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Reliability of analytical models for the prediction of out-of-plane capacity of masonry infills

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Abstract. The out-of-plane response of infill walls has recently gained a growing attention and has been recognised fundamental in the damage assessment of reinforced concrete and steel framed buildings subjected to seismic loads. The observation of damage after earthquakes highlighted that out-of-plane collapse of masonry infills may occur even during seismic events of low or moderate intensity, causing both casualty risks and unfavourable situations affecting the overall structural response. Even though studies concerning the out-of-plane behaviour of infills are not as many as those focused on the in-plane response, in the last decades, a substantial number of researches have been carried out on the out-of-plane behaviour of infills. In this study, the out-of-plane response is investigated considering different aspects. First, damages observed after past earthquakes are examined, with the aim of identifying the main parameters involved and the most critical configurations. Secondly, the response recorded in about 150 experimental tests is deeply examined, focusing on the influence of geometrical characteristics, boundary conditions, prior in-plane damage, presence of reinforcing elements and openings. Finally, different theoretical capacity models and code provisions are discussed and compared, giving specific attention to those based on the arching theory. The reliability of some of these models is herein tested with reference to experimental results. The comparison between analytically predicted and experimental values allows to appreciate the extent of approximation of such methods.

Keywords: infill walls; masonry; frames; out-of-plane loads; experimental tests; predicting models; out-of-plane capacity

1. Introduction

Masonry walls are widely used as infills in steel and reinforced concrete framed structures. The presence of regularly distributed infills is generally beneficial due to their contribution to withstand seismic actions. Usually, stiffness and strength of the infill and connections between infill and frame are such that the infill affects the overall structural response. Uniformly distributed infills produce significant increase in both stiffness and strength, thus reducing the deformation demand and improving the energy dissipation capacity of the system. On the contrary, irregular arrangement of infills may cause unfavourable distribution of plastic hinges, high demand of inelastic deformations and reduction of the global dissipation capacity. Moreover, collapse of infills, which may develop both in-plane and/or out-of-plane, produces casualty risk and heavy socio-economic consequences, such as loss of building functionality. In this regard, the usability aspect is crucial, especially for buildings with emergency management functions, that need to remain fully operational after both frequent and rare seismic events. Furthermore, the total or partial failure of an infill sometimes produces adverse conditions affecting the overall structural response, like for example the formation of an open storey, which may result in a soft-storey collapse mechanism.

In the last decades, most studies focused on the in-plane response of infills and on their influence on the seismic response of framed structures (Liberatore and Decanini 2011, Di Trapani *et al.* 2015, Tarque *et al.* 2015, Liberatore *et al.* 2017). Some experiments have been carried out on single-storey, single-bay, frame specimens under in-plane horizontal loading showing the influence of the infills on the load carrying capacity (Mehrabi and Shing 1997, Cavaleri and Di Trapani 2014) and on the global seismic behaviour (Colangelo 2005), also analyzing the influence of size and shape of openings (Kakaletsis and Karayannis 2009). Several models have been also proposed and applied, such as those based on the equivalent strut approach (Mainstone 1974, Dolšek and Fajfar 2008, Kakaletsis and Karayannis 2009, Cavaleri and Di Trapani 2014, Asteris *et al.* 2016, Cavaleri *et al.* 2017, Mohammad Noh *et al.* 2017).

However, in recent years the concern for the out-of-plane behaviour has been growing also due to observation of damage after earthquakes.

Framed structures are usually designed and assessed based on static or dynamic analyses without directly considering the infill elements. However, many seismic codes provide specific additional measures for masonry infills, generally devoted to limit their damage and to control their possible negative effects on the frame. In any case, the presence of infills can be quite significant for safety evaluations.

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Fig. 1 Failure of infills due to poor infill-frame connection, L'Aquila, Italy, earthquake of 2009

Fig. 2 Failure of infills at the first storey, L'Aquila, Italy, earthquake of 2009

Fig. 3 Expulsion of the infill at the first storey, Emilia, Italy, earthquake of 2012

Fig. 4 Failure of the external layer of the infill, Emilia, Italy, earthquake of 2012

In general, both analytical models and experiments analyse the failure of masonry infills considering a single frame rather than the effects of infills on the behaviour of the whole structure. This approach is justified by the fact that infills are generally more vulnerable to seismic actions, in particular to the out-of-plane ones, than the reinforced concrete or steel elements, also due to the general lack of ductility of the masonry material.

Notwithstanding the apparent simplicity of the single frame scheme, the behaviour of the infill is conditioned by several parameters. This dependency is shown by observed real damages as well as a number of experiments that have been performed over the years in order to assess the load carrying capacity of masonry infill walls in different conditions; based on test results, some analytical models have been proposed (Pasca and Liberatore 2015). A state of the art for numerical modelling is given in Asteris *et al.* (2017). Some of these models are quite simple to be applied; on the other hand, they are strongly related to the experiment from which they were derived.

In this study, with the aim of identifying the main parameters involved, the out-of-plane behaviour of infills is investigated considering both damage observed after past earthquakes and response recorded during experimental tests. Observed responses after earthquakes are reported in section 2 and existing experimental tests are analysed and classified based on the influence of the different parameters involved in section 3. Afterword, different theoretical capacity models and code provisions for the prediction of the out-of-plane infills strength are discussed (section 4) and, finally, a comparison of analytical models developed for the assessment of the out-of-plane carrying capacity of masonry infills and code provisions is performed, focusing on models based on the arching theory. These simplified models and code provisions are applied to experimental tests in order to estimate the degree of approximation of such methods and their reliability.

2. Observations after earthquakes

Recent earthquakes have shown that the out-of-plane failure of infills may occur even for moderate intensity of the ground motion. Some observations reported after earthquakes of magnitude ranging between 5.2 and 6.3 are presented in this section. Similar, but more catastrophic effects, have been observed during stronger earthquakes (Saatcioglu *et al.* 2001, Miyamoto *et al.* 2008, Varum *et al.* 2017).

During the 2009 L'Aquila, Italy, earthquake (M=6.3), damage to reinforced concrete (RC) frames was often restricted to exterior infill walls and interior partitions, varying from small cracks to collapse (Braga *et al*. 2011, Decanini *et al*. 2012). Masonry infill panels failed primarily out-of-plane due to the lack of connections between the two wythes of the masonry panels and between the infill and the surrounding frame. Due to thermal insulation purposes, in many buildings the external wythe is placed partially outside the frame (Braga *et al*. 2011), thus increasing the seismic vulnerability of masonry infills, especially in the out-of-plane direction (Fig. 1). Similar conditions have been reported after the 2011 Simav, Turkey, earthquake (M=5.7) by Inel *et al*. (2013), who also highlighted the higher vulnerability of infill walls placed at the overhang portions of buildings due to the effect of the vertical acceleration, which loosen the contact between the wall and the

Fig. 5 Infill failures at the first storey, Central Italy earthquake of 2016

Fig. 6 Failure of the parapet at the top storey, Emilia, Italy, earthquake of 2012

surrounding beams.

A predominant role in the out-of-plane response is played by the type of connection between the masonry and the frame. As an example, during the 1999 Athens, Greece, earthquake (M=5.9), defective joints between the infill and the upper beam triggered the tilting of the panel (Decanini *et al*. 2005). Enhanced behaviour was observed when the connection was improved by inclining the top layer of brick at about 45°.

Often, failure of infills takes place at the lower stories of buildings (Fig. 2). Out-of-plane expulsion of the infills at the ground or first storey was observed after the 2012 Emilia (Italy) and 2016 Central Italy earthquakes (Fig. 3, Fig. 4 and Fig. 5). This circumstance can be ascribed to the interaction between in-plane and out-of-plane loads; namely, damage produced by in-plane shear forces, which are larger at the bottom storeys, increases the out-of-plane vulnerability of the infills. This indicates that out-of-plane damage cannot be merely related to out-of-plane floor accelerations, which are generally higher at the upper stories. By contrast, during the 2011 Lorca, Spain, earthquake (M=5.2), failures of roof parapets at upper floors were caused by inertia forces acting out-of-plane (Hermanns *et al*. 2014). The same failure mechanism was observed after the 2012 Emilia, Italy, earthquake (Fig. 6).

From these observations, no general rule can be given, having shown a wide variety of effects depending mainly on construction techniques and praxes.

3. Experimental evidences

As for experimental tests, out-of-plane structural

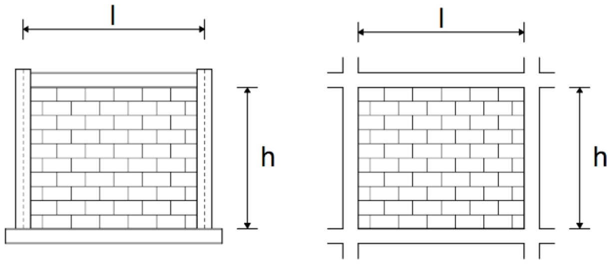


Fig. 7 Basic model

response of masonry walls has been investigated by different researchers. The published literature reports monotonic, cyclic and dynamic tests on masonry panels. In most cases, a single frame is considered (Fig. 7). The frame can be either steel or concrete; the stiffness of surrounding columns and beams is important for the response of the masonry infill as well as the supporting conditions. Those studies were aimed at estimating the influence of various parameters, e.g., the slenderness ratio (h/t), the panel thickness (t), the boundary conditions, the presence of prior in-plane damage. In this section, the influence of different factors affecting the out-of-plane response of infill panels is discussed with reference to experimental evidences.

3.1 Influence of the panel dimensions

One of the first experimental campaigns aimed at evaluating the out-of-plane resistance of masonry walls was performed at the Massachusetts Institute of Technology in 1954, as reported by McDowell *et al.* (1956). The experimental tests concerned 17 reinforced brick beams tested under fixed end conditions. Beams of different length (ranging from about 91 to 366 cm) and width (about 26 and 46 cm), made of double wythe masonry with total thickness of about 21 cm, were considered. The test beams were loaded statically at their third-points. One of the most important inference of the study was that the ultimate static load is inversely proportional to the square of the span length. On the contrary, the variation of stiffness and strength with the aspect ratio (h/l) did not show a clear trend. The experimental tests also put into evidence a behaviour that can be described as the formation of a rigid 3 hinges arch between the two supports. One way arching models take origin from these observations.

In the experimental study of Drysdale and Essawy (1988), walls with different span lengths (constant height and constant thickness) and different support conditions were considered. The walls, made of concrete blocks, were loaded in the out-of-plane by a uniformly distributed load applied by means of an air bag. Differently from what found by McDowell *et al.* (1956), the ultimate static load of walls supported at two opposite edges was found to vary inversely to the span length.

The influence of the wall thickness was studied by Dawe and Seah (1989). The test specimens consisted of unreinforced hollow concrete blocks laid up in a steel frame. Aspect ratio was the same for all walls whereas three different thickness values were considered. The ultimate strength was found to increase parabolically with increasing thickness. Nonetheless, other studies highlighted that this effect diminishes rapidly with increasing panel length.

The influence of the slenderness ratio was investigated also by Angel *et al.* (1994), by means of full-scale tests of reinforced concrete one-storey single-bay frames infilled with different clay brick and concrete block masonry infills. The slenderness ratio ranged between 9 and 34 and between 11 and 18 for clay brick and for concrete block infills, respectively. The panels were loaded out-of-plane by applying a monotonically increasing uniform pressure with an airbag. Main conclusions were that the strength is strongly affected by the slenderness ratio and depends on the compressive strength of the masonry, whereas it is not affected by its tensile strength.

Flanagan and Bennet (1993, 1999a) performed cyclic tests on one-bay one-storey steel frames infilled with single wythe walls (100 and 200 mm thick) and a double wythe walls (330 mm thick). Load-unload cycles of increasing pressure were applied to the infill with an airbag. The increment of thickness from 100 to 200 mm led to an increase of the peak load of more than three times.

More recently, the effect of the slenderness ratio was studied by Dazio (2008) and Tu *et al.* (2010) by means of shaking table tests. The experiments performed by Dazio, concerning full-scale walls with slenderness ratio ranging between 12 and 19.2, confirmed that reducing the wall slenderness increases the out-of-plane stability. However, boundary conditions were found to have a larger effect on the lateral stability than the slenderness of the wall. The tests conducted by Tu *et al.* (2010) on four full-scale single-storey structures included one bare frame, two frames with confined masonry panels with slenderness ratio of about 14 and 29, and one infilled frame with slenderness ratio of about 29. Every specimen was subjected to out-of-plane ground motions with growing intensity until severe damage. Wall thickness was found to have a significant influence on out-of-plane strength and stiffness.

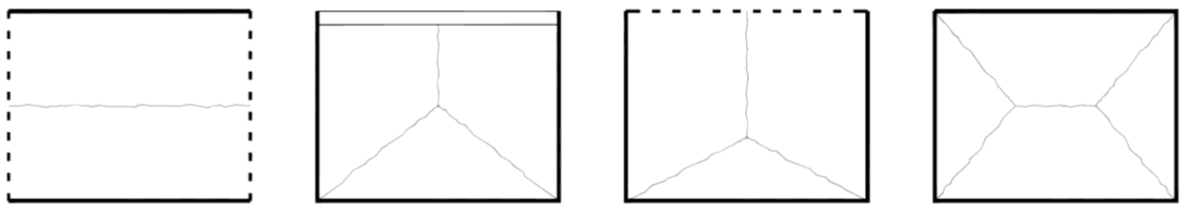
Finally, as reported by Komaraneni *et al.* (2011), the slenderness may also affect the distribution of acceleration along the height of the panel. The study concerned three moment-resisting frames infilled with solid clay bricks. The first two models had a slenderness ratio of 23 while for the third one it was equal to about 11. All the specimens maintained structural integrity and out-of-plane stability under design-level out-of-plane inertial forces, even after being damaged by in-plane drifts higher than 1%. However, the slender walls experienced larger out-of-plane displacements and collapsed for in-plane drift values smaller than those withstood by the third wall. Moreover, the specimens with slender walls experienced higher amplification of accelerations at mid-height. In contrast, the less slender wall experienced a nearly linear profile of acceleration response along the height, with the maximum value near the top.

3.2 Influence of the boundary conditions

Boundary conditions are undoubtedly one of the main factors affecting the out-of-plane strength and ductility of infill panels. Many researchers have highlighted how the presence of the arching effect can develop only if adequate support conditions are provided.

In the in-situ test performed by Fricke *et al.* (1992) on the ground floor of a five-storey steel-frame structure, the out-of-plane strength of the wall was found to be much greater than that predicted by theories not accounting the arching action of the wall within the steel frame. In this test the panel was mortared at all sides to the frame and the arching phenomena provided a noticeable increase of the predicted capacity.

Drysdale and Essawy (1988) considered four different boundary conditions: supports on four edges; on the bottom and the two sides; on the two sides and at the top and bottom. In order to enable the subsequent verification of analytical models, the edge conditions were not constructed to simulate the actual conditions of walls in buildings but were simple supports (rotations were allowed in the constrained edges). In one case, a vertical pre-compression loading was applied. In the specimen with four supported edges, the initial crack was a horizontal crack running along a bed joint near mid-height of the panel. The collapse mechanism was formed when additional cracks running approximately 45° from the horizontal cracks to the corners of the panels developed, like in Fig. 8(d). The formation of the horizontal crack corresponds to a sharp change in slope of the deflection curve (pressure against mid-point horizontal displacement curve). The application of a vertical pre-compression load resulted in higher first cracking and failure pressures. In the specimens with no support along the top, a vertical crack developed at the centre of the panel over the mid-height region of wall. The deflection curve indicated a very little increase in pressure between initial cracking and failure mechanism. The failure pressure was about 57% of that of the similar walls supported along all edges. Considering the panels with only two supported edges, deflection curves showed a nearly linear behaviour up to the failure, which is consistent with the sudden propagation of the first crack to form the collapse



(a) (b) (c) (d)

Fig. 8 Crack patterns with varying boundary conditions mechanism.

As highlighted by Dawe and Seah (1989), the support conditions influence noticeably the crack pattern (Fig. 8). The effect of lateral edge restraint on ultimate load has been specifically investigated considering specimen constructed with a 100 mm gap between the panel and column flanges. Failure mode indicated in Fig. 8(a) occurred in the initial stage of loading. When the panel was restrained from further slippage by the column flanges, failure mode in Fig. 8(b) developed and significant increase in load capacity was obtained.

In the experimental investigation carried out by Dafnis *et al.* (2002) different boundary conditions at the top of the wall were considered (vertical edges were not restrained). Shaking table tests of full-scale walls were performed considering the following situations at the top: joint completely filled with mortar (Fig. 8(b)), partially filled joint, joint with a horizontal gap of 3 mm (Fig. 8(a)), and unsupported top (Fig. 8(d)). No significant difference in the behaviour of the walls with the complete joint and that with the partially filled joint was found, whereas a horizontal gap in the upper mortar joint caused a clearly modified behaviour of the specimen; namely, the presence of an initial gap increases the relative displacements, leading to the tilting of the panel. The masonry wall with the unsupported top behaved as a cantilever, cracking at the horizontal joint within the first (at the bottom) brick layer.

The effect of boundary conditions at the top was investigated also by Dazio (2008) on a series of full-scale slender walls tested on shaking table. Different top boundary conditions were considered: simply supported (rotation and elongation of the top allowed), fixed condition (rotation and elongation at the top fully restrained), crosswise pre-stressed condition, and eccentrically pre-stressed condition with various values of the overburden loads. The study highlighted that the simply supported condition is not always the most critical situation. Boundary conditions that introduce an eccentric axial force to the wall can cause failure of the wall at considerably smaller shaking levels.

Tu *et al.* (2010) stressed the importance of boundary conditions in the out-of-plane resistance of unreinforced masonry, too. The shaking table tests conducted on four full-scale single-storey structures included one bare frame, two frames with confined masonry panels and one infilled frame. Each specimen was subjected to out-of-plane ground motions with growing intensity until severe damage. The confined masonry had toothed shear-keys inserted into columns and the top edge embedded in the beams, while infill-type panel was built after the frame and the gaps were filled with mortar. The confined masonry panels showed significant resistance to out-of-plane inertial forces due to the arching mechanism. Infill panels also showed arching at low motion intensity, but separated from the frames at higher intensity and collapsed under the inertial force caused by their self-weight.

3.3 Influence of combined in-plane and out-of-plane loads

During an earthquake, infills are called to simultaneously withstand in-plane and out-of-plane actions. Damage caused by in-plane forces, e.g., diagonal cracks in the wall or corner crushing, may accelerate the out-of-plane collapse. Contrarily, it was found that prior out-of-plane damage slightly affect the in-plane strength. The effect of combined in-plane and out-of-plane loads has been investigated in different studies.

In the experimental campaign described in Angel *et al.* (1994), the specimens were first loaded in-plane, up to twice the cracking drift. They were successively tested out-of-plane by applying a monotonically increasing uniform load on the surface of the infill with an airbag. In-plane cracking reduced the out-of-plane strength by a factor as high as two compared to a panel with no prior in-plane damage.

The experimental program reported by Henderson *et al.* (1993) consisted of the following large-scale tests: i) out-of-plane testing of a bare steel frame; ii) out-of-plane drift testing of an infilled frame; in this case the test structure was loaded out-of-plane by four quasi-static actuators, two on each column, to simulate seismic drift; iii) in-plane testing up to failure of the infilled frame previously damaged by out-of-plane drift in order to determine residual strength; iv) in-plane testing of an infill with no prior out-of-plane damage. For both out-of-plane and in-plane testing, reversed-cyclic quasi-static loads were applied. The load cycles produced hysteretic behaviour under out-of-plane loads, even though the load-deflection response was significantly more linear than in the in-plane direction, indicating less inelasticity and energy absorption compared to the in-plane response. The tests highlighted that prior out-of-plane damage reduces the in-plane initial stiffness. However, after the first few loading cycles, the two specimens (one with and one without prior out-of-plane damage) showed very similar load-deflection plots. It is then concluded that prior out-of-plane damage has little effect on the in-plane capacity of infills provided that the confinement by the frame is retained.

An extensive experimental study on the effect of in-plane and out-plane response of infill panels was conducted by Flanagan and Bennet (1993, 1999a, 1999b). Several tests were performed: in-plane, out-of-plane with air bag, and out-of-plane with imposed out-of-plane inter-storey drift. Different loading sequences were considered: combined in-plane and out-of-plane loading, before in-plane and then out-of-plane loads and vice versa. The main results are summarised below.

- In the out-of-plane tests with uniform load (air bag), early cracking in the mortar joints occurred, being followed by the development of membrane forces. The cracks divided the panels into separate portions; when these portions of the wall moved out of the plane and rotated about their boundaries, arching effect developed until failure. Usually, vertical arching took place until failure of the top and bottom course tiles. After failure of these courses, horizontal arching developed allowing the panels to maintain stability.
- In the tests with in-plane loading followed by out-of-plane uniform load (air bag), the specimen was loaded cyclically in-plane up to a drift of about 1% (loading was stopped before corner crushing began). The peak capacity

of the specimen was 85% of that loaded only out-of-plane. However, the peak capacity occurred at a mid-panel displacement 68% greater than that of the control specimen. Considerable softening of the panel was evident when comparing the response with that of the specimen without prior in-plane damage.

• In the case of out-of-plane drift followed by in-plane loading, the panels tested under horizontal out-of-plane drift showed significant cracking resulting in a 15% decrease in the first out-of-plane frequency. Little relative movement was observed between the infill and the bounding frame. The infill panels constructed tightly to the frame, but without ties or other reinforces, remained stable when subjected to cyclic out-of-plane drift displacements of 1%. The specimens were then loaded in-plane up to failure. The results were compared to that of the frame loaded only in-plane showing that in-plane strength and stiffness degradation resulting from the out-of-plane drift loading was limited.

• In the combined out-of-plane (air bag) and in-plane loading tests, a reduction of in-plane strength of about 40% with respect to the specimen loaded only in-plane was found. However, even though the panel was noticeably damaged, it remained stable and the in-plane resistance was still 4-5 times that of the bare frame at the end of the test.

The effect of previous in-plane damage on the out-of-plane strength is highlighted also by Calvi and Bolognini (2001). The experimental tests concerned one-bay, one-storey full-scale reinforced concrete frames infilled with weak masonry panels. Out-of-plane tests were performed for different levels of in-plane drifts, i.e., 0.1, 0.2, 0.3, 0.4% for serviceability, 1.2% for heavy damage. Both the strength and the secant stiffness in the out-of-plane direction decreased noticeably with increasing in-plane drift.

In the experimental study of Komaraneni *et al*. (2011), the specimens were loaded in-plane by displacement-controlled slow cycles of gradually increasing storey drift up to 2.2%. The specimens were then subjected to simulated earthquake ground motions generated by a shaking table in the out-of-plane direction. All the specimens maintained out-of-plane stability under design-level out-of-plane inertial forces, even when previously damaged by in-plane drifts higher than 1%.

Pereira *et al.* (2011) performed experimental tests on RC frames infilled with horizontal hollow brick masonry. In-plane tests were first performed applying cyclic horizontal displacements up to a drift equal to 0.5%. Previous in-plane damage introduced cracking at the interface between masonry and RC surrounding frame thus modifying the support conditions of the infill. Failure modes typical of cantilever beams occurred.

3.4 Influence of reinforcing elements

The presence of reinforcing elements can produce a positive effect, as highlighted by Dawe and Seah (1989). They found that bed joint reinforcement placed at alternate courses allowed considerable ductility and avoided the sudden failure of the specimen. In specimens with a gap between the top edge of the panel and the beam (see Fig. 8(c)), the horizontal span was mobilised in resisting the applied loads; in this case the inclusion of joint reinforcement resulted in a higher first crack load. In the other cases (i.e., without gap) the inclusion of joint reinforcement had little effect on first crack load.

Al-Chaar *et al.* (1994) and Angel *et al.* (1994) investigated the effect of different repairing techniques. According to what reported in Angel *et al.* (1994), some specimens, previously loaded in-plane and out-of-plane, were repaired in order to increase the infill resistance and tested again in the out-of-plane direction. The repairing method consisted in applying a ferrocement coating to one or both sides of the masonry panel, thus increasing the out-of-plane strength of damaged infills by a factor as high as five, regardless of the extent of pre-existing damage. In the tests reported by Al-Chaar *et al.* (1994), an infilled frame was subjected to a series of out-of-plane ground motions until severe cracking. Afterward, the masonry infill was repaired on both sides by a steel wire mesh, covered by a ferrocement coating. The steel mesh was not anchored to the infill nor to the frame. This specimen was subjected to a new series of increasing out-of-plane ground motions until severe damage occurred; the obtained strength augmented of about 70%.

In the tests performed by Calvi and Bolognini (2001), two reinforcement conditions were considered: reinforcement in the mortar layers at 60 cm distance and light wire meshes in the external plaster. In all cases, there was no continuity between the steel used for reinforcing the panels and the frame reinforcement. Nevertheless, the effects resulting from the insertion of reinforcement in the mortar layers, and to a larger extent of the external mesh, were strongly beneficial.

The experimental program of Komaraneni *et al.* (2011) included the testing of a specimen with interior grid elements, which divided masonry into four subpanels. These elements helped in reducing the out-of-plane deflection and greatly improved the in-plane response and the overall energy dissipation capacity. Out-of-plane failure of the masonry was delayed and the wall could safely sustain in-plane drifts up to 2.2%.

In the study performed by Pereira *et al.* (2011), three types of infill material have been considered: unreinforced masonry, masonry with bed joint reinforcement, and masonry with reinforcement in the plaster. Both reinforcing methods produced a significant increase in stiffness and strength, the latter presenting the highest displacement capacity.

In Walsh *et al.* (2015), the effects of different cavity ties is studied on two real buildings by means of airbags. The use of tie retrofits with proper spacing and adequate compressive and shear stiffness resulted in a significant improvement of the out-of-plane capacity of cavity infill walls.

3.5 Influence of openings in the panel

As observed by many researchers, the out-of-plane strength of infills strongly depends on the arching effect, which may be affected by the presence of an opening in the wall. Moreover, the openings reduce the in-plane stiffness and strength (Decanini *et al.* 2014), thus increasing the in-plane damage, and, in turn, reducing the out-of-plane resistance.

The influence of openings on the out-of-plane resistance has been investigated by Dawe and Seah (1989) and by Dafnis *et al.* (2002). In the former study, a panel (3.6×2.8 m) was perforated by a 1.6×1.2 m (about 19% of the wall area) central opening. The presence of the opening reduced noticeably the ductility but no significant decrease of the ultimate load was observed. In the experimental investigation carried out by Dafnis *et al.* (2002), a small opening (1.0×0.8 m over a 3.0×3.5 wall, that is about 7% of the wall area) was located at the top of the wall. The opening did not cause significant modifications in the dynamic behaviour when compared to the walls without the opening and no local effects at the corners of the opening were observed.

4. Theoretical predicting models and code provisions

Several predicting analytical models have been developed in the past years. Most of them are based on rigid body mechanisms, while more complex modelling is performed when numerical or iterative solutions or the application of finite element (FE) methods are applied. Starting from the first type of models, some codes have introduced provisions for the evaluation of the ultimate load (see section 4.1.3)

In the following subsection (4.1), a state of the art review on available analytical models for the assessment of the out-of-plane response of masonry infills is given; some of them, in particular those that provide close form solutions, are discussed in more details and the resulting expressions will be applied to several cases extracted from experimental campaigns in section 5.

Main parameters introduced in the sections are:

- h panel height
- l panel length
- t panel thickness
- h/t slenderness ratio
- f'_m masonry compressive strength
- E_m modulus of elasticity of the masonry
- E modulus of elasticity of the frame material
- G shear modulus of the frame material
- I moment of inertia of beams ($_b$) and columns ($_c$)
- J torsional constant of beams ($_b$) and columns ($_c$)

4.1 Analytical models

Many models, even recently analysed and adopted by current codes, are based on rigid body mechanisms, either with or without the description of the arching behaviour (Fig. 9). In the rigid-body mechanism that considers the wall as a whole with a hinge at the bottom of the panel (Fig. 9(a)), out-of-plane stability is verified by the equilibrium

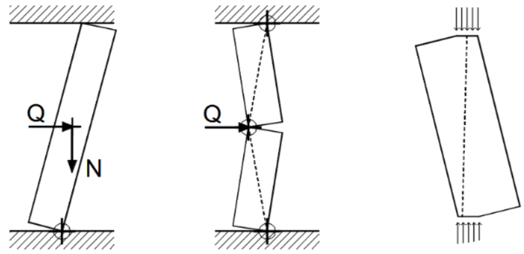


Fig. 9 Rigid body models: (a) without arching effect; (b-c) with arching effect

condition between the stabilizing action (weight of the wall) and the overturning action (seismic load) (Sorrentino *et al.* 2003, 2008a, 2008b, 2014, Braga *et al.* 2011). This model is consistent with panels having a weak vertical restraint at the top.

Field observations have suggested that the failure may occur due to local cracks at the centre of the panel, as a consequence of the presence of the surrounding frame. To take this phenomenon into account, the formation of an intermediate hinge is introduced. The static scheme is therefore defined by assuming an arching behaviour (Fig. 9(b)); in this case the collapse is related to a three hinges mechanism, which is usually activated along the shorter dimension. This model is consistent with panels restrained by the surrounding frame.

4.1.1 One-way arching models

One of the first models formulated to predict the lateral strength of one-way spanning brickwork beams with rigid supports due to arching was proposed by McDowell *et al.* (1956). The wall is modelled as an ideal beam constrained between rigid supports on the two edges and the masonry material is considered to be unable to sustain tensile stress, with an elastic-perfectly-plastic behaviour under compression. According to the model, cracks develop on the tension side at the centre and edges of the beam and, after this phase, the two portions of the beam are supposed to behave as rigid bodies, rotating around one edge and the centre (Fig. 9(b)). Further resistance is given by the crushing of the material at the hinges location (Fig. 9(c)).

Ultimate capacity q determined by McDowell *et al.* (1956) can be expressed as

$$q = \frac{f'_m}{2(h/t)^2}\,\gamma \tag{1}$$

where q is the uniform pressure which causes the out-of-plane collapse and γ is a dimensionless parameter (Table 1 in McDowell *et al.* 1956) which depends on: the slenderness ratio, the strain associated with the masonry compressive strength, the deflection at the centre of the wall and the stress distribution along the contact area (Fig. 9(c)).

Comparisons of this model with experimental results (Angel *et al.* 1994) have shown that it overestimates the stiffness and the strength, notwithstanding the fact that the test panel spans in two directions rather than one as assumed in the analytical model. This difference was attributed to the pre-cracked condition of the infill.

Within the models based on the arching effect, Anderson (1984) proposed a theory for predicting the behaviour of one-way spanning unreinforced masonry walls subjected to out-of-plane loading that includes the effects of shrinkage, initial boundary gaps, and abutment stiffness; different expressions are given for the ultimate transverse lateral load.

Button and Mayes (1992) have developed a structural component model, intended to directly predict the global response (maximum moments, forces, and deflections) of a wall. The model consists of a number of inelastic elements arranged vertically to represent the wall. Each element is a series combination of elastic beams with inelastic hinges at each end: once the first hinge enters in the inelastic range, for numerical stability, plasticization in the other hinges is not permitted. Provision is made for the formation of a base hinge in a fixed-base wall. The model was numerically implemented and no close form expression was given.

In Angel *et al.* (1994), Abrams *et al.* (1996), the authors proposed the so-called compressive strut method for infill walls surrounded by concrete frames, supposed to be stiff. The model is based on the one-way arching mechanism. After the formation of a given cracking pattern at mid-span, the wall is divided into segments; as the wall segments rotate, axial compressive struts develop in those segments. The contact width between two wall segments depends on the wall geometry and the maximum strain of the masonry. The out-of-plane strength is calculated by the equilibrium of horizontal forces between the acting pressure and the horizontal component of the compressive struts. Failure of the walls is related to crushing of the masonry of a wall segment; accordingly, the arching mechanism takes place when the rotation is small enough so that an internal compression strut can develop; as the load increases, the panel is supposed to "snap through"; while under static loads, snap through will result in collapse, under dynamic load, the collapse of the panel will depend on the maximum displacement value. Furthermore, the model considers the effect of in-plane damage on the out-of-plane resistance. The following expression for the ultimate capacity q derives from both equilibrium considerations and experimental results

$$q = \frac{2f'_m}{(h/t)}\,\lambda R_1 R_2 \tag{2}$$

In Eq. (2), λ is a term that includes the effect of the maximum masonry compressive stress, the maximum strain and the ratio between width and height of the panel, all quantities being related to the slenderness ratio, while R_1 and R_2 are reduction factors. The reduction factor, R_1 accounts for the magnitude of prior in-plane damage and is given by

$$R_1 = [1.08 - 0.015(h/t) - 0.00049(h/t)^2 + 0.000013(h/t)^3]^{\frac{\Delta}{2\Delta_{\mathrm{crack}}}} \tag{3}$$

where Δ is the in-plane maximum horizontal displacement and Δ_{crack} is the in-plane displacement at which the first crack is expected to occur. The reduction factor R_2 accounts for the flexibility of the confining frame. If an infill panel is confined within a frame having neighbouring panels in each direction, then $R_2 = 1$. Otherwise the following expressions apply

$$\begin{aligned} &R_2 = 0.357 + 2.49\ \times 10^{-14}\mathrm{EI} \\ &\text{for}\quad 5.74 \times 10^{12}\ \le EI \le 25.83\ \times 10^{12}\ \ \mathrm{N\,mm^2} \\ &R_2 = 1 \qquad \text{for}\quad EI > 25.83\ \times 10^{12}\ \ \mathrm{N\,mm^2} \end{aligned} \tag{4}$$

where *EI* is, in this case, the flexural rigidity of the smallest frame member at the side where a neighbouring panel is missing.

Eq. (2) is valid when the out-of-plane strength is governed by arching of the panel; such a mechanism takes place when the slenderness of the panel is smaller than the following critical value

$$\left(\frac{h}{t}\right)_{cr} = 0.981\sqrt{\frac{2}{\varepsilon_{cu}}} \tag{5}$$

where ε_{cu} is the ultimate compressive strain. When the slenderness of the panel is greater than the critical one, the snap through occurs before the attainment of ε_{cu}.

Morandi *et al.* (2013) proposed a formula that takes into account the contribution of vertical reinforcement

$$q = \left(0.72\frac{f'_m}{(h/t)^2} + 7.2\ \frac{t}{l\,h^2}\ A_s f_y\right)\beta_a \tag{6}$$

where the first term represents the out-of-plane strength of the infill based on arching mechanism following the assumptions of Eurocode 6 (see section 4.1.3), and the second term is the resistance due to vertical reinforcement, being A_s the total cross sectional area of reinforcement and f_y its yield strength. Previous in-plane damage is taken into account by means of a reduction coefficient, β_a, expressed as a function of the expected in-plane drift demand and depending on the reinforcement in the infill considering the following situations: unreinforced masonry, masonry with reinforcement in the bed joints, and with mesh reinforcement in the plaster.

4.1.2 Two-way arching models

As shown in section 3, when the infill is restrained at four edges, a two-way arching action develops. Therefore, there is the necessity to include this phenomenon in the evaluation of the load carrying capacity of the infill.

In the elastic field, classical solutions for the elastic plate by Timoshenko (1959) has been used to consider the two-way bending of an infill. Failure is assumed to occur when the tensile stress reaches the tensile strength of the masonry. The limit of this solution is the elastic behaviour with no post-cracking analysis, thus failure is assumed to occur in correspondence with the stress value at first crack, without taking into account the flexibility of a cracked infill (Angel *et al*. 1994).

Approaches based on the modified yield-line analysis have been developed by Hendry (1973) and Haseltine *et al.* (1977). The yield-line analysis consists in defining a kinematically admissible mechanism (yield-line mechanism) and calculating the limit load by equating the internal and external works. In the equations proposed by these authors, the out-of-plane strength is expressed as a function of the flexural tensile strength normal to the bed joints.

The two-way arching action was investigated by Dawe and Seah (1989), who developed a strength model based on virtual work concepts, modifying the conventional yield-line method. Specifically, the wall is divided into several horizontal and vertical strips (see Fig. 10); flexural resisting moments between strip segments are then calculated as a function of the compressive strut forces developed by an arching action. The flexibility of the steel frame is explicitly considered. A finite element elastic analysis is performed in order to predict the first crack, with four failure criteria: i) debonding along bed joints, ii) simultaneous bond failure through head joints and units, iii) a stepped failure through head and bed joints, and iv) splitting directly through the units. Afterwards, a modified yield-line technique is used to predict the post-cracking behaviour and the ultimate infill capacity; the deflected configuration is defined by a given lateral deflection at a convenient location on the selected yield-line pattern, under the assumption of rigid plate rotation within yield-line boundaries. The corresponding applied load is found with an iterative technique, similar to a successive displacement technique used for solving large systems of equations.

As it can be expected, the model by Dawe and Seah (1989) produces a stiffer and stronger response than that from McDowell *et al.* (1956) because two-way action is considered rather than one-way action. However, since edge flexibility is introduced, the differences between the two models are reduced.

Based on this method, Dawe and Seah (1989) performed a parametric study to evaluate the effect on ultimate load q of several parameters and proposed the following empirical relations, for panels supported on four sides

$$q = 4.5(f'_m)^{0.75} t^2 (\alpha / l^{2.5} + \beta / h^{2.5}) \tag{7}$$

and panels supported on three sides and free at the top

$$q = 4.5\,(f'_m)^{0.75} t^2\, \alpha / l^{2.5} \tag{8}$$

where quantities are given in kPA and mm, α and β are parameters which depend on the bending (EI) and torsional (GJ) stiffness of the columns and of the beams, respectively

$$\alpha = \frac{1}{h}(EI_c h^2 + GJ_c th)^{0.25} \tag{9}$$

$$\beta = \frac{1}{l}(EI_b l^2 + GJ_b tl)^{0.25} \tag{10}$$

with $\alpha \leq 50$ for panels supported on four sides and $\alpha \leq 75$ for panels supported on three sides and free at the top, while $\beta \leq 50$ in both cases. The above equations were derived for hollow concrete block panels within steel frames having pinned joints.

Eq. (7) was later modified by Flanagan and Bennett (1999c) based on 36 experimental tests on steel and concrete frames infilled with clay and concrete masonry panels. The numerical constant 4.5 was modified into 4.1 and the expressions for parameters α and β were simplified by eliminating the terms of torsional stiffness of the frame members.

In order to include two-way arching action, Bashandy *et al.* (1995) extended the analytical method developed by McDowell *et al.* (1956). The panel is divided into vertical and horizontal strip segments experiencing the crack pattern shown in Fig. 10. All horizontal strips and some vertical strips will not experience the maximum moment, and the maximum out-of-plane deflection will be governed by the crushing of masonry in the central vertical strips.

The total force resistance, Q, is calculated assuming an equivalent rectangular stress pattern in the contact area at hinges location and it is obtained by the sum of the forces resisted by all the horizontal and vertical strips according to the following expression

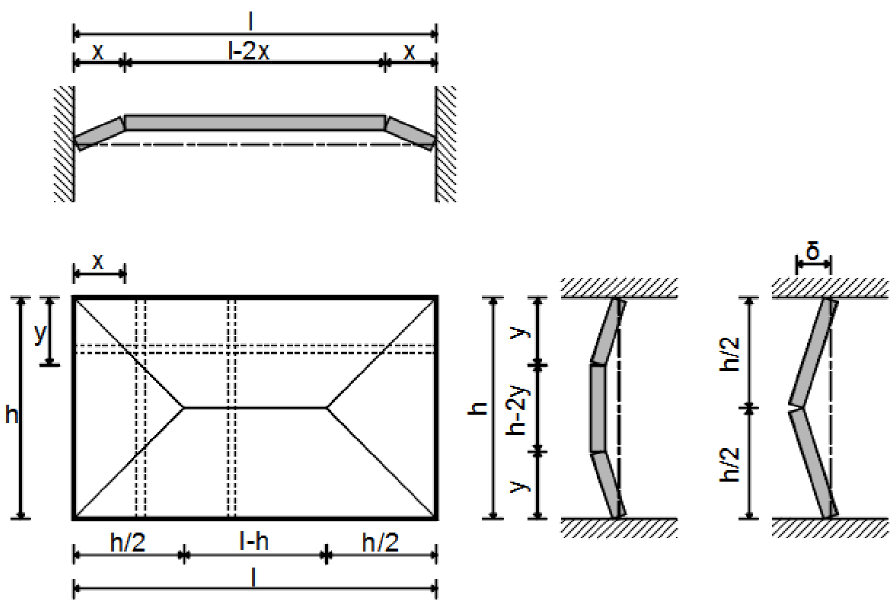


Fig. 10 Crack pattern in infill wall and strips model by Dawe and Seah (1989)

$$Q = 8\frac{M_{yv}}{h}(\mathrm{l}-\mathrm{h}) + 8M_{yv}\ln(2) + 8\frac{M_{yh}}{h}\left(\frac{x_{yv}}{x_{yh}}\right) ln\left(\frac{l}{l-h/2}\right) l \tag{11}$$

where x_{yv} and x_{yh} are

$$x_{yv} = \frac{t\, f_m'}{E_m\left(1 - h/\left(2\sqrt{(h/2)^2 + t^2}\right)\right)} \tag{12}$$

$$x_{yh} = \frac{t\, f_m'}{E_m\left(1 - l/\left(2\sqrt{(l/2)^2 + t^2}\right)\right)} \tag{13}$$

M_{yv} and M_{yh} are obtained by substituting the values of x_{yv} and x_{yh}, respectively, in Equation (14) in lieu of x_y

$$M_y = \frac{0.85 f_m'}{4}\left(t - x_y\right)^2 \tag{14}$$

In Eq. (11), the first term is the force resisted by the central vertical strips, the second term is the force resisted by the lateral vertical strips and the third term is the force resisted by the horizontal strips. In the case in which the panel is not restrained at each side, only the contribution of the strips in which the arching action can develop should be considered in the calculation of the total resistance.

4.2 Design codes provisions

Infill walls subjected to out-of-plane loads are addressed in International and European design provisions, giving either expressions for the evaluation of the design load carrying capacity or recommendations to avoid damages.

In FEMA 306 (1998) and NZSEE (2006) recommendations, the equation proposed by Angel *et al.* (1994) for the assessment of the out-of-plane infill strength (Eq. (2)) is directly used.

According to FEMA 356 (2000), unreinforced infill panels with slenderness ratios less than specified values and meeting the requirements for arching action (i.e., panel in full contact with the surrounding frame elements, frame components with sufficient stiffness and strength to resist thrusts from arching actions, etc.) need not to be verified under out-of-plane seismic forces. Limit values of the slenderness ratio vary from 8 to 16 depending on the performance level and on the seismic zone. If the slenderness limit is not accomplished but requirements for arching action are met, then the lower bound out-of-plane strength, q, of an infill panel should be assessed according to the following expression

$$q = \frac{0.7 f_m' \lambda_2}{h/t} \tag{15}$$

where f_m' represents the lower bound of masonry compressive strength, λ_2 is a slenderness parameter given in a specific table (section 7.5.3.2 of FEMA 356, 2000) for different values of the slenderness ratio. This expression is a modification of Eq. (2), where the numerical constant 2 is changed to 0.7 in Eq. (15) and the parameter λ_2 in Eq. (15) is lower than λ in Eq. (2). These modifications are due to the fact that the FEMA 356 (2000) expression provides a lower bound prediction of out-of-plane strength. When arching action is not considered, the lower bound strength of the infill panel q should be evaluated as a function of the lower bound masonry flexural tension strength.

As far as European codes are concerned, the problem of walls arching between supports is dealt with in Eurocode 6 (2005), but without any specific reference to infills. It is suggested that, in case the wall is built solidly between supports capable of resisting an arch thrust that may develop in horizontal or vertical direction, the analysis may be based on a three-pin arch. The design lateral strength, q_d, is then given by

$$q_d = f_d\left(\frac{t}{l_a}\right)^2 \tag{16}$$

where f_d is the design compressive strength of the masonry in the direction of the arch thrust, and l_a is the length or the height of the wall between supports capable of resisting the arch thrust. This expression is valid provided that the slenderness ratio (l_a /t) does not exceed 20.

Furthermore, Eurocode 8 (2004) requires that out-of-plane collapse of slender masonry panels should be avoided by means of specific measures. Particular attention is required for masonry panels with slenderness ratio greater than 15. Examples of measures which are suggested for the improvement of both in-plane and out-of-plane behaviour include: light wire meshes, wall ties fixed to the columns, wind-posts and concrete belts.

In the Italian specifications (NTC 2009), the use of light wire meshes with wires spaced no more than 500 mm out anchored on both sides of the masonry panel and connected to the frame elements or the adoption of reinforcing steel bars in the bed joints are suggested. If such measures are taken, then the verification under seismic actions perpendicular to the infill may be neglected, otherwise the effects of the seismic force acting in the out-of-plane direction should be assessed. No capacity models are suggested in both Eurocode 8 (2004) and current Italian code (NTC 2009).

4.3 Numerical approaches

With reference to numerical approaches, two different typologies of methods have been developed. The first one, which recurs to one or multiple diagonal struts to model the panel, is particularly suitable to take into account the in-plane/out-of-plane interaction in the analyses of multi-storey buildings. In the second approach, the masonry is described in greater detail (e.g., Liberatore *et al.* 2008) by means of finite or discrete elements. This second approach is generally used for the assessment of the in-plane/out-of-plane response of a single infill panel.

In the equivalent strut method, the infill is represented by several struts (Hashemi and Mosalam 2006, 2007, Furtado *et al.* 2016, Shing *et al.* 2016), one diagonal strut (Kadysiewski and Mosalam 2009, Mosalam and Günay 2015) or two diagonal struts (Asteris *et al.* 2017).

Hashemi and Mosalam (2006) proposed a model composed of 8 no-tension struts connecting the beam-column joints to two central nodes, linked to one another by

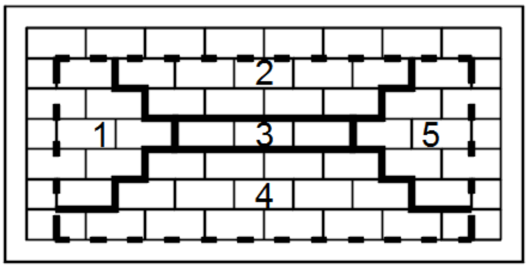


Fig. 11 Idealised cracking pattern used for predicting maximum pressure (Varela-Rivera *et al.* 2011)

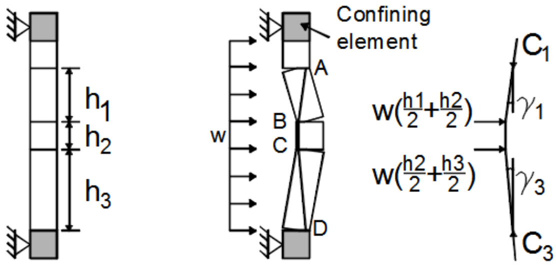


Fig. 12 Equilibrium of horizontal forces (Varela-Rivera *et al.* 2011)

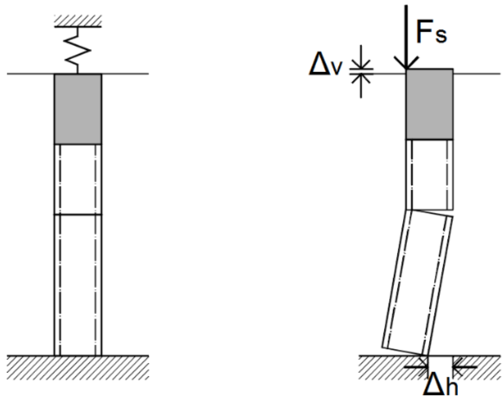


Fig. 13 Spring strut model: (a) at rest; (b) after deformation

a rigid element resisting only in tension. The in-plane/out-of-plane interaction is taken into account by defining a failure surface and an element removal algorithm is used, consisting in removing from the building model an infill once it is collapsed.

In the model proposed by Furtado *et al.* (2016), 4 diagonal rigid struts link the beam-column joints with a central non-linear element, where the out-of-plane mass is lumped. The in-plane and out-of-plane components are modelled independently even though an element removal algorithm, based on a linear collapse surface, is used under biaxial loadings.

A single diagonal strut, reacting both in tension and compression is presented in Kadysiewski and Mosalam (2009), Mosalm and Günay (2015). In the model, each infill wall is represented by a single diagonal, composed of two beam elements connected at a midpoint node, where the out-of-plane mass is lumped. The beam section is modelled through fibre elements so that the beam acts as truss in the in-plane direction and as a flexural element in the out-of-plane direction. In this way, the in-plane/out-of-plane interaction is directly considered.

In Shing *et al.* (2016), Asteris *et al.* (2017) four and two, respectively, diagonal no-tension elements are used to represent the infill. They are divided at the midspan by a joint, where the out-of-plane mass is concentrated. The diagonal cross sections are modelled by fibres so that the cracking of the cross-sections and the arching mechanism are automatically taken into account.

The main difficulty of fibre element methods lies on the definition of the beam cross-section geometry and on the characterization of the material mechanical properties, which must be fixed to satisfy a certain interaction domain.

Different methods are those which represent the panel by means of finite or discrete elements. For example, Varela-Rivera *et al.* (2011) used commercial code SAP to test different models for the prediction of the out-of-plane strength, namely, the yield line method, the failure line method, and the compressive strut method. In all cases, the wall is divided in segments as in Fig. 11. In the first method, cracking moments used to calculate internal work are those used for predicting cracking pressures, while for the failure line method the two central horizontal cracks are not considered in the internal work. In the third method, maximum pressure is calculated by the equilibrium of horizontal forces between this pressure and the horizontal component of the compressive struts (Fig. 12).

In Varela-Rivera *et al.* (2012), the procedure has then been modified to consider the actual stiffness of the confining elements and the ‘‘snap through’’ failure mechanism observed from tests, introducing a spring located at the top of the wall (spring-strut model in Fig. 13). By using an iterative procedure, the model predicts the out-of-plane strength of the walls and the two failure types observed in the laboratory: crushing of masonry and snap-through. The models based on the yield and failure line methods underestimate the out-of-plane strength of the walls studied. The model based on the compressive strut method overestimates the out-of-plane strength of those walls.

In the field of numerical analysis, Tasnimi and Zomorody (2010) implemented a FE model to evaluate the out-of-plane capacity of an infill wall surrounded by an RC frame and compared these results with experimental tests. The model is three dimensional and the columns and the upper beam are modelled by 40 degrees of freedom 8-node 3D curved shell elements, and the interface between the infill wall and the surrounding frame is modelled by interface 6-node 3D curved shell elements; mortar and unit-mortar interface are smeared out. The compressive and tensile behaviour of masonry is defined with the concrete model in the FE software DIANA. In order to avoid an overestimate of the stiffness of the infill wall (Flanagan and Bennett 1993), the infill wall is assumed to be pinned to the frame members such that the out-of-plane sliding of the infill wall is prevented, but relative rotations around the edges between frame member and infill wall are allowed. Two models are considered: in the first model, the infill wall is constrained at the four sides, thus representing the two-way arching action; in the other model, the infill wall is constrained only to the top beam and bottom support with no connection to the columns, therefore it represents the one-way arching action. According to the authors (Tasnimi and Zomorody 2010), the capacity of the model with side releases represents better agreement with the empirical approaches than the first model. As a result of the analyses, a strong interaction between bidirectional loading and out-of-plane loading is suggested, that can significantly reduce the in-plane capacity and the amount of this reduction is such that it should not be neglected for analysis and design purposes. Furthermore, by increasing exerted out-of-plane pressure on the infill panel, in-plane yielding and maximum capacity of the infilled frame reduce approximately by constant rate.

In Liberatore *et al.* (2016) the LS-DYNA software package (2013), used within an ANSYS environment, is used to test different modelling strategies. Firstly, a combined finite and discrete modelling approach is used. Units are modelled as linear elastic 8-node solid elements with a single integration point. Mortar is not explicitly considered in the model; contact interfaces are used to transmit both compressive and tensile forces instead. Moreover, a tangential motion with friction sliding is permitted. Secondly, a FE model resorting to a smeared-crack approach is implemented. In these cases, the contact surfaces are used only at the interface between the masonry panel and the surrounding structure. It is concluded that the smeared-crack approach is suitable to reproduce experimental results in terms of stiffness and strength, whereas the finite-discrete method is not able to provide the maximum strength due to local stress increment in the contact interfaces.

5. Comparative assessment of different analytical models

The range of validity of the aforementioned analytical expressions for the evaluation of the ultimate out-of-plane capacity of infills is tested against a data-set of experimental test results. Specifically, the out-of-plane capacity has been estimated using the equations suggested by: Dawe and Seah (1989), Angel *et al*. (1994), Bashandy *et al*. (1995), Flanagan and Bennett (1999c), Eurocode 6 (CEN 2005) and compared with experimental results. For the sake of conciseness, the above-mentioned models will be cited in tables and figures as: DS, A *et al*, B *et al*, FB, EC6.

Twenty-two experimental tests available in the literature have been selected so as to represent different types of frames and infills, namely: i) steel frames infilled with hollow concrete blocks (Dawe and Seah 1989); ii) hollow brick masonry supported on the top and the bottom by rigid reinforced concrete elements (Modena and da Porto 2005), and iii) hollow concrete blocks confined masonry with reinforced concrete confining elements (Varela-Rivera *et al*. 2012).

In Table 1 to Table 3, the measured peak loads and the comparison with predicted values are reported for the three groups of experimental tests. The average values of the ratio between predicted and experimental ultimate loads are also shown. Values predicted by EC6 equation are estimated by considering the mean compressive strength of masonry in lieu of the design strength.

Table 1 Comparison between predicted and experimental out-of-plane capacities, Dawe and Seah (1989) tests.

Spec.		DS*	A *et al**	B *et al**	FB*	EC6*
	Exp. (kN)	Pred./ exp.	Pred./ exp.	Pred./ exp.	Pred./ exp.	Pred./ exp.
WE1	22.30	1.74	3.90	8.01	1.59	6.30
WE2	19.20	1.90	4.18	8.62	1.73	6.74
WE4	11.20	1.51	2.54	3.46	1.38	5.07
WE5	7.80	0.82	0.70	0.48	0.75	2.68
WE6	10.60	0.95	2.39	5.84	0.86	5.86
WE7	14.70	0.70	1.78	4.07	0.64	4.36
WE8	13.40	1.45	2.56	3.37	1.32	9.42
mean		1.30	2.58	4.83	1.18	5.77
COV		0.34	0.43	0.55	0.34	0.34

*DS=Dawe and Seah (1989); A *et al*=Angel *et al.* (1994), B *et al*=Bashandy *et al.* (1995), FB=Flanagan and Bennett (1999c); EC6=Eurocode 6 (CEN 2005).

Dawe and Seah tested nine steel frames infilled with vertical hollow concrete blocks. The specimens were loaded by a uniform pressure normal to the panel surface applied by means of air bags. Two specimens, i.e. WE3 and WE9, are not considered in the comparison because in WE3 the frame is infilled with a dry-stack masonry and in WE9 a window opening is present. In specimens WE6 and WE7 a 20 mm gap at the top beam to panel interface was provided. For these tests, in order to take into account the gap, the analytical strength was determined with Dawe and Seah's and Flanagan and Bennett's methods by setting β to zero, with Angels *et al.*'s method by setting R_1 to zero, h equal to the length of the panel and calculating R_2 for horizontal arching only, and with the Bashandy *et al.*'s model by eliminating the contribution of the vertical strips. The measured strength and the comparison with predicted values are reported in Table 1.

The method which better predicts the observed experimental values is the Flanagan and Bennett's one. In this case the mean of the ratios between predicted and experimental values is 1.18. As expected, the Dawe and Seah equation predicts a slightly higher strength than that given by Flanagan and Bennett, but is still giving quite good results. The models developed by Angel *et al*. and Bashandy *et al*. overestimate the actual resistance for all specimens except WE5, which has a thickness much lower than the other specimens. The EC6 equation overestimates the experimental strength noticeably. It has to be pointed out that Dawe and Seah's test setup (Dawe and Seah 1989) is quite different from the others under investigation; first, the frame is in steel; second, different boundary conditions are tested, i.e., all sides mortared to frame members in specimens WE1 to WE4, vertical edges restrained from slipping in WE5, vertical edges restrained from slipping and a 20 mm top gap in WE6 and WE7, all sides restrained from slipping for WE8. These type of restrains are not always correctly modelled by the models developed by

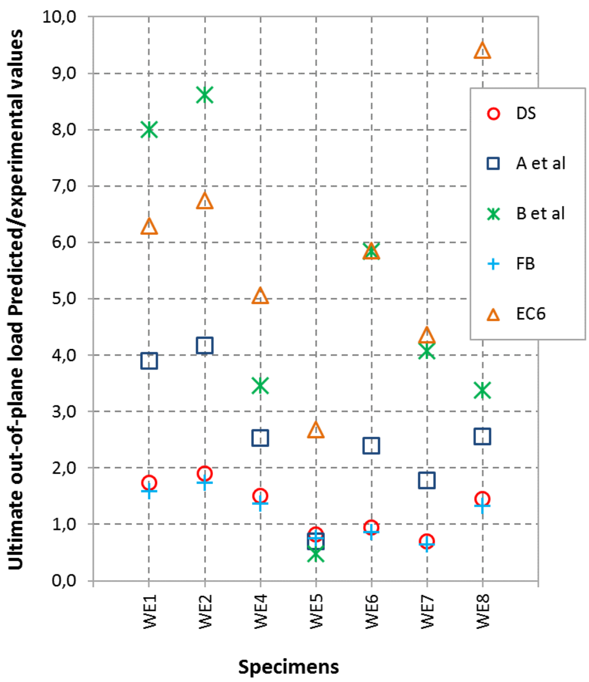


Fig. 14 Predicted/experimental ratio of the ultimate out-of-plane load according to different models for the Dawe and Seah's tests. Boundary conditions: WE1 to WE4 all sides mortared to frame members, WE5 vertical edges restrained from slipping, WE6 and WE7 vertical edges restrained from slipping and a 20 mm top gap, WE8 all sides restrained from slipping.

Angel *et al*. and Bashandy *et al*. and by EC6, as shown in Fig. 14, where the ultimate load predicted/experimental ratio is shown for the models considered. As already observed, the results are fairly well reproduced by the Dawe and Seah's and Flanagan and Bennett's models, whereas the other models overestimate noticeably the actual strength. This outcome suggests the opportunity to include the frame deformability in the case of pinned steel frames, where the beam-column joints are somewhat deformable.

The experimental investigation by Modena and da Porto (Table 2) concerns nine hollow brick masonry panels tested under a horizontal out-of-plane force applied at mid-height. The panels are mortared to rigid reinforced concrete supports at the top and at the bottom, whereas vertical edges are not restrained. Specimens FOA and FOB were constructed with horizontal hollow brick masonry while specimens FVC with vertical hollows brick masonry, thus enhancing the masonry vertical compressive strength.

Analytical out-of-plane strength was determined with Dawe and Seah's and Flanagan and Bennett's methods by setting α to zero, with Angels *et al*.'s method by setting R_1 to zero and calculating R_2 for vertical arching and with the Bashandy *et al*.'s method by eliminating the contribution of the horizontal strips.

The models by Dawe and Seah and Flanagan and Bennett underestimate the actual capacity. The Angel *et al*.'s and Bashandy *et al*.'s methods predict the actual capacity fairly well with ratios between predicted and experimental capacity ranging between 0.67 and 1.17 and

Table 2 Comparison between predicted and experimental out-of-plane capacities, Modena and da Porto (2005) tests

Spec.		DS*	A *et al**	B *et al**	FB*	EC6*
	Exp. (kN)	Pred./ exp.	Pred./ exp.	Pred./ exp.	Pred./ exp.	Pred./ exp.
FOA 1	43.17	0.53	1.07	1.19	0.48	0.77
FOA 2	43.75	0.52	1.05	1.17	0.47	0.76
FOA 3	45.06	0.50	1.02	1.14	0.46	0.74
FOB 1	63.93	0.46	1.01	1.13	0.42	0.73
FOB 2	55.29	0.53	1.17	1.30	0.48	0.85
FOB 3	58.00	0.51	1.11	1.24	0.46	0.81
FVC 1	174.06	0.28	0.74	0.82	0.26	0.54
FVC 2	192.73	0.25	0.67	0.74	0.23	0.48
FVC 3	179.18	0.27	0.72	0.80	0.25	0.52
mean		0.43	0.95	1.06	0.39	0.69
COV		0.27	0.19	0.19	0.27	0.19
mean FVC		0.27	0.71	0.79	0.25	0.51

*DS=Dawe and Seah (1989); A *et al*=Angel *et al.* (1994), B *et al*=Bashandy *et al.* (1995), FB=Flanagan and Bennett (1999c); EC6=Eurocode 6 (CEN 2005).

between 0.74 and 1.30, respectively. The use of Equation (16) (EC6) is conservative, in this case the mean value of the ratio between predicted and experimental values is 0.69.

As reported by Modena and da Porto, the arching behaviour has been observed only in specimens FVC, where vertical hollow bricks were used. In the other cases, the collapse occurred due to local shear mechanisms. Considering the FVC results, all of the considered models underestimate the actual resistance, which was, on the average, more than three times that of the other specimens.

Varela-Rivera *et al.* tested six confined masonry walls under incremental uniform static pressures applied to the walls by means of an air bag. Wall specimens were made of vertical hollow concrete blocks. Confining concrete elements for specimens E-1 and E-4 were designed to induce a snap-through failure of the walls. Confining elements for specimens E-2, E-3, E-5, and E-6 were designed to induce crushing of masonry.

The frame elements of confined masonry are usually very flexible compared to those of typical infilled frames. In this case, Eq. (4) is not applicable being the flexural rigidity less than 5.74×10^{12} Nmm^2, therefore in the Angels *et al.*'s method the factor R_2 was set to one, thus obtaining un upper bound of the predicted strength.

All the considered analytical methods give a conservative estimate of the experimental strength (Table 3). The best predictions are given by the Bashandy *et al.*'s method and by the EC6 equation.

The results related to the tests performed by Modena and da Porto and by Varela-Rivera *et al.* are summarised in Fig. 15, where the ultimate load predicted/experimental ratio is shown for the models considered. The models proposed by Angel *et al.* and Bashandy *et al.* are able to reproduce the Modena and da Porto's experimental results. The experimental results measured by Varela-Rivera *et al.* are underestimated by all of the considered models. This

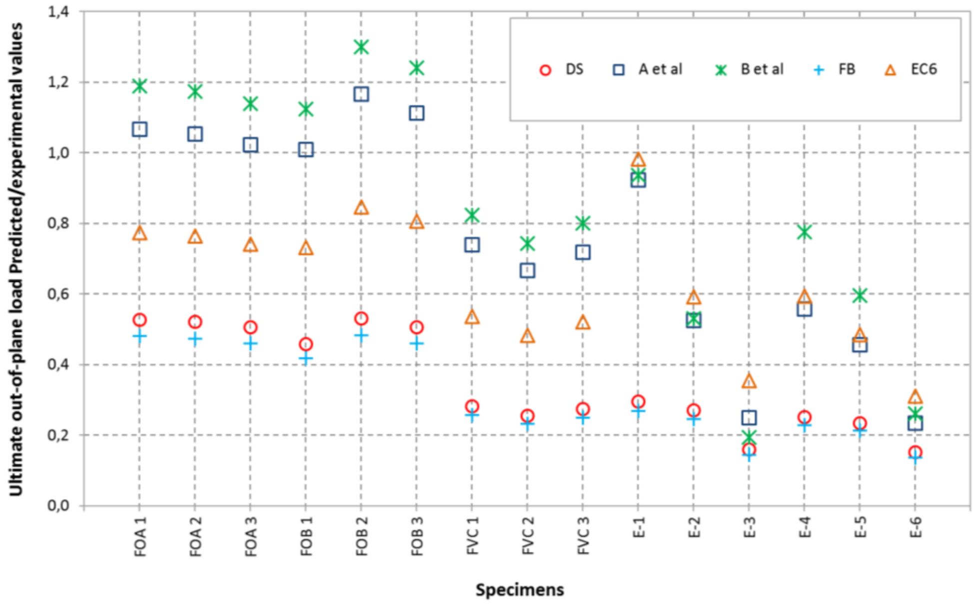


Fig. 15 Ultimate load predicted/experimental ratio of the ultimate out-of-plane load according to different models. From FOA1 to FVC3 tests by Modena and da Porto (2005), from E-1 to E-6 tests from Varela-Rivera *et al.* (2012)

Table 3 Comparison between predicted and experimental out-of-plane capacities, Varela-Rivera *et al*. (2012)

Spec.	Exp. (kN)	DS* Pred./ exp.	A *et al** Pred./ exp.	B *et al** Pred./ exp.	FB* Pred./ exp.	EC6* Pred./ exp.
E-1	8.79	0.30	0.92	0.94	0.27	0.98
E-2	13.01	0.27	0.53	0.53	0.24	0.59
E-3	12.01	0.16	0.25	0.19	0.14	0.35
E-4	14.53	0.25	0.56	0.78	0.23	0.59
E-5	17.83	0.23	0.46	0.60	0.21	0.48
E-6	15.40	0.15	0.23	0.26	0.14	0.31
mean		0.23	0.49	0.55	0.21	0.55
COV		0.24	0.47	0.48	0.24	0.40

*DS=Dawe and Seah (1989); A *et al*=Angel *et al.* (1994), B *et al*=Bashandy *et al.* (1995), FB=Flanagan and Bennett (1999c); EC6=Eurocode 6 (CEN 2005).

outcome suggests that in confined masonry structures the effective contact between masonry and reinforced concrete members compensates the reduced stiffness of the latter in enhancing the out-of-plane load carrying capacity.

Finally, coefficients of variations, COV, of the ultimate load predicted/experimental ratio are reported in Table 1 to Table 3. They vary between 0.19 and 0.55 without following a clear trend.

6. Conclusions

The assessment of the out-of-plane capacity of infills has been recently recognised as an essential issue in the damage prevention of reinforced concrete and steel frames subjected to seismic actions. Studies concerning the out-of-plane response of infills are not as many as those related to the in-plane response. Nevertheless, in the last decades, a substantial number of researches concerning the out-of-plane behaviour of infills have been carried out, both experimental and analytical. In this study, almost 150 experimental tests available in the literature have been examined to identify the main parameters affecting the out-of-plane capacity of infills. An account of damages occurred during recent earthquakes is also reported in the manuscript. The observation of damage permitted to highlight important features of the infill response, the most relevant being the fact that the out-of-plane collapse often occurs at the lower storeys of a building, although inertia forces are higher at the upper storeys. This circumstance may be attributed to the interaction between in-plane and out-of-plane loads.

Experimental tests performed to assess the capacity of infill masonry walls in resisting out-of-plane loading are generally carried out on one-bay one-storey specimens. In these experimental studies, the role of different parameters has been investigated, such as the slenderness of the infill wall, the boundary conditions (including the deformability of the frame elements), the masonry type, the effect of in-plane damage on the out-of-plane behaviour and vice versa. Main inferences can be summarised as follows.

• Both the panel slenderness and the presence of prior in-plane damage affect the out-of-plane stiffness and strength of the wall. However, such dependence is in turn influenced by the boundary conditions.

• When the infill is confined along all the edges, experimental curves show an initial linear elastic phase followed by the formation of cracks and the development of a yield-line failure mechanism; afterward, arching of infill produces a strength increase and finally, a load drop off is observed due to crushing of masonry at the crack lines and at the interface with the confining frame until total collapse. Different boundary conditions may not allow the arching mechanism to develop thus reducing the out-of-plane capacity of the wall. Moreover, certain type of masonry, e.g., those made of horizontal hole bricks, may fail due to shear forces at the top and bottom of the wall.

• Concerning the mechanical characteristics of masonry, the out-of-plane capacity resulted affected by the compressive strength rather than the tensile strength. The presence of reinforcing elements, e.g., reinforcement in the mortar layers or wire meshes in the external plaster, was found to be strongly beneficial.

• Thus far, the effect of openings has not been investigated adequately and deserves further investigation. As a matter of fact, the few studies available in the literature present dissimilar results. However, it is possible to state that openings may accelerate the out-of-plane failure because the arching mechanism cannot develop as in the case of a solid infill wall. Nonetheless, when a small opening is located at the centre of the panel, the arching mechanism can still develop in the lateral masonry segments.

Analytical and numerical analyses have confirmed the importance of the arching effect (one-way or two-way, depending on the boundary conditions) in the evaluation of the ultimate carrying capacity. Different analytical predictive expressions are available to estimate the out-of-plane capacity of infill walls. Usually, these expressions are calibrated or verified through comparison with experimental results and are thus related to a specific type of frame (reinforced concrete or steel) and of masonry (brick masonry, concrete block, etc.) and their use in different situations should be examined carefully. Models based on the arching behaviour may provide conservative or unconservative estimates of the capacity according to the model under consideration. Moreover, there are situations in which the arching behaviour does not develop even in the case of small slenderness ratios.

A review of methods proposed for the assessment of the out-of-plane response of infills is reported in this study, including those specified in current code provisions. These methods can be roughly divided in two groups according to whether they are based on analytical or numerical approaches. The formers are generally set up on the consideration of the arching mechanism, either one-way or two-way, the latter are developed in the framework of the finite or discrete elements methods. Numerical methods allow to study in detail the interaction between the frame and the infill, whereas close form equations derived by analytical implementations have the undeniable advantage of simplicity.

The comparison between five analytical models is carried out; namely, five models have been applied to reproduce the out-of-plane strength measured in 22 experimental tests. The experimental tests are selected so as to include different materials and boundary conditions: hollow brick masonry with rigid supports at the top and the bottom, hollow concrete blocks confined masonry with reinforced concrete confining elements and steel frames infilled with hollow concrete blocks. It is shown that the use of predictive equations under conditions that differ from those used for their calibration is not always appropriate. For example, as expected, the equations proposed by Dawe and Seah (1989) and Flanagan and Bennett (1999c) reproduce fairly well the results (in term of out-of-plane strength) of the experimental tests by Dawe and Seah (1989) but underestimate the resistance measured in other experimental tests. The equations proposed by Angel *et al.* (1994), Bashandy *et al.* (1995) and, with a smaller extent, Eurocode 6 (CEN 2005) are more appropriate for the estimation of the resistance of brick masonry walls, while fail to describe Dawe and Seah’s tests and are not sufficiently flexible to take into account different intermediate boundary conditions. The resistance of confined masonry is underestimated by all the considered models.

Summarising, the great variability of the materials and the large number of parameters involved, makes it difficult the selection of a unique model for the assessment of the out-of-plane strength. And, even though different models take into account essential parameters, such as the slenderness and the boundary conditions of the panel, the strength of the masonry and the presence of cracks due to prior in-plane damage, the interaction among these factors is not straightforward and requires further investigation.

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