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Assessment of the Out-of-Plane Resistance of Masonry Infill Walls

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Abstract

Observation of damage after earthquakes has highlighted that out-of-plane failures of masonry infill walls may often occur even in the case of low or moderate earthquakes, causing both casualty risks for people and unfavourable situations affecting the overall structural response. In recent years, several experimental tests have been carried out trying to determine the relevance of different parameters that may affect the out-of-plane carrying capacity of infill walls, such as the slenderness ratio and the boundary conditions of the panel, the mechanical characteristics of the masonry, the stiffness of the surrounding frame elements and the presence of cracks arising from prior in-plane damage. Several analytical models have been developed too, and also included in code provisions; the application of these models to experimental data performed in this paper shows that the problem is still open and need further investigation. As a general observation, analytical models underestimate or overestimate the ultimate load carrying capacity according to the type of frame and infills under consideration, thus showing that the validity of each model is limited to specific configurations.

Keywords: masonry infills, predicting models, out-of-plane loads, slenderness, boundary conditions.

1 Introduction

In recent years the interest in the out-of-plane response of infills has been growing due to the necessity to limit damages to these elements, commonly considered as non structural. Their failures may cause casualty risk and heavy socio-economic consequences, such as loss of building functionality. Moreover, the total or partial collapse of an infill may determine injures to people and affect the overall structural response causing, for example, the formation of an open storey, which may result in a soft-storey mechanism.

The observation of damage after recent earthquakes has shown several cases of out-of-plane failures of masonry infill even in case of low or moderate earthquakes [1] [2] [3] [4] [5] [6]. In many cases, the structure withstands the ground shaking with minor damage, whereas infills collapse in-plane or out-of-plane (Figure 1).

(a) (b)

Figure 1: Out-of-plane expulsion of infills: a) L'Aquila (Italy) earthquake of 2009; (b) Emilia (Italy) earthquake of 2012.

Most of the studies on infill panels have been focused on their in-plane response. However, in the last decades several experimental tests have been conducted by different researchers to assess strength and ductility of masonry walls loaded in the out-of-plane direction. The published literature reports monotonic, cyclic and dynamic tests on masonry panels. Such studies have investigated the influence of various factors, e.g. the slenderness ratio, the panel thickness, the boundary conditions, the presence of prior in-plane damage. A review is presented in [7].

Experimental tests have shown that both the slenderness (i.e. the height/thickness ratio) of the panel 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. The presence of reinforcing elements in the masonry, e.g. reinforcement in the mortar layers or wire meshes in the external plaster, has been found to be strongly beneficial.

A really important factor that affects the infill behaviour is given by the boundary conditions [8]. Figure 2 shows the effect of boundary conditions on crack patterns: (a) panel supported on top and bottom; (b) panel supported along three sides and free at the top; (c) panel supported on three sides, but with no gap at the top boundary (arching action develops also in the vertical span, even though slippage at the top interface is not restricted); (d) panel with all four sides supported.

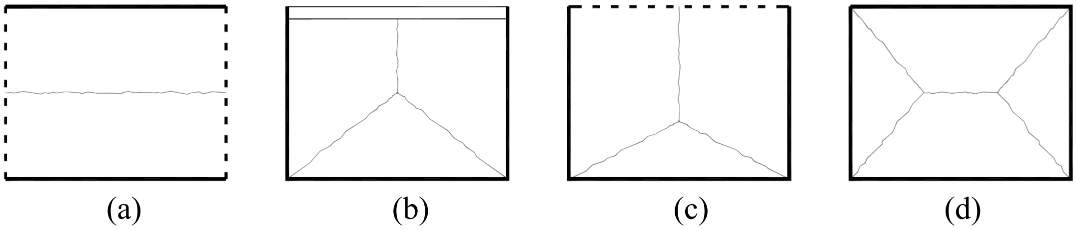


Figure 2: Effect of boundary conditions on crack patterns

If the infill is confined along all the edges, experimental curves of out-of-plane load versus deflection of the infill show four stages. In the first stage the behaviour is linear elastic; in the second stage the propagation of cracks and the development of a yield-line failure mechanism occur; in the third stage, arching of infill causes the load to increase and finally, in the fourth stage the load drops off due to crushing of masonry at the crack lines and at the interface with the confining frame until total collapse. Different boundary conditions or high flexibility of the frame elements may not allow the arching mechanism to develop thus reducing the out-of-plane capacity of the wall.

The effect of openings on the out-of-plane resistance may be relevant but the few studies available in literature up to now present dissimilar results. However, it is possible to state that openings may accelerate the out-of-plane failure because they affect the in-plane stiffness and strength [9], thus increasing the in-plane damage. This problem is not dealt with in the present paper.

Different analytical models have been developed for the assessment of the out-of-plane response of masonry infills taking into account the above mentioned parameters. However, they are usually based on single experimental campaigns and their suitability to situations different from those of the specific test must be carefully assessed. As a matter of fact, equations proposed in the literature for the evaluation of out-of-plane stiffness and/or strength of infills are based on experimental or numerical campaign in which few parameters change, e.g. the slenderness ratio, the aspect ratio, the mechanical characteristics of masonry, the level of prior in-plane damage, etc. and in some cases only few of them. The type of frame (e.g. steel or reinforced concrete), the type of the test (e.g. cyclic, monotonic, dynamic, pseudo-dynamic, numerical, etc.) does not generally change within each study. Therefore the results and the related proposed formulations are valid under the same conditions of the test.

In this paper, a review and a comparison of analytical models developed for the assessment of the out-of-plane carrying capacity of masonry infills and Code provisions is presented, focusing on models based on the arching theory. The application of these simplified models to experimental tests allows to appreciate the degree of approximation of such methods and their reliability.

2 Predicting models

The aim of the present study is mainly to perform a comparison between several predicting models and experimental result. To this aim, on the basis of the state of the art review on available analytical models for the assessment of the out-of-plane response of masonry infills carried out in [7], the more relevant models are resumed and applied to several cases.

The models that are considered are based on rigid body mechanisms with the description of the arching behaviour. More complex modelling, which involves numerical or iterative solutions or the application of finite element methods (e.g. [10], [11]), are not tackled in this paper. On the other side, some code provisions are analysed as well.

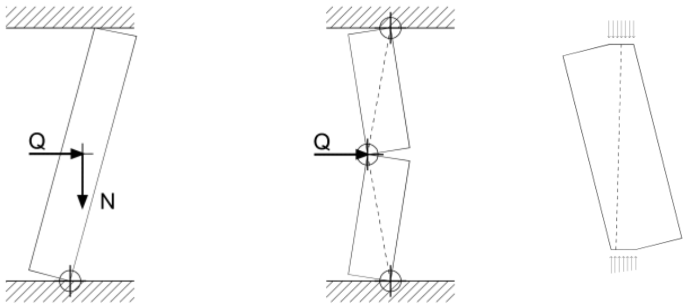
2.1 Analytical models

The prediction of the out-of-plane ultimate load carrying capacity of masonry infill walls has been approached analytically according to different models, either in the elastic field or in the rigid-plastic one. Notwithstanding the circumstances that many models are often able to take into account various factors, 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 the different factors is not clear and the large number of parameters required makes the solution quite complex.

In the elastic field, classical solutions for an elastic plate by Timoshenko (1959) can be used to represent two-way bending of an infill. With this model, failure is assumed to occur when the tensile stress reaches the tensile strength of the masonry.

Approaches based on the modified yield-line analyses have been developed too. 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. Based on the yield-line analysis, Haseltine et al. [12] and Hendry [13] proposed equations in which the out-of-plane strength is expressed as a function of the flexural tensile strength normal to the bed joints.

Several 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 (Figure 3). In the rigid-body mechanism that considers the wall as a whole with a hinge at the bottom of the panel (Figure 3a), out-of-plane stability is verified by the equilibrium condition between the stabilizing action (weight of the wall) and the overturning action (seismic action). Static and dynamic models for this condition have been proposed by Sorrentino et al. [14]. Field observations have suggested that the failure may occur due to local cracks at the centre of the panel. 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 (Figure 3b); 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.



(a) (b) (c)

Figure 3: Rigid body models: a) without the arching effect; b-c) with the arching effect.

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. [15] in 1956; the wall is modelled as an ideal beam constrained between rigid supports on the two edges. 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 (Figure 3b). Further resistance is given by the crushing of the material at the hinges location (Figure 3c). Ultimate capacity is given by the following equation:

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

where f'_m is the masonry compressive strength, h is the panel height, t is the panel thickness and γ is a function which depends on: strain associated with the crushing strength, stress distribution along contact area, deflection at the centre of the wall and wall thickness.

When the infill is restrained at four edges, a two-way arching action develops, as also confirmed by experimental tests.

The two-way arching action was investigated by Dawe and Seah [8], who developed a strength theory based on virtual work concepts, modifying the conventional yield-line method. Specifically, the wall is divided into a number of horizontal and vertical strips. Flexural resisting moments between strip segments are then calculated using the compressive strut forces developed by an arching action. The flexibility of the frame is explicitly taken into account. Based on this method, Dawe and Seah performed a parametric study to evaluate the effect on ultimate load of several parameters and proposed the following empirical relations for the design of panels supported on four sides:

$$q = 4.5\left(f_m^{'}\right)^{0.75} t^2\left(\alpha/l^{2.5} + \beta/h^{2.5}\right) \tag{2}$$

and panels supported on three sides and free at the top:

$$q = 4.5\left(f_m^{'}\right)^{0.75} t^2 \, \alpha / l^{2.5} \tag{3}$$

where: q is the ultimate capacity (kPa), f'_m is the masonry compressive strength (kPa), t is the panel thickness (mm), l is the panel length (mm) and h is the panel height (mm); α and β are parameters which depend on the bending (EI) and torsion (GJ) stiffness of the columns and of the beams, respectively:

$$\alpha = \frac{1}{H}\left(EI_c H^2 + GJ_c tH\right)^{0.25} \tag{4}$$

$$\beta = \frac{1}{L}\left(EI_b L^2 \; + \; GJ_b tL\right)^{0.25} \leq 50 \tag{5}$$

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. The above equations have been derived for hollow concrete block panels within steel frame having pinned joints.

Equation (2) was modified by Flanagan and Bennett [16] on the basis of 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 torsion stiffness of the frame members.

Angel et al. [17] and Abrams et al. [18] developed a model which takes into account the effect of in-plane damage on the out-of-plane resistance. The model is based on the one-way arching mechanism. The following expression derives from both equilibrium considerations and experimental results:

$$q = \frac{2f'_m}{(h/t)} \, \lambda R_1 R_2 \tag{6}$$

in which q is the uniform pressure on the wall which causes out-of-plane failure, f'_m is the masonry compressive strength, λ is the slenderness parameter, h and t are the height and thickness of the infill, respectively, R_1 and R_2 are reduction factors accounting for previous in-plane damage and for the frame flexibility, respectively.

The reduction factor, R_1, which accounts for the magnitude of prior in-plane damage [18], is given by:

$$R_1 = \left[1.08 - 0.015(h/t) - 0.00049(h/t)^2 + 0.000013(h/t)^3\right]^{\frac{\Delta}{2\Delta_{\mathrm{crack}}}} \tag{7}$$

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 panels is confined within a frame having neighbouring panels in every direction, then $R_2 = 1$. Otherwise the following expressions apply:

$$\begin{aligned} R_2 &= 0.357 + 2.49 \times 10^{-14} EI \quad \text{for} \quad 5.74 \times 10^{12} \leq EI \leq 25.83 \times 10^{12} \\ R_2 &= 1 \qquad\qquad\qquad\qquad \text{for} \quad EI > 25.83 \times 10^{12} \end{aligned} \tag{8}$$

where EI is the flexural rigidity (expressed in Nmm^2) of the smallest frame member at the side where a neighbouring panel is missing [17].

Equation (6) 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 [18]:

$$\left(\frac{h}{t}\right)_{cr} = 0.981 \sqrt{\frac{2}{\varepsilon_{cu}}} \tag{9}$$

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}.

In order to include two-way arching action, Bashandy et al. [19] extended the analytical method developed by McDowell et al. [15]. The panel is divided into vertical and horizontal strip segments experiencing the crack pattern shown in Figure 4. 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.

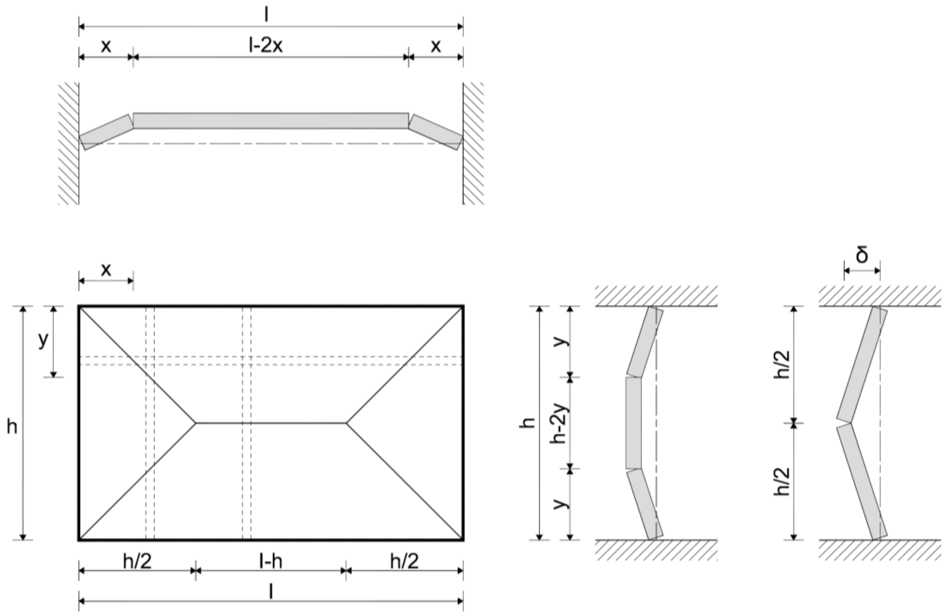


Figure 4: Crack pattern in infill wall and strips model [19].

The total force resistance, Q, is calculated assuming an equivalent rectangular stress pattern in the contact area of 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:

$$Q = 8\,\frac{M_{yv}}{h}(l-h) + 8\,M_{yv} ln(2) + 8\,\frac{M_{yh}}{h}\left(\frac{x_{yv}}{x_{yh}}\right) ln\left(\frac{l}{l-h/2}\right) l \tag{10}$$

where x_{yv} and x_{yh} are:

$$x_{yv} = \frac{t\,f_m^{'}}{E_m\left(1 - h/\left(2\sqrt{(h/2)^2 + t^2}\right)\right)} \tag{11}$$

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

where f'_m is the ultimate compressive strength; E_m is the masonry modulus of elasticity, t is the panel thickness; h and l are the panel height and width, respectively; M_{yv} and M_{yh} are obtained by substituting the values of x_{yv} and x_{yh}, respectively, in Equation (13) in lieu of x_y:

$$M_y = \frac{0.85 f_m^{'}}{4}\left(t - x_y\right)^2 \tag{13}$$

In Equation (10) 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.

2.2 Design code provisions

Attention to the behaviour of non-structural elements such as infill walls that might, in case of an earthquake, cause severe damages to people, even if the structure still maintains its bearing capacity has been given by several codes and commentaries. In particular infill walls subjected to out-of-plane loads are addressed in International and European design provisions.

In FEMA 306 [20] and NZSEE recommendations [21], the equation proposed by Angel and Abrams [17] for the assessment of the out-of-plane infill strength and given by Equation (6) is used.

According to FEMA 356 [22], 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 analysed for out-of-plane seismic forces. Limit values of the slenderness vary from 8 to 16 depending on the performance level and on the seismic zone. If the slenderness limits are not accomplished but requirements for arching action are met, then the following expression is provided for the evaluation of the lower bound out-of-plane strength, q, of an infill panel:

$$q = \frac{0.7 f'_m \lambda_2}{h/t} \tag{14}$$

where f'_m is the lower bound of masonry compressive strength, λ_2 is the slenderness parameter, h and t are the height and thickness of the infill, respectively. This expression is a modification of Equation (6), the numerical constant 2 in Equation (6) is changed to 0.7 in Equation (14) and the parameter λ_2 in Equation (14) is lower than λ in Equation (6). These modifications are due to the fact that the FEMA 356 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 should be evaluated as a function of the lower bound masonry flexural tension strength.

Analysing European codes, Eurocode 6 [23] deals with the problem of walls arching between supports, suggesting that in case the wall is built solidly between support 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 given by:

$$q_d = f_d \left(\frac{t}{l_a}\right)^2 \tag{15}$$

where f_d is the design compressive strength of the masonry in the direction of the arch thrust, t is the thickness of the wall 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 does not exceed 20.

Furthermore, Eurocode 8 [24] 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 [25], 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 and current Italian code.

3 Comparison between analytical and experimental results

To assess the range of validity of various methods, the out-of-plane capacity has been estimated using the equations suggested in [8] [16] [17] [19] [23] and compared with the capacity measured in experimental tests. To this aim 22 experimental tests available in the literature have been selected so as to represent different types of frame and infill, i.e.: i) hollow brick masonry supported on the top and the bottom with rigid reinforced concrete elements [26], ii) hollow concrete blocks confined masonry with reinforced concrete confining elements [11] and iii) steel frames infilled with hollow concrete blocks [8]. The results are presented in Table 1, Table 2 and Table 3, respectively. In the tables, the measured peak loads and the comparison with predicted values are reported. The mean value of the ratio between predicted and experimental values is also shown. For these comparisons the values predicted by EC6 equation are estimated by considering the mean compressive strength of masonry in lieu of the design strength.

The experimental investigation by Modena and da Porto [26] (Table 1) concerns nine hollow bricks masonry panels tested under a horizontal out-of-plane force applied at mid-height. At the top and at the bottom the panels are mortared to rigid reinforced concrete supports whereas vertical edges are not restrained. Masonry of specimens FOA and FOB were constructed with horizontal hollow bricks whereas masonry of specimens FVC with vertical hollows bricks thus enhancing the masonry compressive strength.

Analytical 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 1.11 and between 0.74 and 1.30, respectively. The use of Equation (15) (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, whereas 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 more than three times that of the other specimens.

Varela-Rivera et al. [11] (Table 2) tested six confined masonry walls under incremental uniform static pressures applied to the walls using an air bag. Wall specimens were constructed using 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, Equation (8) 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.

All the considered analytical methods give a conservative estimate of the experimental strength. The best prediction are given by the Bashandy et al.'s method and by the EC6 equation.

Specimen		Dawe and Seah [8]	Flanagan and Bennett [16]	Angel et al. [17]	Bashandy et al. [19]	EC6 [23]
	Exp. (kN)	Pred./ exp.	Pred./ exp.	Pred./ exp.	Pred./ exp.	Pred./ exp.
FOA 1	43.17	0.53	0.48	1.07	1.19	0.77
FOA 2	43.75	0.52	0.47	1.05	1.17	0.76
FOA 3	45.06	0.50	0.46	1.02	1.14	0.74
FOB 1	63.93	0.46	0.42	1.01	1.13	0.73
FOB 2	55.29	0.53	0.48	1.17	1.30	0.85
FOB 3	58.00	0.51	0.46	1.11	1.24	0.81
FVC 1	174.06	0.28	0.26	0.74	0.82	0.54
FVC 2	192.73	0.25	0.23	0.67	0.74	0.48
FVC 3	179.18	0.27	0.25	0.72	0.80	0.52
mean		0.43	0.39	0.95	1.06	0.69
mean FVC		0.27	0.25	0.71	0.79	0.51

Table 1: Modena and da Porto tests [26]: comparison between predicted and experimental values of the out-of-plane capacity.

Specimen		Dawe and Seah [8]	Flanagan and Bennett [16]	Angel et al. [17]	Bashandy et al. [19]	EC6 [23]
	Exp. (kPa)	Pred./ exp.	Pred./ exp.	Pred./ exp.	Pred./ exp.	Pred./ exp.
E-1	8.79	0.30	0.27	0.92	0.94	0.98
E-2	13.01	0.27	0.24	0.53	0.53	0.59
E-3	12.01	0.16	0.14	0.25	0.19	0.35
E-4	14.53	0.25	0.23	0.56	0.78	0.59
E-5	17.83	0.23	0.21	0.46	0.60	0.48
E-6	15.40	0.15	0.14	0.23	0.26	0.31
mean		0.23	0.21	0.49	0.55	0.55

Table 2: Varela-Rivera et al. tests [11]: comparison between predicted and experimental values of the out-of-plane capacity.

Dawe and Seah [8] tested nine steel frames infilled with 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 top beam to panel interface was provided. For these tests 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, 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 3.

Specimen		Dawe and Seah [8]	Flanagan and Bennett [16]	Angel et al. [17]	Bashandy et al. [19]	EC6 [23]
	Exp. (kPa)	Pred./ exp.	Pred./ exp.	Pred./ exp.	Pred./ exp.	Pred./ exp.
WE1	22.30	1.74	1.59	3.90	8.01	6.30
WE2	19.20	1.90	1.73	4.18	8.62	6.74
WE4	11.20	1.51	1.38	2.54	3.46	5.07
WE5	7.80	0.82	0.75	0.70	0.48	2.68
WE6	10.60	0.95	0.86	2.39	5.84	5.86
WE7	14.70	0.70	0.64	1.78	4.07	4.36
WE8	13.40	1.45	1.32	2.56	3.37	9.42
mean		1.30	1.18	2.58	4.83	5.77

Table 3: Dawe and Seah tests [8]: comparison between predicted and experimental values of the out-of-plane capacity.

The method which better predicts the observed experimental values is the Flanagan and Bennett's method. In this case the mean of the ratios between predicted and experimental values is 1.18. Clearly, the Dawe and Seah equation predicts a slightly higher strength than that predicted by Flanagan and Bennett. 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 noticeably overestimates the experimental strength.

A summary of results is reported in Figure 5a, where the mean values of the predicted/experimental values are shown for the models considered. As already observed, the results of the experimental tests by Dawe and Seah 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. Contrarily, 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. The coefficients of variations of the predicted/experimental values are reported in Figure 5b for each model. They vary between 0.19 and 0.55 without following a clear trend.

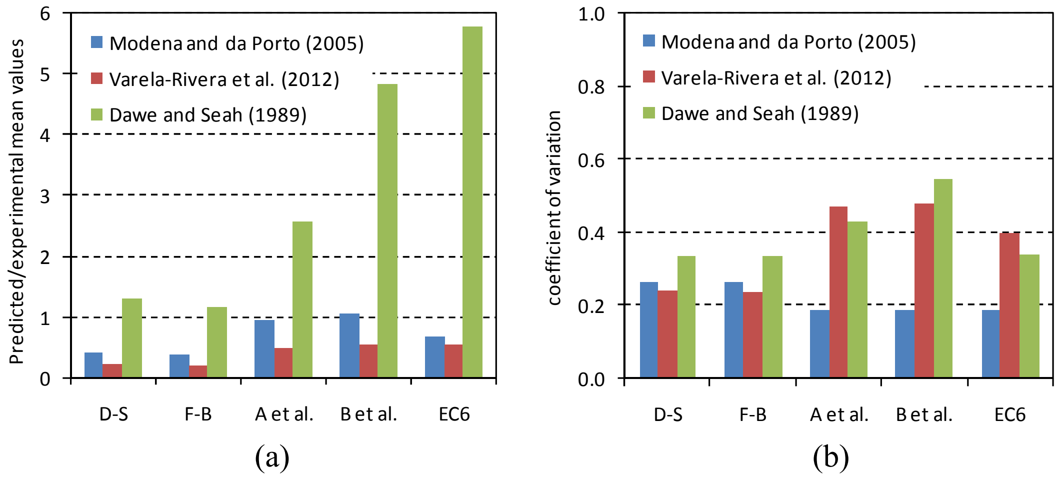


Figure 5: Ratios between predicted and experimental values: a) mean values; b) coefficient of variation. (D-S = Dawe and Seah, F-B = Flanagan and Bennett, A et al. = Angel et al., B et al. = Bashandy et al., EC6 = Eurocode 6).

4 Conclusions

Analytical predictive expressions are available in the literature to estimate the out-of-plane carrying capacity of infill walls. 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.). In this paper, a brief review of analytical methods proposed by different authors is presented. Afterward, five analytical methods are used to reproduce the out-of-plane strength measured during several experimental tests.

The experimental tests concern 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 the predictive equations in situations that differ from those used for their calibration is not always appropriate. For example, as expected, the equations proposed by Dawe and Seah [8] and Flanagan and Bennett [16] reproduce fairly well the results (in term of out-of-plane strength) of the experimental tests by Dawe and Seah [8] but underestimate significantly the resistance measured in other experimental tests. The equation proposed by Angel et al. [17], Bashandy et al. [19] and Eurocode 6 [23] are more appropriate for the estimation of the resistance of brick masonry walls. The resistance of confined masonry is underestimated by all the considered models.

On the basis of these results, it can be inferred that the issue of the assessment of the out-of-plane capacity of infill walls is still open. The great variability of the materials and the large number of parameters involved, makes difficult the selection of a unique model for the assessment of the out-of-plane strength. Finally, although some initial conclusions have been drawn from this study, more research needs to be conducted on this subject, including the modelling of the effect of prior in-plane damage and of the presence of openings.

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