particularly, Fig. 22 and Fig. 23 (dealing with two storey "Type A" and "Type B" frames, respectively) show a reduction in terms of both top-displacements



(a) Earthquake #1 (b) Earthquake #2

Fig. 25 Top displacements for bare and infilled structures (two-storey frame)



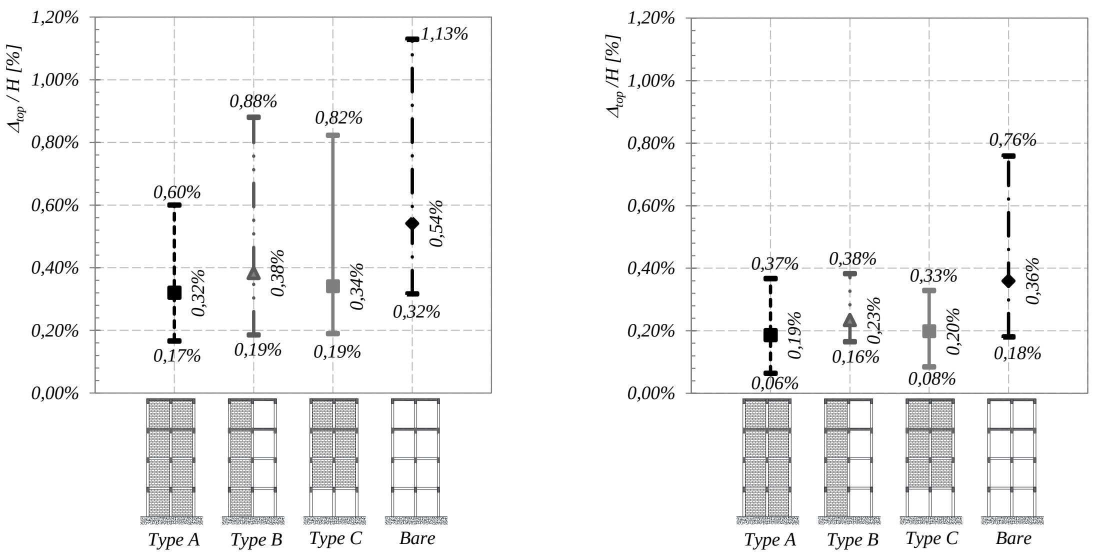
(a) Earthquake #1 (b) Earthquake #2

Fig. 26 Top displacements for bare and infilled structures (three-storey frame)

and interstory drifts for infilled structures with decreasing opening ratio (parameter “a” Eq. (8)). On the contrary, Fig. 24 shows that interstory drift obtained for the “Type C” frame at the first level can even be higher than those obtained on the bare structure, as a result of the soft-storey effect clearly due to the “pilotis” configuration.

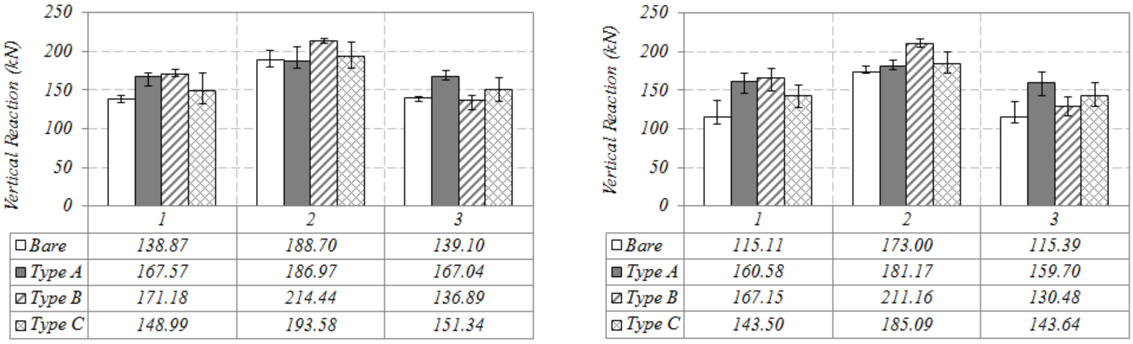
Moreover, a further concerning aspect arises from the results obtained by NLTH analyses. It deals with the record-by-record variability of the global structural response which affects the two sets of seven natural accelerograms considered to match the two target design spectra (Fig. 17). Fig. 25 reports the minimum, average and maximum values of top displacements-total height ratio evaluated through NLTH for the two storey structures. It confirms that average values of such displacements are lower for the infilled structures. Furthermore, the expected values of the Coefficient of Variation (CoV) of displacement demand would be higher for infilled structures and, then, the supposed reduction of displacement demand in terms of a deterministic criterion, just based on comparing the average values, would be significantly lower in terms of reliability index (Pinto *et al*. 2004). Similar observations arise for the three and four storey frames in Fig. 26 and Fig. 27.

Finally, the force-related aspects are analysed to check if the concerning predictions derived through NLS analyses (Fig. 20) are confirmed by the supposedly more accurate NLTH. Fig. 28 reports the value of the vertical components of the base node reactions obtained through NLTH analysis performed on two storey structures. It ideally corresponds to Fig. 20 which reports the same quantities obtained through NLS analysis. Both figures demonstrate the increase in vertical reactions deriving by considering the role of masonry infill on the seismic response of RC frames. Particularly, the more accurate NLTH analyses lead to even higher and more concerning results in terms of difference between the simulation carried out on the bare structural models and the other infilled structures. It is worth noting that the increase in terms of vertical forces is higher for the infill distributions which results in the higher reduction in terms of displacement demand. This observation further confirms the importance of taking into account masonry infill to obtain a safe and accurate estimate of the action transferred by the frame to the foundation during the earthquake shaking.



(a) Earthquake #1 (b) Earthquake #2

Fig. 27 Top displacements for bare and infilled structures (four-storey frame)



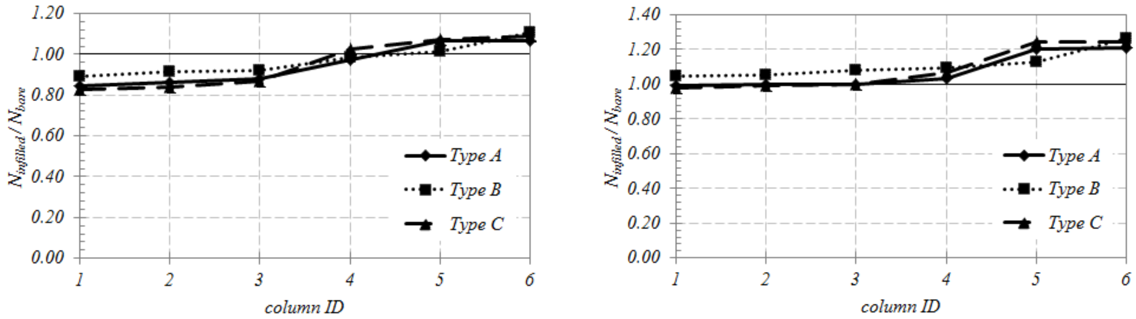
	1	2	3
Bare	138.87	188.70	139.10
Type A	167.57	186.97	167.04
Type B	171.18	214.44	136.89
Type C	148.99	193.58	151.34

(a) Earthquake #1

	1	2	3
Bare	115.11	173.00	115.39
Type A	160.58	181.17	159.70
Type B	167.15	211.16	130.48
Type C	143.50	185.09	143.64

(b) Earthquake #2

Fig. 28 Vertical reactions (NLTH; two-storey frame)



(a) Earthquake #1 (b) Earthquake #2

Fig. 29 Distribution of the axial force ratio in columns (NLTH; two-storey frame)

Finally, some results dealing with the maximum values of the seismic-induced axial forces determined in the six columns of the structures under consideration are reported in Fig. 29 in terms of axial force ratio. Particularly, it reports the distribution of the ratio between axial forces obtained for the infilled frame and the corresponding ones registered on the bare structures. Fig. 29 demonstrates that masonry infill and their contribution to lateral stiffness result in a significant increase of axial forces with respect to the corresponding values possibly determined on the bare structure. This result is of key importance if one considers the (negative) influence of high compressive forces on the nonlinear behaviour (and the displacement capacity) of RC columns.

6. Conclusions

This paper proposed a contribution to further understanding some critical issues of the seismic response of RC frames, generally emerging when the in-plane contribution of masonry infill is explicitly considered. Particularly, this study is the first part of a thorough research on the above mentioned subject and aims to demonstrate that, in spite of usual assumptions, neglecting the role of masonry infill does not generally lead to conservative simulations of the seismic response of RC frames. The following key observations emerge as final remarks to the proposed discussion:

- nonlinear static analyses pointed out that the commonly accepted assumption of reduction in displacement demand of infilled structures with respect to the corresponding bare ones is generally true in terms of top displacement, but strongly depends on the actual infill distribution in terms of interstory displacements;

- the results of nonlinear static analyses in terms of force distribution shed a new concerning light on the importance of considering the infill contribution, as it can result in a significant increase of the global eccentricity of the forces transferred by the structure to the foundation elements;

- the results of time-history analyses confirmed the above mentioned aspects and added further elements of concern, as they emphasised a significant increase in the record-by-record induced variability of the seismic response of structures: this aspect was particularly investigated for structures of different numbers of stores and this emphasized the increased role of masonry infill on globally slender structures (i.e. in terms of height-to-width ratio);

- moreover, NLTH analyses put in even more evidence the role of masonry infill in modifying (generally increasing) the maximum levels of axial forces in RC columns: this observation figured out a further negative effect of the infill contribution on the displacement capacity of RC members (which is significantly affected by the values of axial forces actually developing therein during the seismic shaking);

- finally, the comparison of displacement demand predictions evidenced that the currently adopted methods for nonlinear analyses of RC frames are generally accurate for bare and fully infilled ones, while they are not generally accurate for general distributions of infills.

The above comments can be considered as a motivation for a further in-depth examination of the seismic response of masonry infilled RC frames. Particularly, future developments of this research will be addressing three relevant aspects like i) assessing the currently available methods for nonlinear static analysis of infilled frames, ii) modelling the effect of out-of-plane loads on the in-plane behaviour of masonry infilled RC frames and iii) possibly improving of such methods in view of an enhanced predictive capacity for the cases of generally distributed masonry infill.

References

Ali, S.S. and Page, A.W. (1998), "Finite element model for masonry subjected to concentrated loads", *J. Struct. Eng.*, **114**(8), 1761-1784.

American Society of Civil Engineers (ASCE) (2006), *Seismic rehabilitation of existing buildings (ASCE/SEI 41)*, Reston, Virginia.

Asteris, P.G. (2003), "Lateral stiffness of brick masonry infilled plane frames", *J. Struct. Eng.*, **129**(8), 1071-1079.

Biondi, S., Colangelo, F. and Nuti, C. (2000), "La risposta sismica di telai con tamponature murarie", *CNR - Gruppo Nazionale per la Difesa dai Terremoti*, Roma.

CEB Task Group III/6 (1996), *RC frames under earthquake loading: state of the art report*, Bulletin 231, Thomas Telford Publishing.

Di Sarno, L., Acanfora, M., Manfredi, G. and Pecora, R. (2008), "Vibration control of structures under environmental loading", *Proceedings of the 14th World Conference on Earthquake Engineering*, Beijing, China.

Dolsek, M. and Fajfar, P. (2004), "Inelastic spectra for infilled reinforced concrete frame", *Earthq. Eng. Struct. Dyn.*, **33**, 1395-1416.

Dolsek, M. and Fajfar, P. (2005), "Simplified non-linear seismic analysis of infilled reinforced concrete

frames", *Earthq. Eng. Struct. Dyn.*, **34**, 49-66.
Dolsek, M. and Fajfar, P. (2008), "The effect of masonry infills on the seismic response of a four storey reinforced concrete frame - a deterministic assessment", *Eng. Struct.*, **30**(11), 1991-2001.
Durrani, A.J. and Luo, Y.H. (1994), "Seismic retrofit of flat-slab buildings with masonry infills. Report NCEER-94-0004", *Proc. from the NCEER Workshop on Seismic Response of Masonry Infills*, 1-8.
EN 1998-1 (2005), Eurocode 8: Design of structures for earthquake resistance. Part 1: General rules, seismic actions and rules for buildings.
EN 1998-3 (2005), Eurocode 8: Design of structures for earthquake resistance. Part 3: Assessment and retrofitting of buildings.
Fajfar, P. (1999), "Capacity spectrum method based on inelastic demand spectra", *Earthq. Eng. Struct. Dyn.*, **28**, 979-993.
Fajfar, P. (2000), "A nonlinear analysis method for performance based seismic design", *Earthq. Spectra*, **16**(3), 573-592.
Fardis, M.N. (1997), "Experimental and numerical investigation on the seismic response of RC infilled frames and recommendations for code provisions", Laboratorio Nacional de Engenharia Civil, Lisboa, Report 6 of ECOEST-PREC8 Project.
Fiore, A., Netti, A. and Monaco, P. (2012), "The influence of masonry infill on the seismic behaviour of RC frame buildings", *Eng. Struct.*, **44**, 133-145.
Hao, H., Ma, G. and Lu, Y. (2002), "Damage assessment of masonry infilled RC frames subject to blasting induced ground excitations", *Eng. Struct.*, **22**, 799-809.
Holmes, M. (1961), "Steel frames with brickwork and concrete infilling", *ICE Proc.*, **19**(4), 473-478.
Iervolino, I., Galasso, C. and Cosenza, E. (2010), "REXEL: computer aided selection for code-based seismic structural analysis", *Bull. Earthq. Eng.*, **8**, 339-362.
Kakaletsis, D.J. and Karayannis, C.G. (2008), "Influence of masonry strength and openings on infilled R/C frames under cyclic loading", *J. Earthq. Eng.*, **12**(2), 197-221.
Klingner, R.E. and Bertero, V.V. (1978), "Earthquake resistance of infilled frames", *J. Struct. Div.*, **104**(ST6), 973-989.
Koutromanos, I., Stavridis, A., Benson Shing, P. and Willam, K. (2011), "Numerical modelling of masonry-infilled RC frames subjected to seismic loads", *Comput. Struct.*, **89**, 1026-1037.
Kuang, J.S. and Yuen, Y.P. (2010), "Effect of out-of-plane loading on in-plane behaviour of unreinforced infilled RC frames", *Proceedings of the International Conference on Computing in Civil and Building Engoneering*, Nottingham University Press, 1-6.
Liauw, T.C. and Kwan, K.H. (1984), "Nonlinear behaviour of non-integral infilled frames", *Comp. Struct.*, **18**, 551-560.
Lima, C., Martinelli, E. and Faella, C. (2012), "Capacity model for shear strength of exterior joints in RC frames: experimental assessment and recalibration", *Bull. Earthq. Eng.*, **10**(3), 985-1007.
Lima, C., Martinelli, E. and Faella, C. (2012), "Capacity model for shear strength of exterior joints in RC frames: state-of-the-art and synoptic examination", *Bull. Earthq. Eng.*, **10**(3), 967-983.
Mainstone, R.J. (1971), "On the stiffnesses and strengths of infilled frames", *ICE Proc. Suppl.*, **4**, 57-90.
Mazzoni, S., McKenna, F., Scott, M.H., Fenves, G.L. *et al*. (2007), "OpenSEES Open System for Earthquake Simulation, Command Language Manual", s.l.:Pacific Earthquake Engineering Research Center.
Ministerial Decree (2008), "New Italian Code for Construction", Italian Ministry of Public Work, Ordinary Supplement n.30 to the Italian Official Journal of 04 February. (in Italian)
Page, A.W. (1978), "Finite element model for masonry", *J. Struct. Div. ASCE*, **104**(8), 1267-1285.
Panagiotakos, T.B. and Fardis, M.N. (1994), "Proposed nonlinear strut model for infill panels", 1st Year Progress Report of HCM-PREC8 Project, University of Patras.
Panagiotakos, T.B. and Fardis, M.N. (1996), "Seismic response of infilled RC frames structures", *11th World Conference on Earthquake Engineering*, Paper 225.
Papia, M. and Cavaleri, L. (2001), "Effetto irrigidente dei tamponamenti nei telai in c.a.", Atti della 2° conferenza plenaria "La sicurezza delle strutture in calcestruzzo armato sotto azioni sismiche con

riferimento ai criteri progettuali di resistenza al collasso e di limitazione del danno dell'Eurocodice 8", Firenze, Ed. Politecnico di Milano, 85-94.
Paulay, T. and Priestley, M.J.N. (1992), *Seismic design of reinforced concrete and masonry buildings*, John Wiley & Sons, New York.
Penelis, G.G. and Kappos, A.J. (1997), *Earthquake-resistant concrete structures*, London, E & FN Spon.
Pinto, P.E., Giannini, R. and Franchin, P. (2004), *Seismic reliability analysis of structures*, Pavia, IUSS Press.
Ricci, P., De Luca, F. and Verderame, G.M. (2011), "6th April 2009 L'Aquila earthquake, Italy: reinforced concrete building performance", *Bull. Earthq. Eng.*, **9**, 285-305.
Smith, B.S. (1966), "Behaviour of square infilled frames", *J. Struct. Div.*, **92**(1), 381-403.
Uva, G., Raffaele, D., Porco, F. and Fiore, A. (2012), "On the role of equivalent strut models in the seismic assessment of infilled RC buildings", *Eng. Struct.*, **42**, 83-94.
Verderame, G.M., De Luca, F., Ricci, P. and Manfredi, G. (2011), "Preliminary analysis of soft-storey mechanism after the 2009 L'Aquila earthquake", *Earthq. Eng. Struct. Dyn.*, **40**, 925-944.
Yuen, Y.P. and Kuang, J.S. (2012), "Nonlinear response and failure mechanism of infilled RC frame structures under biaxial seismic excitation", *Proceedings of the 15th World Conference on Earthquake Engineering*, Lisboa, Portugal.
Zarnic, R. and Gostic, S. (1997), "Masonry infilled frames as an effective structural subassemblage", *Seismic design methodologies for the next generation of codes*, Balkema (Rotterdam), Fajfar & Krawinkler editors, 335-346.

JL

Notations

a:	aspect ratio between opening and masonry wall
a_g:	acceleration
A_b:	sectional area of the beam
A_c:	sectional area of the column
b:	hardening factor of steel
d:	length of the equivalent strut
δ_m:	maximum horizontal displacement
Δ_{top}:	top displacement
E_c:	elastic modulus of the concrete column
E_f:	elastic modulus of the generic concrete frame
E_s:	elastic modulus of steel
E_w:	elastic modulus of the masonry
ε_{c0}:	strain of concrete at the maximum strength
ε_{cu}:	ultimate strain of concrete
f_{cm}:	average cylindrical compressive strength of concrete
$f_{cm,u}$:	ultimate cylindrical compressive strength of concrete
f_k:	average compressive strength of masonry
f_{sm}:	average tensile strength at yielding of steel
f_{vk0}:	average shear strength of masonry

f_{ws}	shear strength of the equivalent strut
F_m:	lateral strength of the equivalent strut
F_0:	dynamic amplification
F_u:	ultimate force of the equivalent strut
F_y:	force at the onset of cracking of the equivalent strut
G_w:	shear modulus of masonry
H:	total height of the building
h':	theoretical distance between the beams axes
h_v:	opening height
h_w:	height of the masonry wall
I_c:	inertia of the column
λ:	relative stiffness between the masonry wall and the column
λ_c:	relative stiffness between the masonry wall and RC members in the model by Papia & Cavalleri (2001)
λ_0:	coefficient for accounting openings in the model by Dolsek & Fajfar (2008)
l':	theoretical distance between the column axes
l_v:	opening length
L_0:	horizontal projection of openings in the wall under consideration
l_w:	length of the masonry wall
r:	reduction factor of the strut width
R_1:	initial stiffness of the equivalent strut
R_2:	stiffness of the equivalent strut after the cracking
R_3:	stiffness of the equivalent strut after the maximum force
R_{cm}:	average cubic compressive strength of concrete
t_w:	thickness of the infill
T_C*:	period at constant velocity
T_B, T_C, T_D:	key periods of the Linear Elastic Design Spectrum
θ:	inclination of the equivalent diagonal strut
w:	width of the equivalent strut
w_v:	width of equivalent strut representing the open masonry wall
ν:	Poisson ratio
z:	contact length between the column and the wall