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Combining in-plane and out-of-plane behaviour of masonry infills in the seismic analysis of RC buildings

V. Manfredi* and A. Masi

School of Engineering, University of Basilicata, viale dell'Ateneo Lucano, 85100 Potenza, Italy

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Abstract. Current seismic codes (e.g. the NTC08 Italian code and the EC8 European code) adopt a performance-based approach for both the design of new buildings and the assessment of existing ones. Different limit states are considered by verifying structural members as well as non structural elements and facilities which have generally been neglected in practice. The key role of non structural elements on building performance has been shown by recent earthquakes (e.g. L'Aquila 2009) where, due to the extensive damage suffered by infills, partitions and ceilings, a lot of private and public buildings became unusable with consequent significant socio-economic effects. Furthermore, the collapse of infill panels, particularly in the case of out-of-plane failure, represented a serious source of risk to life safety. This paper puts forward an infill model capable of accounting for the effects arising from prior in-plane damage on the out-of-plane capacity of infill panels. It permits an assessment of the seismic performance of existing RC buildings with reference to both structural and non structural elements, as well as of their mutual interaction. The model is applied to a building type with RC framed structure designed only to vertical loads and representative of typical Italian buildings. The influence of infill on building performance and the role of the out-of-plane response on structural response are also discussed.

Keywords: RC buildings; seismic assessment; masonry infills; out-of-plane behaviour; in-plane damage; seismic code

1. Introduction

Seismic assessment of Reinforced Concrete (RC) existing buildings has a key role in risk mitigation management, especially in Italy, where a large part of the building stock, both private and public, was designed only to gravity loads or by considering non ductile seismic criteria.

Modern seismic codes, e.g. the NTC08 Italian code (Minister of Infrastructures 2008) and the EC8 European code (CEN 2004), adopt a performance-based approach for both the design of new buildings and the assessment of existing ones, considering various limit states. Specifically, NTC08 defines four limit states related to increasing levels of damage to structural and non structural elements, as well as to facilities. Therefore, all the main components of the building system should be considered to adequately assess seismic performance. However, in actual practice, non structural elements are generally neglected, although they represent a remarkable share of the whole building cost. Moreover, the prominent role of non structural elements in the

*Corresponding author, Ph.D, E-mail: enzo.manfredi@alice.it

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seismic performance of RC buildings has been highlighted by recent earthquakes. In the 2009 Abruzzo earthquake many buildings, both private and public, suffered heavy damage to infills, partitions and ceilings, with an elevated socio-economic impact.

In Italy as in all the world, infill panels are used as enclosure elements of RC framed buildings to ensure an adequate thermal and sound insulation. As a consequence of their modest bearing capacity, which also does not offer any guarantee over time, they are usually considered non structural elements and, thus, neglected in the seismic modelling of building structures.

Nevertheless, since infills are generally executed without either adequate gaps or effective connections with respect to the confining structural members, their specific contribution as well as their interaction with structural elements can influence both local and global seismic performance. An example is given in the 2009 Abruzzo earthquake, where, due to the diagonal reaction of infill panels under in-plane horizontal forces, beam-column joints were subjected to increased local stress that determined brittle failure (Mucciarelli *et al*. 2011). Furthermore, due to the remarkable in-plane stiffness of infill panels, their irregular arrangement in plan or in elevation resulted in the formation of unforeseen unfavourable torsional response or soft storey mechanism (Verderame *et al*. 2011).

In turn, interactions with the elements assumed in design to resist also influence infill performance. Due to their lower deformation capacities with respect to the surrounding structural members, infill panels in framed structures suffer earlier damage under low inter-storey drift values, thus determining loss of functionality up to unusability of the affected buildings. Moreover, in-plane damage can reduce the out-of-plane capacity of infill panels leading to their collapse causing severe risk for life safety.

The role of infill panels on the seismic response of RC framed buildings has been widely analysed in the technical literature. Beyond identifying their role on the dynamic characteristics of RC frame buildings (e.g. Masi and Vona 2010), studies show that where infill panels are regularly arranged both in plan and in elevation and effectively connected to the surrounding structural members, the lateral load resistance of structures increases (Fardis *et al*. 2000; Dolšek and Fajfar, 2001; Masi and Vona, 2012). Furthermore, the premature cracking of infill panels contributes significantly to the dissipation of energy due to ground shaking, thus enhancing the seismic performance (Negro and Verzelletti 1996; Fardis and Panagiotakos 1997). In some cases, the possible collapse of weak RC structures as well as structures designed only to vertical loads was prevented by the presence of strong infills (Panagiotakos and Fardis 1996). On the contrary, many studies have highlighted the poor performance of framed structures with irregular arrangement of infill panels, both in plan and in elevation (Fardis and Panagiotakos 1997). Through an extensive parametric analysis on RC Italian buildings designed only to vertical loads, Masi (2003) identified the poor seismic performance of pilotis building types due to the soft storey mechanism already activated for low seismic intensities. Similar results have been obtained by Repapis *et al*. (2006) on some existing RC building types representative of the Greek building stock.

In the research reported above, seismic analyses have generally focused on the evaluation of the influence of infill panels on structural performance and, consequently, only the in-plane seismic response of infills has been accounted for. Only a few studies have been carried out so far to assess the influence of structures on infill performance considering both in-plane and out-of-plane response. Among these, Fardis (2000) carried out analyses on twelve 3D models of RC infilled frame structures subjected to bidirectional seismic input where a single degree of freedom (SDOF) elastic-brittle model for the out-of-plane response of infills was adopted. Results show that in-plane deformation and damage to infills increase from the top to the bottom of the building, while the contrary happens with respect to the out-of-plane effects, which increase from bottom to upper storeys.

On the basis of the results of an extensive campaign of experimental tests on one bay-one storey infilled frame subjected to both in- and out-of-plane horizontal forces, Calvi and Bolognini (2001) analysed the seismic performance of infills in multi-storey buildings by means of pushover simulations. Out-of-plane performance were evaluated by comparing the seismic demand (in terms of the floor acceleration at each storey) with respect to the capacity of infills (determined from experimental tests as a function of in-plane damage). Results show that in-plane damage causes remarkable reductions of the out-of-plane capacity. Furthermore, severe damage to infills as well as out-of-plane expulsion, precedes any significant damage to the frame members. However, the possible influence on structural performance due to the premature out-of-plane collapse of infills was not evaluated in the study.

As a result of the recognized role of non structural elements on the seismic performance of RC buildings, EC8-1 (CEN 2004) and NTC08 (Minister of Infrastructures 2008) provide specific criteria to take into account the irregularities due to masonry infills considering both in plan and in elevation arrangement, as well as some provisions for damage limitation in the design of new buildings. No additional provisions are specifically provided for the assessment of existing buildings. With regard to the out-of-plane safety verification, both EC8-1 and NTC08 give directions for the calculation of seismic demand, whereas no capacity model is provided. Therefore, on one hand the Italian and European codes need to be improved by defining a suitable capacity model. On the other hand, as discussed by Sullivan *et al*. (2013), the code expressions can provide inaccurate seismic demand values. In fact, on the basis of numerical simulations carried out on two RC wall case study structures, they show that the EC8 approach (as well as other international codes) underestimates the peak acceleration demands at the roof level. Furthermore, the EC8 approach does not consider both the damping value of non-structural elements and the possible amplification of demands due to higher modes, whose role can be significant in multi-storey structures. In the light of these results, a simple approach for the prediction of acceleration spectra for the design of secondary structural and non-structural elements on single degree of freedom (SDOF) supporting structures has been proposed and validated by means of non-linear time-history analyses in Sullivan *et al*. (2013). Further studies are currently in progress to extend the proposed methodology to multi degree of freedom (MDOF) supporting structures.

The present paper, starting from an overview of the performance of non structural elements during recent earthquakes, makes a first step towards carrying out an integrated seismic assessment of RC buildings as a whole, i.e., referred to both structural and non structural elements and their mutual interaction. To this purpose, a model able to take into account the effects due to prior in-plane damage on the out-of-plane capacity of infills is proposed. In this way, the seismic performance of infill panels considering both in- and out-of-plane seismic actions and the effects on the global structural response due to the outward expulsion of infills can be evaluated. The proposed procedure is applied for the evaluation and the comparison the seismic performance of some Italian building types with RC framed resisting structure designed only to vertical loads.

2. Non structural infill damage in past earthquakes

Damage to non structural elements can be caused by either accelerations or deformations experienced during an earthquake and, consequently, it can be defined as either acceleration- or deformation-sensitive. Specifically, infills in RC frames are deformation-sensitive (or drift-sensitive) since, as a consequence of deformation capacities lower than those of the surrounding resisting frame, they suffer in-plane damage due to increasing inter-storey drift values. At the same time, the seismic forces acting in an orthogonal direction on infills panels can determine out-of-plane failure (acceleration-sensitive).

Damage to infill panels caused by in-plane and/or out-of-plane seismic actions has been frequently observed after past earthquakes.

In 1999 a M_w = 7.4 earthquake with peak ground acceleration (a_g) ranging from 0.21g to 0.41g hit Northern Turkey. Zuccaro *et al*. (2002) carried out field surveys after the event, pointing out that infill walls were made up of a single line of terracotta bricks, 15 cm thick, badly connected to the RC frame and usually supported by an external cantilever beam. Shear cracks (i.e. in-plane damage) were rarely observed in the infill panels, while they frequently suffered out-of-plane collapse.

L'Aquila 2009 earthquake (M_w = 6.3) had a_g values between 0.33g and 0.66g (Masi *et al*. 2011a). Infill panels, generally made up of two wythes, both of hollow brick masonry, having a total thickness of around 30 cm, suffered widespread damage. Some typical in-plane damage patterns are shown in Fig. 1:

a) cracking/failure due to separation from the structural frame (Fig. 1a);
b) cracking/failure due to horizontal bed joint sliding (Fig. 1b);
c) cracking/failure due to tension normal to the diagonals of the panel (Fig. 1c);
d) cracking/failure due to crushing of panel corners (Fig. 1d).

Fig. 1(a) Damage mechanism due to separation of the infill panel from the structural frame

Fig. 1(b) Damage mechanism due to diagonal (stairstep) cracking along with horizontal bed joint sliding

Fig. 1(c) Damage mechanism due to due to tension stress across the panel diagonals

Fig. 1(d) Damage mechanism due to crushing of panel corners

Fig. 1 Damage mechanism

Fig. 2(a) Incipient out-of-plane collapse of the external panel of a masonry infill

Fig. 2(b) Ejection of the external panel and incipient out-of-plane collapse of the internal panel of a masonry infill

Fig. 3(a) External collapse of infill panels in RC building

Fig. 3(b) Collapse curtain of the Mirandola hospital service centre building

As a consequence of their weak connection with the surrounding frame and/or of their large slenderness, a great deal of masonry infills failed with out-of-plane mechanisms. Examples of incipient or occurred collapse due to the overturning of the external and internal layer, are shown in Fig. 2. In some cases, the external layer was placed on a slab jutting out from the beam floor, without both lateral constraint and connections to the internal one. Such an arrangement determined poor connection to the surrounding structural frame and, consequently, exacerbated vulnerability to out-of-plane actions. In the damaged buildings, generally having RC frame structure up to 4-6 storeys, the higher inter-storey drift values are expected at the lower storeys. In fact, in-plane damage affected the panels placed at the first and second storey of buildings. Furthermore, prior in-plane damage also decreased the out-of-plane capacity and, consequently, expulsion due to overturning collapses were observed at the bottom storeys.

In 2011 a M_w = 5.1 earthquake severely struck the town of Lorca (90,000 inhabitants) in Spain. The maximum a_g value recorded in the area was around 0.37g, causing structural and non-structural damage in several RC buildings. Most of casualties were caused by masonry infills and parapets falling down from the floors of RC buildings. Generally, infills were cavity wall types made up of two layers of hollow clay blocks and placed in external position with respect to the surrounding frame or poorly connected to it. As a consequence, infills mainly collapsed by out-of-plane failure mechanisms (Fig. 3a).

More recently, damage to infills was observed after the 2012 Emilia earthquake sequence (Mw = 5.9), which had a_g values up to 0.27g. Fig. 3b shows the out-of-plane collapse of the external layer in some infills of the Mirandola hospital which was consequently declared unusable after the event (Masi *et al*. 2013).

3. Review of seismic code provisions on masonry infills

Specific measures for masonry infilled frames are provided in many seismic codes all over the world (e.g. FEMA standards in US, NZSEE Recommendations in New Zealand, Eurocode 8 in Europe). In particular, FEMA 306 report (FEMA 1998a) provides guidance on damage classification and performance analysis. Infilled panels are categorized according to material and geometric configurations showing that in the US building stock, brick masonry cavity walls are very common for exterior facades. Furthermore, different capacity models are provided to assess the infill strength. In particular, for out-of-plane failure the Angel and Abram (1994) model is reported, for which the ultimate lateral pressure is a function of the masonry strength, the slenderness ratio, the prior in-plane damage and the flexural rigidity of the frame.

Supplementary information is reported in FEMA 307 standard (FEMA 1998b) derived from theoretical analyses of the effects of prior damage on single-degree-of-freedom mathematical models, and experimental tests on masonry infilled frames. Specifically, experimental results show that it is unlikely that out-of-plane failure would occur for usual infill height-to-thickness aspect ratios, thus suggesting that if an out-of-plane failure is observed, then contribution from prior in-plane damage can be expected.

In FEMA 356 pre-standard (FEMA 2000) modelling, safety verification and acceptance criteria are provided for RC existing frames with infills. It is prescribed that the masonry infills with height-to-thickness ratio (slenderness H/s) lower than some specified limit values should not be verified against out-of-plane seismic forces. Slenderness limit values are reported in Table 1 as a function of the required performance levels (Immediate Occupancy IO, Life Safety LS, and Collapse Prevention CP) and of the seismic zone.

In the New Zealand Recommendations (NZSEE 2006) approaches and procedures for the seismic evaluation of existing buildings of various materials and configurations are described, and some guidance for improving their performance is also offered. Section 9 is specifically devoted to the detailed assessment of moment resisting frames with masonry infill panels. After a short discussion on the possible effects of infills on frames (e.g. infills not affecting or significantly contributing to the structural response), in-plane behaviour of infill panels is addressed. Provisions to evaluate out-of-plane infill strength are also given adopting the same capacity model reported by FEMA 306, e.g. Angel and Abram (1994).

EC8-1 (CEN 2004) provides additional measures for masonry infilled frames. Firstly, specific criteria and requirements to take into account irregularities due to masonry infills, both in plan and in elevation, are reported. This is followed by the provision of some measures for damage limitation. Particularly, it is prescribed that *"appropriate measures should be taken to avoid brittle failure and premature disintegration of the infill walls (in particular of masonry panels with openings or of friable materials), as well as the partial or total out-of-plane collapse of slender masonry panels. Particular attention should be paid to masonry panels with a slenderness ratio (ratio of the smaller of length or height to thickness) of greater than 15"*. EC8-1 specifies the conditions masonry infills have to fulfil to apply these measures, that is: (i) infills are constructed after the hardening of the concrete frames, (ii) they are in contact with the frame (i.e. without special separation joints), but without structural connection to it, (iii) they are considered in principle as non-structural elements. No further provisions are provided in EC8-3 for existing buildings (CEN 2005), but reference is specifically made to EC8-1 provisions.

The current Italian code NTC 2008 also offers criteria and requirements to take into account irregularities due to masonry infills and provisions for their damage limitation, in accordance with EC8-1.

Table 1 Maximum H/s ratios: out-of-plane analysis shall not be required for infills with H/s ratios less than the values listed in the table (adapted from (FEMA 2000)).

	Low seismic zone	Moderate seismic zone	High seismic zone
IO	14	13	8
LS	15	14	9
CP	16	15	10

In both EC8-1 and NTC 2008 the effects of the seismic action on non-structural elements may be determined by applying a force normal to the plane on the infill (F_a) defined as

$$F_a = S_a \cdot W_a \cdot \gamma_a / q_a \tag{1}$$

where S_a is the seismic coefficient, W_a the weight of the element, γ_a the importance factor and q_a the behaviour factor.

The seismic coefficient Sa is given by the following expression

$$S_a = \alpha \cdot S \cdot \left[\frac{3 \cdot (1 + Z/H)}{1 + (1 - T_a/T_1)^2} - 0.5 \right] \tag{2}$$

where, for the considered limit state, α is the ratio of the peak ground acceleration on ground type A (a_g) to the acceleration of gravity g, S is the soil factor, T_a and T_1 are the fundamental vibration periods of the non-structural element and of the building, respectively, H is the building height measured from the excitation level, and Z is the elevation of the non-structural element above the level of application of the seismic action. The behaviour factor q_a may be taken equal to 2 for masonry infills, as prescribed in the codes (see Table 4.4 in EC8-1). No capacity models are provided to verify masonry infills against the out-of-plane collapse.

4. Out-of-plane performance of infill panels

Although only a limited number of experimental and analytical studies have been performed so far on the out-of-plane behaviour of infill panels, the main factors affecting this behaviour have been identified. In particular, the out-of-plane response of infill panels confined within frame elements is governed by the arching mechanism, providing ultimate capacity values mainly depending on the panel slenderness and the compressive strength of the constituent materials. Besides, as reported by to Hamed and Rabinovitch (2008), out-of-plane dynamic response also depends on the non-linear effects due to both geometry and constitutive materials.

Furthermore, the out-of-plane capacity can suffer large reductions due to ineffective connection with the surrounding frame elements that, in turn, could be caused by both poor quality of execution and prior in-plane damage. According to Braga *et al*. (2011), the out-of-plane response of infills in RC frames can be associated with two limit conditions:

1) in case of effectively confined infills and no prior in-plane damage, seismic response depends on the arching mechanism (Fig 4a);

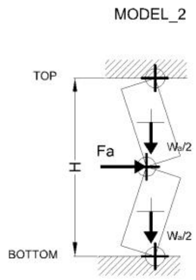


Fig. 4(a) Arching mechanism

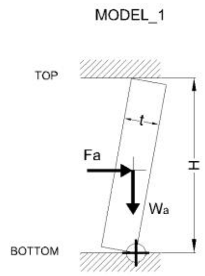
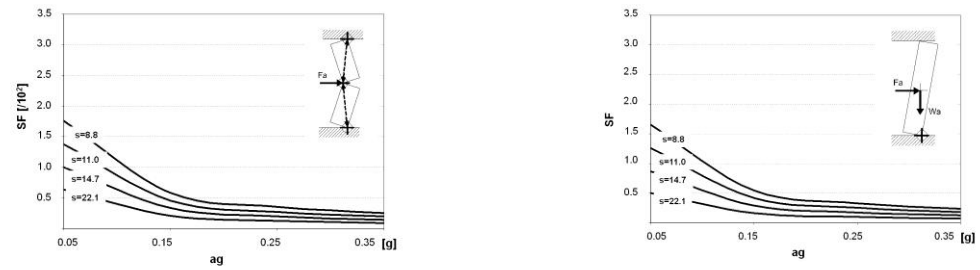


Fig. 4(b) Rigid-body mechanism



(a) (b)

Fig. 5 Safety factor (SF) values for a masonry panel placed at the 4th storey with slenderness varying from s = 22.1 to s = 8.8, considering both the arching (a) and the rigid-body mechanisms (b)

2) for infills having a weak connection with the surrounding frame elements due to bad execution and/or prior in-plane damage or for untied veneer wall construction, infill seismic response can be represented by a rigid-body mechanism (Fig 4b).

Very different values of the out-of-plane capacity can be estimated for the two limit conditions as obtained from a parametric analysis carried out by Braga *et al*. (2011) considering different values of the panels' slenderness. The out-of-plane capacity based on the formation of the arching mechanism has been calculated using the expression related to the stability of masonry walls given in EC6 (CEN 1998)

$$q_{lat} = f_u \cdot (\frac{t}{H})^2 \tag{3}$$

where q_{lat} is the limiting lateral failure pressure, f_u is the compressive strength of the masonry wall, t and H are the thickness and height of the masonry infills, respectively. Results from Eq. (3) were compared with the seismic actions computed applying Eq. (1) to the infills of a typical four storey RC framed structure. Considering both capacity models, the safety factor values (i.e. the demand to capacity ratio) were evaluated at each storey. Results are shown in Fig. 5 as a function of slenderness (height to thickness ratio varying in the range 8.8 ÷ 22.1) and ground acceleration (a_g varying in the range 0.05 ÷ 0.35 g). For ground acceleration values a_g > 0.15g the safety factor values corresponding to the rigid-body mechanism are always lower than 1 (i.e. not verified) also for the lower slenderness value (s = 8.8). On the contrary, SF is always much greater than 1 (i.e. verified) when the arching mechanism is considered (note that SF values displayed in Fig. 5 are divided by 100).

Earthquakes occurring in the past have shown that damage to the infills at the upper storeys is generally absent, despite the higher seismic forces at these storeys. In fact, out-of-plane damage is generally located at the lower stories where, due to in-plane drift values greater than those generally experienced at the upper storeys in RC frame structures, infills can suffer early damage due to in-plane actions, which reduces the connection effectiveness with the structural members and, consequently, can determine out-of-plane collapse. Nevertheless, the rigid-body mechanism appears to largely underestimate the real out-of-plane capacity of in-plane damaged infills. Thus, even if severe damage to the connection sections were present, a low boundary resistance able to give a not negligible contribution to the stability in orthogonal direction can be generally expected.

To better understand the out-of-plane behaviour of masonry infills, some experimental studies and capacity models able to assess the out-of-plane capacity through both considering and neglecting the effects due to prior in-plane damage, are briefly described in the following sections.

4.1 Experimental results

A limited number of experimental investigations specifically aimed at understanding the out-of-plane behaviour of masonry infills have been performed so far. Modena and Da Porto (2005) performed out-of-plane tests on nine infill strips having dimensions 30x100x252 cm (thickness, width and height, respectively) made up of hollow clay bricks. The specimens were constructed arranging the hollow bricks with their holes in either vertical or horizontal direction, and considering different mortar layer arrangements. Infill strips were connected to a rigid support and subjected to monotonic out-of-plane forces applied at mid-height until reaching the point of collapse. On the basis of the mechanical properties of the constituent materials (i.e. bricks and mortar) the infill compressive strength values were computed in accordance with the EC6 provisions (CEN 1998) obtaining values in the range of 5.94-18.26 MPa. Test results show that the collapse load increased almost proportionally with the compressive strength, ranging from 57.1 to 181.9 kN. These values appear to be much greater than the seismic actions generally expected in RC buildings to be applied on infill panels. As an example, by comparing the capacity values provided by the experimental tests with the seismic actions at the upper level of buildings with 5-10 storeys (evaluated considering a_g = 0.35g), the safety factors (capacity to demand ratio) are always higher than 1, ranging from 12 to 39.

Nine hollow concrete block panels infilling a steel portal (3.6 × 2.8 m, width by height value, respectively) subjected to uniform pressure orthogonal to the panel surface until collapse were tested by Dawe and Seah (1989). Different thickness values (3 cases: 90, 140 and 190 mm) and boundary conditions (3 cases) were considered and the effect of a central opening window was also evaluated. The infill compressive strength normal to bed joints is equal to 24.3 MPa. Initially, the infill resisted lateral applied loads by flexural action while, in the post-cracking range, the arching mechanism was activated. This latter action lessened as a function of the panel slenderness and of the connection effectiveness between the panel and the surrounding frame: in the case of a gap between the panel and the top beam, the out-of-plane capacity decreased by about 25% with respect to a fully restrained panel. Finally, in the case of a small central opening no significant reduction of the arching capacity was found.

Flanagan and Bennett (1993) studied infill panels subjected to earlier damage in-plane. They carried out monotonic load tests on one bay-one storey steel frames with dimensions of 2.25 by 2.37 m and 2.86 by 3.47 m. The infills were made up of hollow clay blocks having either 200 mm

Table 2 Results of out-of-plane experimental tests. * refers to a concentrated force applied at the mid-height; ** refers to an uniform pressure on the panel surface

Experimentation	*Compressive strength* $f_m^{'}$ *[MPa]*	*Slenderness H/t*	*In-plane drift*	*Ultimate out-of-plane capacity*
Modena and Da Porto (2005)	5.94 18.26	8.4	0%	57.1kN* 181.9kN*
Dawe and Seah (1989)	24.3	14.7 20 31	0%	19.2kPa** 11.2kPa** 7.8kPa**
Calvi and Bolognini (2001)	1.1	20.3	0% 0.4% 1.2%	5.6kPa** 1.5kPa** 1.0kPa**

or 330mm nominal thickness (in the latter case a double layer infill type with 100 mm and 200 mm thick panels was adopted), laid with horizontal holes. The infill compressive strength normal to bed joints is equal to 5.6 MPa. Several tests considering either alternate in- and out-of-plane load sequences or simultaneous combination of them were performed. Out-of-plane loads were simulated by applying incremental loading-unloading pressure cycles with an airbag. Results show that the out-of plane capacity of infill panels decreases with prior in-plane damage. On the contrary, no significant reduction of in-plane capacity due to prior out-of-plane loaded panels was observed.

Angel *et al*. (1994) carried out tests on one bay-one storey RC frames with masonry infills made up of either clay brick or concrete block having different compressive strength (8 cases), thickness (5 cases) and type of mortar (2 cases). Tests were arranged in order to analyse both in- and out-of-plane response of infills. To this purpose, the specimens were loaded by alternating monotonically increasing in-plane and out-of-plane forces: the former is applied at the top of the frame while the latter was applied across the infill surface with an airbag able to apply a uniform load. Furthermore, vertical loads were applied to estimate the influence on the infill seismic behaviour of the gravitational loads transferred by the over floor. Out-of plane capacity was evaluated for different values of the top displacement corresponding to increasing in-plane damage levels. Results show that the drift value corresponding to the first crack on the infill panels (Δ_{cr}) is in the range 0.031-0.195%, respectively for the lower and the higher considered compressive strength values of the masonry panel, which are in the range 3.5-22.9 MPa. For infill panels with the higher slenderness ratio (equal to 34), prior in-plane cracking strongly reduces the out-of-plane capacity: in particular, for in-plane drift values equal to 0.21% and 0.34%, the reduction of the out-of-plane capacity was 27% and 51%, respectively. By determining a better connection with the surrounding RC beam, vertical loads increase the initial stiffness of panels whereas they give a negligible contribution to the out-of-plane capacity.

Calvi and Bolognini (2001) carried out monotonic tests on infill panels having dimensions of 420x275 cm (width by height value, respectively) confined within one bay-one storey RC frame and subjected to different in-plane drift values. The thickness value was 135 mm, as a result of clay block thickness (115 mm) and 10 mm thick plaster on both sides. Infill panels are made up of hollow clay bricks placed with their holes along the horizontal direction. The infill compressive strength normal to the bricks' holes is equal to 1.10 MPa, which can be considered representative of infill panels commonly constructed in the Mediterranean area. Tests were performed by applying initially in-plane displacement cycles according to pre-defined inter-storey drift ranging from 0.1% to 1.2% and, subsequently, applying the out-of-plane load. Results show that, from zero drift (no in-plane damage) to drift values of 0.4% and 1.2%, the reduction of the out-of-plane capacity was equal to about 73% and 82%, respectively.

In table 2 the main results from some of the described experimental programs are summarized. They aim at highlighting the role of the most influential factors on the panel out-of-plane capacity, i.e. infill compressive strength, slenderness and in-plane drift.

4.2 Capacity models

In the following some capacity models referring to the arching mechanism and able to estimate the out-of-plane response of infill panels are briefly described, and their results are compared with experimental results.

The first formulation based on the arching theory was proposed by McDowell *et al*. (1956). According to this approach, the resisting mechanism of panels subjected to uniform orthogonal loads can be idealized by two struts going from the hinges at the bottom and the top of the panel to that at mid-height (Fig 4a). Starting from the kinematic relationships obtained by considering a strip panel fully restrained at two opposite edges, and assuming the elastic behaviour of constituent materials, the ultimate out-of-plane capacity can be computed as follows

$$q_u = 1.7 \cdot \frac{f_m^{'}}{\left(\frac{H}{t}\right)^2} \cdot \left(1 - \frac{f_m^{'}}{E_v \varepsilon_v}\right)^2 \tag{4}$$

where $f_m^{'}$ and E_v are the compressive strength and the elastic modulus of the infill material, respectively, and $\varepsilon_v = \frac{\sqrt{\left(\frac{H}{2}\right)^2 + t^2} - \frac{H}{2}}{\sqrt{\left(\frac{H}{2}\right)^2 + t^2}}$ is the ultimate strain, all referred to the vertical direction. H and t are, respectively, the height and the thickness of the masonry infills.

As a consequence of the adopted assumptions, the McDowell model is ascribable to one-way arching mechanisms.

With the aim of considering panels restrained at all four sides by the surrounding structural members (i.e., panels able to activate the arching mechanism along two orthogonal directions), a generalized McDowell model was proposed by Bashandy *et al*. (1995). Starting from geometric relationships based on the cracking pattern of confined panels (yield-line approach) and assuming a uniform compressive stress over the whole compression area, the lateral uniform pressure q_u causing out-of-plane failure can be estimated through the following expression

$$q_u = \frac{8}{H^2 L}\left\{M_{yv}\left[(L-H) + H \cdot ln2\right] + M_{yh} \cdot \left(\frac{x_{yv}}{x_{yh}}\right) ln\left(\frac{L}{L - H/2}\right) \cdot L\right\} \tag{5}$$

where L is the panel width, $M_{yv}=\frac{0.85f_m'}{4}\left(t-x_{yv}\right)^2$, $M_{yh}=\frac{0.85f_m'}{4}\left(t-x_{yh}\right)^2$, $x_{yv}=\frac{t\cdot f_m'}{1000\cdot E\cdot\left[1-\frac{H}{2\sqrt{(H/2)^2+t^2}}\right]}$, $x_{yh}=\frac{t\cdot f_m'}{1000\cdot E\cdot\left[1-\frac{L}{2\sqrt{(L/2)^2+t^2}}\right]}$.

Based on several experimental tests performed on infilled steel frames (see Section 4.1), Dawe and Seah (1989) proposed two empirical expressions which, in addition to the thickness and the compressive strength of the infill panels, also take into account the boundary conditions and the frame rigidity. For a panel restrained on four sides the proposed expression is as follows

$$q_u=800\cdot f_m^{'\,0.75}\cdot t^2\cdot\left(\frac{\alpha}{L^{2.5}}+\frac{\beta}{H^{2.5}}\right) \tag{6}$$

where $\alpha=\frac{1}{H}\left(EI_CH^2+GJ_CtH\right)^{0.25}\le 50$ and $\beta=\frac{1}{L}\left(EI_CL^2+GJ_btL\right)^{0.25}\le 50$

In the latter expressions E and G are the Young and the shear modulus of the frame members, I_c and I_b are the moments of inertia of columns and beams, J_c and J_b are the torsional constants of columns and beams, respectively.

Angel *et al*. (1994) proposed an expression to estimate the out-of-plane capacity including the effects due to prior in-plane damage. The proposed expression derives from both equilibrium conditions on a strip panel fully restrained between two rigid supports and experimental results from tests on infills earlier damaged in-plane (see Section 4.1). The ultimate uniform pressure q_u of infill panels confined into frame elements can be evaluated as follows

$$q_u=\frac{2f_m'}{\frac{H}{t}}\cdot R_1\cdot R_2\cdot\lambda \tag{7}$$

where

λ is a strength factor dependent upon the H/t ratio;

R_2 is a reduction factor to account for the confining frame flexibility equal to:

$R_2=0.357+7.14\cdot 10^{-8}EI$ for $2.0\cdot 10^6\le \mathrm{EI}\le 9.0\cdot 10^6$

$R_2=1$ for $\mathrm{EI}>9.0\cdot 10^6$ (with EI in $\mathrm{k}\cdot\mathrm{in}^2$ units);

R_1 is a strength reduction factor accounting for previous in-plane damage equal to

$$R_1=\left[1.08+\left(\frac{H}{t}\right)\left(-0.015+\left(\frac{H}{t}\right)\left(-4.9\cdot 10^{-4}+1.3\cdot 10^{-5}\left(\frac{H}{t}\right)\right)\right)\right]^{\frac{\Delta}{2\Delta_{cr}}} \tag{8}$$

where Δ is the displacement demand, Δ_{cr} is the displacement corresponding to the first infill crack. R_1 is equal to 1 for $\Delta/2\Delta_{cr}<0.5$.

By adopting the expression provided by Angel *et al*. a good agreement with the experimental results obtained by Calvi and Bolognini (2001) can be found. Specifically, comparing experimental and analytical results (where R_2 = 1, Δ_{cr} = 0.2%H, H/t = 20.4 and λ = 0.021 are considered) differences equal to -7.3% and +28% are found for in-plane drift values equal to 0.4%

and 1.2%, respectively. It is worth specifying that negative differences signify experimental values less than analytical ones.

5. Modelling interaction between in-plane and out-of-plane response

On the basis of the analyses carried out on the capacity models and experimental studies presented in the previous sections, a procedure is proposed which is able to take into account:

1) the role of in-plane damage on the out-of-plane ultimate capacity, from which a capacity model for the local verification of infill panels is derived;

2) the loss of in-plane contribution to the lateral load resistance due to early out-of-plane failure of infill panels, from which a capacity model for the global analysis of infilled RC frames is derived.

Specifically, in order to verify the stability conditions of infill panels within frames subjected to out-of-plane seismic forces, it is firstly necessary to evaluate the in-plane damage level as a consequence of the inter-storey drift value (Δ/h value in Fig. 6). Secondly, by adopting a relationship between the out-of-plane behaviour and the in-plane damage based on the study of Angel *et al*. (1994), the ultimate out-of-plane capacity ($F_{out,C}$, Fig. 6) related to the computed inter-storey drift can be estimated. Finally, out-of-plane stability is verified by comparing the orthogonal force acting on the panel (for example using Eq. 1) and the capacity value calculated above.

With respect to point 2), note that out-of-plane behaviour of infills is characterized by low plastic deformation values (brittle behaviour) with a strong degradation beyond the maximum capacity. Furthermore, out-of-plane failure determines the expulsion of the panel from its side and, consequently, its in-plane contribution to the lateral load resistance of the whole structure vanishes. Therefore, to account for the effects due to early out-of-plane failure on in-plane behaviour, the force-displacement (F_{in}-d_{in}) relationship which is generally adopted in equivalent strut infill modelling needs to be modified. To this purpose, a procedure to define an in-plane F_{in}-ID_{in} relationship considering both out-of-plane failure and capacity reduction due to inter-storey drift is proposed. Fig. 7 displays the relationship between the out-of-plane capacity and the in-plane drift (F_{out}-ID_{in}) in accordance with the expression proposed by Angel *et al*. (see Fig. 7b), together with a commonly adopted force-drift (F_{in}-ID_{in}) relationship to describe the in-plane behaviour of masonry infills (Fig. 7c).

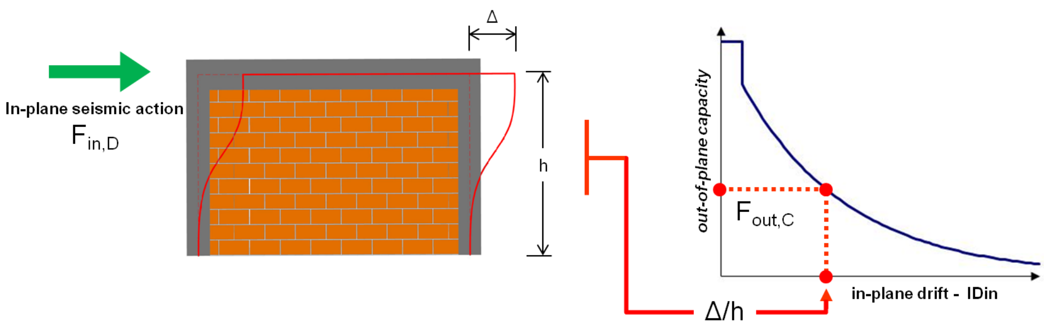


Fig. 6 Definition of the ultimate out-of-plane capacity related to the in-plane drift value (Δ/h)

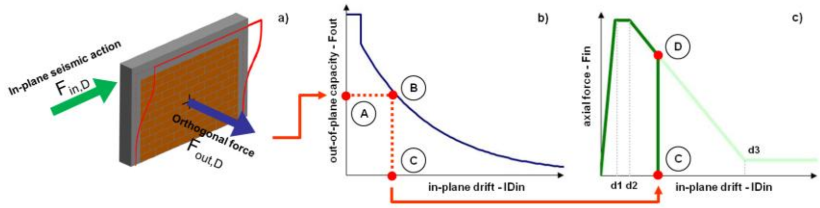


Fig. 7 Definition of in-plane F-d relationship (c) accounting for early out-of-plane failure (b) of infill panels subjected to both in-plane and orthogonal seismic action (a)

Starting from the F_{out}-ID_{in} relationship, for a given orthogonal force $F_{out,D}$ acting on the panel (point A in Fig. 7b), it is possible to determine the in-plane drift value ID_{in} (point C in Figs. 7b and 7c) in which $F_{out,D}$ is equal to the out-of-plane capacity (point B in Fig. 7b). For this ID_{in} value, the out-of-plane ultimate capacity is reached and, as a consequence, the contribution of the in-plane infill strength and stiffness to the lateral load capacity of the whole building should be removed. Therefore, in correspondence with the ID_{in} value at point C, the in-plane strength (in terms of the axial force acting on the equivalent diagonal strut) drops to zero in the F_{in}-ID_{in} relationship (point D in Fig. 7b).

Summarizing, with regard to the infill performance (see Section 6.3 for further details), the ultimate limit state for in-plane actions is reached if the drift demand is greater than a suitable value conventionally assumed (i.e., d3 value in Fig. 7c), while the ultimate limit state for out-of-plane actions is reached for drift demand values greater than the ID_{in} value at point C (see Fig. 7c) as obtained from Fig. 7b.

The proposed procedure permits the modelling of the in-plane response of infill panels and its effects on the out-of-plane capacity, and viceversa. Specifically, it permits: (i) to take into account the reduction on the out-of-plane capacity of infill panels due to in-plane drift, and (ii) the loss of the in-plane reaction in case of prior out-of-plane failure. In this way, it is possible to assess the seismic performance of both infills and structural members by taking into account their mutual interactions.

A conservative condition has been pursued with the aim of proposing a safety verification procedure to be used within a design structural code. To this end, the seismic response can be effectively evaluated through non-linear static analyses (NLSA) which are largely adopted in design practice and should provide more conservative results with respect to non-linear time-history analyses (NLTH). Moreover, although NLTH analyses could provide the times when the peak storey drift demand (inertial force along the in-plane direction) and the peak out-of-plane acceleration (inertial force along the orthogonal direction) occur, this prediction can be significantly dependent on the selected time-history (Masi *et al*. 2011b), and thus be rather unreliable as different time-histories could provide different results.

Finally, it has to be pointed out that a consideration of the simultaneous action of the maximum values of the seismic demand on infill panels (i.e. both inter-storey drift and out-of-plane force) is not necessary. In fact, for code complying safety verifications, possible out-of-plane failure depends on the amount of damage due to the in-plane drift (occurring at a certain time) and on the orthogonal force acting on the infill panel at a different time.

6. An application to the seismic assessment of RC buildings

In this section the proposed procedure is applied to assess the seismic performance of a structural type representative of real RC existing buildings. Comparisons between the results found when considering and when neglecting out-of-plane collapse of infills are carried out to provide suggestions for modelling in the seismic assessment of RC buildings.

6.1 Building type description and modelling

The procedure has been applied to a four-storey (4s) existing RC framed structure belonging to the post-1971 Italian building stock designed only to vertical loads (simulated design), according to procedure proposed in Masi (2003) and further detailed in Masi and Vona (2012).

The structural type under study has a rectangular shape in plan (Fig. 8) with total dimensions 22.5×10.0 m (X and Y direction, respectively) and constant inter-storey height equal to 3 m. The structure has lateral load resisting frames only along the longitudinal direction X, with constant bay length equal to 5 m (2.5 m for the bay in correspondence of the staircase). Along the transversal direction Y, the structure has two bays (5m long) with rigid beams (30x50 cm) in the exterior frames and one-way RC slab along the interior frames.

Masonry infills are made up of two panels (cavity walls with 8 and 12 cm thick panels) of hollow brick masonry with effective thickness equal to 20 cm. More details on the structural characteristics of the building under study can be found in (Masi *et al*. 2012).

Structural modelling was performed using the finite element code SAP2000 (1995). Large openings are typically present in the infill walls along the longitudinal direction, therefore the lateral load resistance of these infills was neglected. On the contrary, the infill panels in the external frames along the Y direction were modelled by using an equivalent diagonal strut, whose area was determined by multiplying the panel thickness (t) by an equivalent width (w). The expression from Papia *et al*. (2003) was used to compute w, providing a value equal to about 110 cm (ratio w/d = 0.19, where d is the diagonal length of the infill panels). A macro-modelling based on lumped plasticity was adopted to describe the non linear seismic behaviour of the RC members.

At both ends of each structural member a bending moment–rotation relation was defined through a bi-linear curve described by the values of the yielding moment (M_y), and of the yielding (θ_y) and ultimate (θ_u) chord rotation values. θ_y and θ_u were evaluated according to the expressions provided in EC8-3 (CEN, 2005). When a brittle failure was predicted, the M-θ relation above mentioned was appropriately modified considering a bending moment value $M(V_{Rd})$ calculated as a function of the ultimate shear resistance V_{Rd}.

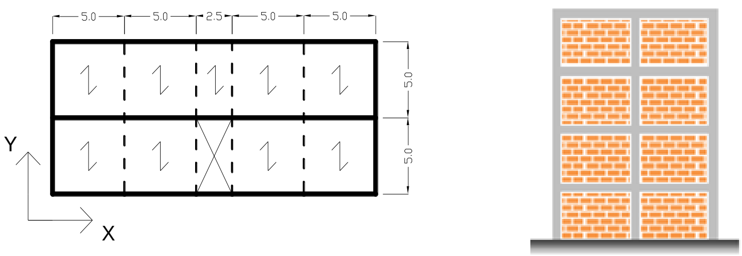


Fig. 8 In plan and in elevation layout of the building type under study

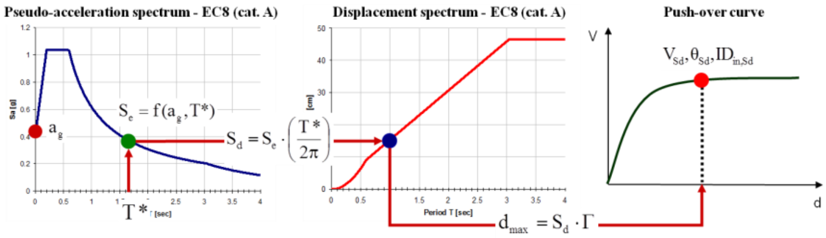


Fig. 9 Scheme of the procedure to assess the seismic performance of the building

On the basis of the typical mechanical properties of the constituent materials found in real buildings, concrete strength value (f_{cm}) equal to 18MPa and steel strength value (f_{ym}) equal to 400MPa were taken into account in evaluating the capacity of the structure under examination. A confidence factor value equal to 1 (CF = 1) was assumed referring to an exhaustive knowledge level (CEN 2005). The non linear and degrading behaviour of the diagonal strut simulating the masonry infill panels was modelled through a plastic hinge located at the ends of the strut acting only under compression loads. The axial force-displacement relation F-d (Fig. 7b) was defined on the basis of the following parameters: ultimate strength $F_{u,in} = w \cdot t \cdot f_w = 189\text{kN}$, where f_w=1.20 MPa is the compressive strength of the masonry, residual strength equal to $10\%F_{u,in}$, $d_1 = 0.1\%$, $d_2 = 2d_1$ and $d_3 = 10d_1$.

In accordance with the procedure described in Section 5, the F_{in}-ID_{in} behaviour was modified to account for possible premature out-of-plane failure. The Angel *et al.* (1994) relationship was defined assuming the following parameter values: R_2=1, λ=0.021, and Δ_{cr}=0.2% h.

6.2 Evaluation of demand parameters

Seismic performance of the building under study were evaluated through Non-Linear Static (pushover) Analyses (NLSAs) according to the Italian NTC08 code provisions, which are substantially consistent with EC8 provisions (CEN 2004 and 2005). Pushover curves were determined under conditions of constant gravity loads and monotonically increasing horizontal loads according to a modal pattern distribution for each of the two orthogonal directions in plan.

A bi-linear curve relevant to an idealized equivalent SDOF system was computed following the provisions outlined in the NTC08 code. Seismic performance were evaluated starting from the period of vibration T^* of the equivalent system assuming an EC8 elastic spectrum with ground type A (Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface). In particular, for a given value of the peak ground acceleration (a_g) it is possible to evaluate the spectral pseudo-acceleration (S_e) and the spectral displacement (S_d) relevant to T^*.

Consequently, the target displacement of the MDOF system (performance point, d_{max}) is calculated by multiplying S_d by the modal participation factor (Γ). On the basis of the determined push-over curve, the seismic demand corresponding to the d_{max} value is evaluated in terms of either shear (V_{Sd}, for brittle elements) or rotation (θ_{sd}, for ductile elements) for the structural elements, and in terms of inter-storey drift values ($ID_{in,Sd}$) for the infill panels (Fig. 9).

In the definition of the expected behaviour (i.e. either ductile or brittle) of the structural members, the presence and integrity of the infill panels have been neglected. However, the influence of the infill response has been considered in the evaluation of seismic demand on the structural members.

Safety verifications relevant to the Limit State of Life Safety (LSLS) according to NTC08 (corresponding to the performance requirements of the Limit State of Significant Damage according to EC8-3) were carried out considering four values of peak ground acceleration (a_g = 0.05g, 0.15g, 025g, 0.35g). For each a_g value the F_{in}-ID_{in} relation relevant to each panel was modified, taking into account the related out-of-plane seismic demand, according to the proposed procedure. Coherently with a code-based safety verification (and taking into account the lack of alternative formulations for MDOF structures), the out-of-plane seismic demand has been evaluated adopting the EC8 expression (see Eq. 1 at Section 3). Eq. (1) was applied assuming the following values of the parameters: behaviour factor q_a = 2.0, soil factor S = 1.0, fundamental period of masonry infills T_a = 0.18sec (assuming uncracked stiffness properties of masonry panels and only translation constrains along their boundaries), and fundamental period of the structure T_1 = 0.8sec.

Infill performance were evaluated in correspondence with the performance point relevant to each a_g value by comparing the inter-storey drift values with the performance levels, as defined in the following Section 6.3.

Further, iterating on the a_g value, the global seismic performance of the building were evaluated making reference to the Limit State of Life Safety according to NTC08. Specifically, the minimum a_g value causing either $\theta_{sd} = ¾\ \theta_u$ (for ductile structural elements) or $V_{Sd} = V_{Rd}$ (for brittle structural elements) or the value of inter storey drift relevant to the infill failure (in-plane or out-of-plane) was calculated.

6.3 Seismic performance of infills

The Italian code NTC08 defines four limit states, that is: the Operational Limit State (LSO), the Limit State of Damage (LSD, corresponding to the limit state of damage limitation according to EC8), the Limit State of Life Safety (LSLS, corresponding to the limit state of significant damage according to EC8), and the Limit State of Collapse (LSC, corresponding to the limit state of near collapse according to EC8).

For LSO and LSD limit states, the Italian code provides design limits in terms of inter-storey drift as a function of the infill type. In particular, for buildings having non-structural elements of brittle materials attached to the frame structure, the inter-storey drift relevant to LSO and LSD are equal to 0.33% and 0.5%, respectively. It is worth noting that the limit values of inter-storey drift related to LSO and LSD do not appear consistent with the levels of damage generally obtained in experimental studies (Colangelo 2005; Calvi and Bolognini 2001; Hak *et al*. 2012). Therefore, the performance levels of the masonry infills under study have been defined by interpreting the experimental results according to the NTC08 state limit requirements. Particularly, Fig. 10 shows the range of the inter-storey drift values related to each considered limit state. From 0 to 0.1% drift values (segment A-B in Fig. 10), masonry infills are in the elastic range and no damage is expected (infill performance consistent with the LSO). For drift values in the range 0.1-0.2%, infills achieve the maximum strength and the damage level is consistent with the LSD. In the range C-D, the in-plane capacity decreases and the damage level can be associated to the LSLS. For drift values larger than 1.0% in-plane failure is assumed (D(in)). When the out-of-plane failure is predicted, the value of in-plane drift is marked with D(out).

On the basis of the previous criteria, it is possible to predict the expected infill failure mechanisms. Specifically, for each infill panel the out-of-plane force ($F_{out,D}$, Fig. 7) acting on it has been preliminarily evaluated as a function of the considered storey and a_g value. Starting from these force values, and according to the procedure shown in Fig. 7, the in-plane drift value (ID_{in}) at which $F_{out,D}$ is equal to the out-of-plane capacity is determined (point C in Fig. 7b). Considering the adopted F_{in}-ID_{in} relationship (Fig 7c, as defined at Section 6.1), if the ID_{in} value is higher than 1%, then in-plane failure is expected. On the contrary, if ID_{in} is lower than 1%, then infill panel fails by out-of-plane mechanism. Table 3 reports the failure mechanism that can be expected for the infill panels at each storey (s1-s4) considering all adopted a_g values. In-plane failure is indicated with IN, while OUT is used when out-of-plane failure can be predicted.

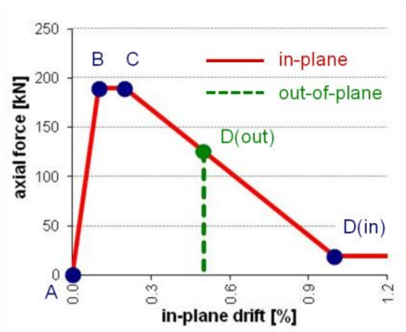
With reference to the higher a_g values (i.e., 0.25-0.35g), it is expected that out-of-plane failure would precede in-plane failure at every storey, because the drift values that would match out-of-plane demand and ultimate capacity should always be less than 1%. For a_g = 0.15g, out-of-plane failure can be expected only for the infills at the upper storeys. Finally, for a_g = 0.05g, in-plane failure can always be predicted.

Table 3 Expected failure mechanism of the infill panels in the case study (IN=in-plane failure, OUT=out-of-plane failure)

	a_g [g]			
Storey	0.05g	0.15g	0.25g	0.35g
s1	IN	IN	OUT	OUT
s2	IN	IN	OUT	OUT
s3	IN	OUT	OUT	OUT
s4	IN	OUT	OUT	OUT

Table 4 Results of NLSAs on the case study in terms of infill performance with respect to limit states' verification (LSO: Operational Limit State, LSD: Limit State of Damage, LSLS: Limit State of Life Safety)

	a_g [g]			
Storey	0.05g	0.15g	0.25g	0.35g
s1	*LSO*	*LSD*	*LSLS*	*LSLS*
s2	*LSO*	*LSLS*	*LSLS*	*D(out)*
s3	*LSO*	*LSD*	*LSD*	*LSLS*
s4	*LSO*	*LSO*	*LSO*	*LSO*



Range	*Performance level*
A-B	LSO
B-C	LSD
C-D	LSLS
D (in)	In-plane failure
D (out)	Out-of-plane failure

Fig. 10 Performance levels of the infills

Whereas Table 3 offers results regarding the infill failure mechanism that can occur in theory, based on the results of the NLSAs, the seismic performance of infills actually found on the building under study can be evaluated with respect to all Limit States. The results are reported in Table 4, in accordance with the performance levels previously defined. The drift values at each storey obtained from the analyses with a_g = 0.05g are less than 0.1%, thus the LSO is verified for all infill panels. The analyses with a_g = 0.15g show drift values at the bottom storeys in the range 0.1-0.2% and, consequently, the performance of infills are substantially consistent with the LSD limit state, while the infills at the upper storey are subjected to drift values lower than 0.1% (LSO). For the analyses with a_g = 0.25g, the drift values at the first and second storey are larger than 0.2% and, consequently, a damage level consistent with the LSLS can be expected.

Out-of-plane collapse (D(out)) has been predicted only for the masonry infills at the second storey in case of a_g = 0.35g. For the infills at the first and third storey, drift values are close to 0.5% (LSLS), while the panels at the top storey are in the elastic range (LSO).

The results reported above depend on two factors that play opposing roles in function of the specific storey under consideration, that is: i) value of inter-storey drift, and ii) value of orthogonal forces acting on the panels. As shown in past earthquakes (Fig. 11), damage to masonry infills is generally located at the lower stories, whereas it is almost absent at the upper stories. In fact, in RC frames the maximum drift values are generally experienced at the lower storeys and it can cause in-plane damage to infill panels that, in turn, reduces their out-of-plane capacity. On the contrary, at the upper stories higher out of plane forces are present but combined with lower in-plane drift values.

6.4 Role of out-of-plane infill failure on global building performance

Taking to account that consistent panels are placed only along the Y direction, in the evaluation the seismic performance considering both the RC structure and the infill panels, the results relevant to the Y direction are analysed. The role of the out-of-plane infills' failure on the building performance is examined by comparing the results in terms of F-d behaviour defined when considering or when neglecting the effects of the out-of-plane failure.

In accordance with the proposed methodology (see Section 6.2), building performance are defined in terms of minimum a_g values determining the limit state of Life Safety (LSLS) considering both RC structure and infill panels. For the latter, the LSLS can be determined by either in-plane (D_{IN}) or out-of-plane failure (D_{OUT}) in accordance with the performance levels previously described.

Results show that, for the structural type under study, D_{IN} should be attained with a_g equal to 0.39g, whereas a lower value a_g = 0.29g has been evaluated for D_{OUT}. With respect to the performance of the RC structure, members fail in plastic rotation capacity for a_g around 0.43g. It is worth noting that this latter value is practically constant when considering or when neglecting the effects of the out-of-plane failure on the in-plane capacity (F-d relationship in Fig. 10). In other words, for the building type considered, the earlier infill failure has a negligible influence on the ultimate capacity of the RC structure.

A comparison between the push-over curves performed either considering (OUT) or not considering (IN) the out-of-plane failure of infill panels is shown in Fig. 12.

The performance points referred to the infill failure (D(in) and D(out)) and the LSLS of RC members (STR), are displayed. “IN” and “OUT” curves tend to coincide up to the out-of-plane failure of infills, D(out). Afterwards, the premature infill failure determines a strong reduction of

Fig. 11 Examples of damage/collapse of masonry infills typically placed at the lower storeys

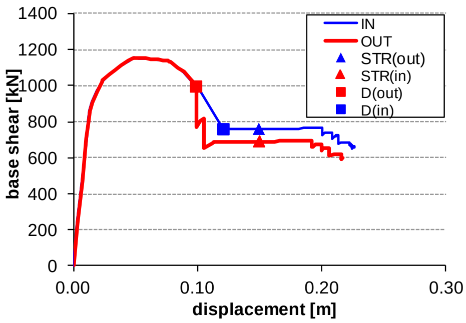


Fig. 12 Push-over curve along the Y direction defined either considering (OUT) or neglecting (IN) the out-of-plane failure of infills; the performance points relevant to the LSLS determined by both structural (STR) and infill (D) elements are displayed.

the base shear force of around 30%. The out-of-plane failure, D(out), precedes the in-plane failure, D(in), and, in terms of displacements, the difference between D(out) and D(in) is about -18%. Both out-of-plane and in-plane failure of infills precede the LSLS of RC members, which is reached for similar displacements considering IN and OUT analyses. “STR” displacements are larger than both D(in) and D(out), with differences equal to +18% and + 37%, respectively.

7. Conclusions

Recent seismic events have highlighted the crucial role of non structural elements on the seismic performance of RC framed buildings. Moreover as a consequence of the widespread damage suffered by infills, partitions and ceilings, a lot of private and public buildings have been judged unusable with serious socio-economic consequences. Furthermore, the external expulsion of infill panels due to out-of-plane failure has represented a major source of risk to life safety.

The main studies in the technical literature show that the out-of-plane response of infill panels confined within frame elements depends on the arching mechanism that, on one hand, can offer large capacity values with respect to the orthogonal seismic demand. On the other hand, out-of-plane capacity can be strongly reduced in the case of ineffective connection to the surrounding RC members, as a result of poor construction quality and/or possible prior in-plane damage.

The paper proposes a model to estimate the ultimate out-of-plane capacity of masonry infills, which is currently not provided in the Italian and European codes. More specifically, a procedure able to account for both the effects of the in-plane damage on the out-of-plane capacity and the influence on global response due to early out-of-plane failure of infill panels has been proposed and applied to a four-storey RC existing building.

Along the building direction where consistent infill panels give a remarkable contribution to the lateral load capacity the LSLS is caused by the infill failure. Specifically, the a_g value related to the out-of-plane failure is equal to 0.29g, less than the value related both to in-plane infill failure (a_g=0.39g) and RC structure failure (a_g=0.43g). Significant infill damage starts from medium-low ground accelerations (a_g around 0.15g) affecting the panels at the bottom storeys. Out-of-plane failure is found for the higher considered ground acceleration (a_g=0.35g): in this case, even though seismic forces increase along the building height, failure affects the infill panels at the second storey. With respect to the effects on the global seismic behaviour, even when the premature expulsion of infill panels has been observed, no soft storey mechanism has been found in the considered case study. Furthermore, no appreciable difference in terms of a_g relevant to the LSLS of RC members has been found as a consequence of the failure mechanism of the infill panels.

The role of infill panels on building performance and, specifically, the influence due to the out-of-plane behaviour, needs further study. Firstly, more experimental investigations on infill types typically of the Italian and European building stock should be carried out to define more accurate relationships between in-plane damage and out-of-plane capacity. Secondly, further analyses have to be performed considering different types of both infill panels and building structures. Finally, an integrated approach to both design of new building and assessment of existing ones should be defined, which is able to evaluate global seismic performance on the basis of the behaviour of all the components of the building system, as well as of their mutual interactions.

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