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Strength and stiffness reduction factors for infilled frames with openings

Luis D. Decanini†, Laura Liberatore‡ and Fabrizio Mollaioli§

Department of Structural Engineering and Geothecnics, "Sapienza" University of Rome, Via Gramsci, 53-00197 Rome, Italy

Abstract: Framed structures are usually infilled with masonry walls. They may cause a significant increase in both stiffness and strength, reducing the deformation demand and increasing the energy dissipation capacity of the system. On the other hand, irregular arrangements of the masonry panels may lead to the concentration of damage in some regions, with negative effects; for example soft story mechanisms and shear failures in short columns. Therefore, the presence of infill walls should not be neglected, especially in regions of moderate and high seismicity. To this aim, simple models are available for solid infills walls, such as the diagonal no-tension strut model, while infilled frames with openings have not been adequately investigated. In this study, the effect of openings on the strength and stiffness of infilled frames is investigated by means of about 150 experimental and numerical tests. The main parameters involved are identified and a simple model to take into account the openings in the infills is developed and compared with other models proposed by different researchers. The model, which is based on the use of strength and stiffness reduction factors, takes into account the opening dimensions and presence of reinforcing elements around the opening. An example of an application of the proposed reduction factors is also presented.

Keywords: infilled frames; openings; strength; stiffness; reduction factor

1 Introduction

Steel and reinforced concrete (RC) framed structures are usually infilled with masonry walls, used as interior partitions and external walls. It is widely recognized that framed structures benefit from the presence of regularly distributed infills, which significantly contribute to withstand the seismic actions (Housner, 1956; Moghaddam and Dowling, 1987; Fardis, 1996; Negro and Verzelletti, 1996; Kappos *et al*., 1998; Liberatore *et al*., 2004; Dolšek and Fajfar, 2008) as also proved during moderate and strong earthquakes (Mostafaei and Kabeyasawa, 2004; Decanini *et al*., 2005; Decanini *et al*., 2012). Usually stiffness and strength of the infill and connections between infill and frame are such that the infill alters the global seismic behavior of the structure. In fact, the infills may cause significant increase in both stiffness and strength. The increase of stiffness shifts the fundamental period such that the story shears are in general slightly larger in the infilled structure than in the bare frame but a large part of them is resisted by the infills themselves (Fardis, 1996). Moreover, the infills reduce the deformation demand and improve the energy dissipation capacity of the system. On the contrary, irregular arrangement of infills may be strongly detrimental, producing unfavorable distribution of plastic hinges, high demand of inelastic deformations and reduction of the global dissipation capacity. Hence, the presence of infill walls should be considered in analysis and design procedures. To this aim, simple models for solid infill walls, such as the equivalent diagonal no-tension strut model, are available. Usually these models do not take into account the presence of openings, which are often provided for functional requirements of buildings.

The openings in the infill walls lead to significant uncertainty in the assessment of the seismic behavior of the structure due to the variability of their size and location. In general, the presence of openings results in a reduction of stiffness and ultimate strength of the panel and in a reduction of the energy dissipation capacity. Moreover, openings may accelerate the out-of-plane failure because the arching mechanism cannot develop as in the case of a solid infill wall. The presence of openings also affects the crack pattern as cracks may develop first at the corners of the opening and propagate towards the compressed corners. However, in general, the crack pattern depends on the position and size of the opening (Mosalam *et al*., 1997).

The influence of openings on strength and stiffness has been investigated by several researchers. One of the first experimental studies on infilled frames with openings was carried out by Polyakov (1956) on eight infilled steel frames with openings of different sizes. In this study, the ultimate strength of the perforated models

Correspondence to: Laura Liberatore, Department of Structural Engineering and Geothecnics, "Sapienza" University of Rome, Via Gramsci, 53-00197 Rome, Italy
Tel: +39 06 49919182; Fax: +39 06 49919192
E-mail: laura.liberatore@uniroma1.it
†Professor; ‡Assistant Professor; §Associate Professor
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was estimated between 23% and 76% of the frame with the solid panel. Another early study was performed by Benjamin and Williams (1958) on an infilled steel frame with a central opening with dimensions of 1/3 of the infill panel dimensions; the reduction of the ultimate strength due to the opening was about 45%. Other experimental and numerical tests were performed since 1960. Very recent research has been carried out by Stavridis *et al*. (2012) and Mohammadi and Nikfar (2013). Literature reviews on the effect of openings in infill panels were carried out by Moghaddam and Dowling (1987) and by Smyrou (2006), who highlighted the lack of recommendations and of an integrated way to quantify the effect of openings.

The main objective of this study is the evaluation of the effects of openings on the lateral strength and stiffness of infilled frames and the development of a model to account for such effects. The influence of openings of different sizes is studied and a simple model to take into account the presence of openings in the infills is proposed and compared with other models available in the literature. The empirical model developed herein is based on the use of reduction factors to be employed in the diagonal no-tension strut modelling approach. The equation proposed for the reduction factor (Eq. (10)) is a function of both the area and length of the opening and depends on the presence and type of reinforcing elements around it. An example of its application is presented in the last section.

2 Code background and modelling aspects

2.1 Code provisions

Infills are used in steel and RC buildings in many countries and have a significant effect on their seismic behavior. The way in which infills are taken into account in the design process differs noticeably from one country to another. As observed by Kaushik *et al*. (2006), current national codes can be roughly grouped into two categories according to whether or not they consider the role of the infills in the design process.

The Russian code (SNIP-II-7-81, 2001) specifically recommends isolating the non-bearing walls from the frame so that they do not affect the overall stiffness of the system. The separation prevents negative effects associated with irregular distribution of the infills and brittle behaviors.

In the "Recommendations of the New Zealand Society for Earthquake Engineering" (NZSEE, 2006), two situations are explicitly considered: i) the infills are very light and flexible, or completely isolated from the frame, or so brittle that a total failure is expected even for moderate ground accelerations; in this case, the presence of infills does not affect the structural response; ii) the infills are assessed to have a significant influence on the response. If they are expected to remain in the elastic range, then a linear elastic analysis can be performed; if they are expected to suffer significant damage during the seismic event, then the high probability of the formation of a soft story has to be identified and taken into consideration.

The provisions for the design of infilled RC frames contained in Eurocode 8 (2003) are mainly related to the definition of irregularity. In the code, penalty factors are specified for the regions where irregularities occur while the effect of uniformly distributed infills on the global response is not considered in the design.

Only a few national codes and standards deal with the effects of openings. One of these is the "Nepal National Building Code" (NBC-201, 1994), according to which only infill walls with openings having an area less than 10% of the gross wall area can be considered as resisting to seismic loads. Such openings must be located outside the restricted zone, i.e., the zone at the corner of a panel bounded by the outer one-third of the panel dimension and, if they are located in the middle two-thirds of the panel, they must be provided by framing RC elements (Fig. 1).

According to the NZSEE Recommendations (2006), the effect of an opening may be accounted for by means of a simplified approach based on the work of Dawe and Seah (1988), in which the reduction of stiffness and strength of the panel due to an opening is taken into account through a reduction factor, λ_{opening}, given by the following equation:

$$\lambda_{\text{opening}} = 1 - \frac{1.5 L_{\text{opening}}}{L_{\text{inf}}}; \quad \lambda_{\text{opening}} \geq 0 \tag{1}$$

where L_{opening} is the maximum horizontal width of the opening and L_{inf} is the length of the infill panel. The above equation entails that if the opening length exceeds two-thirds of the panel length, the infill is not considered.

In Eurocode 8 (2003), the presence of opening is considered for the damage limitation of infills. Particularly, it is prescribed that the edges of large openings must be trimmed with belts and posts. For confined masonry, vertical confining elements should be placed at both sides of any opening having an area of more than 1.5 m^2.

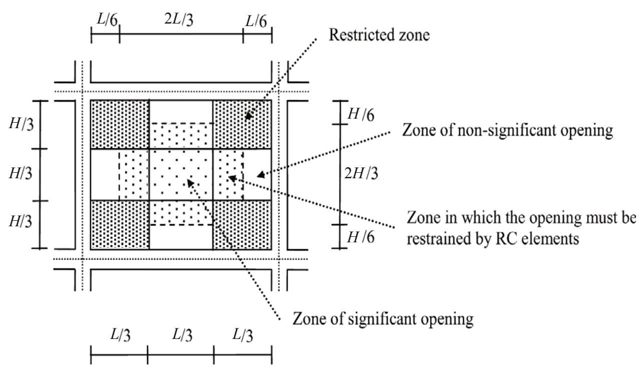


Fig. 1 Definition of different portions of a perforated panel according to the Nepal National Building Code (NBC-201, 1994)

2.2 Modelling of infilled frames

Several analytical models have been developed to represent the behavior of infilled frames. The various methods proposed may be roughly divided into two groups according to whether they are based on a macro-model approach (strut model) or on a micro-model approach (finite element).

The finite element approach is based on a finite element representation of the frame and the infill. In these models, the response of the frame, the infill and their interface is described by means of proper constitutive relations. For example, smeared crack models have often been used to model both the frame and the infills; these models cannot capture different aspects, such as the shear sliding of masonry mortar. To reproduce this effect, several plasticity-based continuous interface models have been developed (Shing and Mehrabi, 2002).

In the finite element method, the presence of openings can be accounted for directly. In general, this approach allows accurately reproducing the behavior of the masonry if it is properly developed, but inadequate use of such models may lead to incorrect results (Shing and Mehrabi, 2002). Moreover, their use is quite complex due to the large amount of information demanded. On the contrary, the macro-models, even though they do not capture the local phenomena occurring between the infill panel and the surrounding frame, are characterized by an advantageous simplicity. Often, the finite element analyses of infilled frames were aimed to the calibration of the parameter for the constitutive laws of simpler models, like the equivalent strut model.

The equivalent diagonal strut model was initially based on the observation that the compressive path in the masonry panel, due to horizontal loads, develops mainly along its diagonal. Therefore, a way to represent the stiffening and strengthening effect of the masonry infill is replacing the panel with an equivalent no-tension strut acting along the compressive path (Stafford Smith, 1963; Mainstone, 1974). The width of the strut depends on different features, such as the extension of the region of interaction between masonry and frame. The ultimate strength of the infills depends also on the failure mechanism, which is somewhat difficult to predict since it is affected by many factors, such as the material properties, the dimensions of the system and the stress level in the panel. Keeping in mind that the masonry is a heterogeneous material, the strut model can be regarded as a method to reproduce only the global behavior and its suitability depends on the appropriate calibration of the parameters involved.

The application of the strut model to the infills with openings can follow two different methods. One is based on the use of several diagonal struts around the opening (Thiruvengadam, 1985; Hamburger and Chakradeo, 1993). The multi-strut configuration takes into account the presence of openings but the evaluation of the characteristics of the struts (position, width, etc.) is somewhat complex. The second method consists of modifying the width of the diagonal strut by means of a proper coefficient (reduction factor). The proposals formulated by Polyakov (1956), Sachanski (1960), Imai (1989), Durrani and Luo (1994), Al-Chaar (2002), Asteris (2003), Mondal and Jain (2008), and Tasnimi and Mohebkhah (2011) belong to this procedure.

When the opening is relatively small, the transfer of shear is still possible with a diagonal strut, while the diagonal compression strut mechanism cannot develop when the opening is larger (Durrani and Luo, 1994). In the latter case, the diagonal strut does not represent the actual stress distribution in the panel but is a way to take into account the role of the infill panel with openings in the global behavior of the frame-infill system.

In the following discussion, the reduction factors proposed by different researchers are reported with the notations adopted in this study (Fig. 2).

Polyakov (1956) proposed the following expression, valid for $\alpha_{\mathrm{h}} \leq 65\%$ and $\alpha_{\mathrm{a}} \leq 60\%$

$$\rho_{\mathrm{Pol}} = 1 - 0.01\left(1.155\alpha_{\mathrm{h}} + 0.385\alpha_{\mathrm{a}}\right) \tag{2}$$

Sachanski (1960), on the basis of theoretical and experimental investigations, suggested the following expression for the strut width reduction factor:

$$\rho_{\mathrm{Sach}} = 1 - \left(0.004\alpha_{\mathrm{l}} + 0.006\alpha_{\mathrm{h}}\right) \tag{3}$$

Imai (1989) used the following reduction factor for the evaluation of the shear strength in panels with openings:

$$\rho_{\mathrm{Imai}} = \min\left(1 - 0.01\alpha_{\mathrm{l}}; 1 - 0.1\alpha_{\mathrm{a}}^{\ 0.5}\right) \tag{4}$$

Durrani and Luo (1994), on the basis of finite element analyses on RC infilled frames with central openings, suggested the following formula:

$$\rho_{\mathrm{DL}} = 1 - \left(\frac{A_{\mathrm{d}}}{LH}\right)^2; \quad A_{\mathrm{d}} = LH - \frac{\left(d\sin(2\theta) - d_o\sin(\theta+\varphi)\right)^2}{2\sin(2\theta)} \tag{5}$$

where L, H, d, d_o, θ, and φ are geometrical features (Fig. 2).

According to Al-Chaar (2002), the width reduction factor should be calculated using the following equation:

$$\rho_{\mathrm{Al\text{-}Chaar}} = 0.6\left(\frac{\alpha_{\mathrm{a}}}{100}\right)^2 - 1.6\left(\frac{\alpha_{\mathrm{a}}}{100}\right) + 1 \tag{6}$$

valid for $\alpha_{\mathrm{a}} < 60\%$. If the area of the openings is greater than or equal to 60% of the area of the infill panel, then the effect of the infill should be neglected.

Asteris (2003) defined an elastic stiffness reduction factor as a function of α_{a} for three different positions of the opening: i) opening underneath the compressed diagonal; ii) opening upon the compressed diagonal; and iii) opening above the compressed diagonal. The reduction factor curves are shown in Asteris (2003).

Mondal and Jain (2008) investigated the effect of central openings on the initial lateral stiffness of infilled

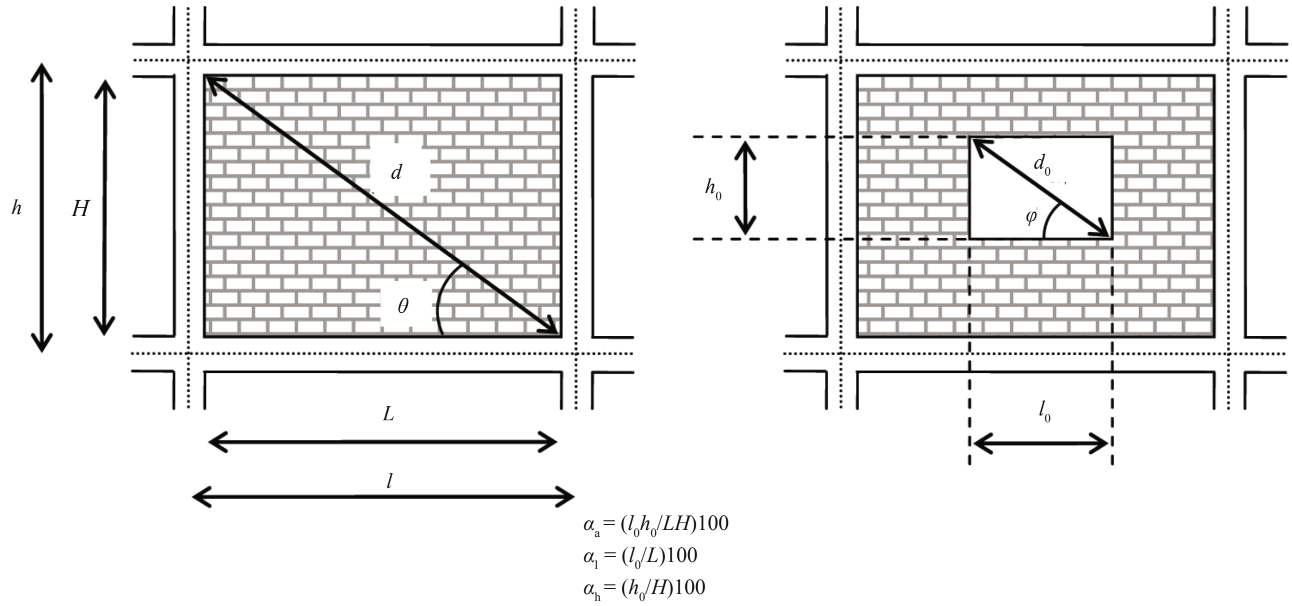


Fig. 2 Notations adopted in the study

frames by means of finite element analyses and proposed the following equation for the reduction of the strut width:

$$\rho_{\mathrm{MJ}} = 1 - 2.6\frac{\alpha_{\mathrm{a}}}{100} \quad (7)$$

This equation implies that when α_{a} > 38.5%, the contribution of the infill is neglected.

Finally, Tasnimi and Mohebkhah (2011), on the basis of experimental tests on brick-infilled steel frames, proposed the following reduction factor, valid for for α_{a} < 40%

$$\rho_{\mathrm{TM}} = 1.49\left(\frac{\alpha_{\mathrm{a}}}{100}\right)^2 - 2.238\frac{\alpha_{\mathrm{a}}}{100} + 1 \quad (8)$$

The above expressions do not take into account the presence of reinforcing elements around the opening.

3 Effect of openings on the lateral stiffness and strength

To identify the main parameters that affect the response of infills with opening and to evaluate their influence on the strength and stiffness of the infills, a number of experimental and numerical tests available in the literature are considered in the present study. A comparison between the strength reduction and stiffness reduction is performed with the aim of verifying if a unique factor is adequate to represent both the stiffness and the strength decrease due to openings, as schematically depicted in Fig. 3.

By means of all the available data, a model for the reduction factor, which takes into account the main parameters involved, is calibrated and compared with other models proposed by different researchers. Finally, additional numerical analyses are performed to evaluate the accuracy of the proposed relationships.

3.1 Tests and numerical simulations used in the study

The experimental and numerical analyses used in the study are listed in Table 1. The data base includes different types of frame-infill systems. Both RC frames and steel frames are considered and the mechanical characteristics of infills and the boundary conditions between frames and infills include a large set of situations. This circumstance reflects the great variability in the materials and in the construction techniques adopted in different countries. Furthermore, the type of tests and the related results are not uniform; in many cases, the ultimate strength is evaluated, whereas few studies are conducted in the linear range and therefore only the initial stiffness or the stress level at a given horizontal load are assessed. As expected, the use of different types of infill-frame systems results in a large scatter of the data. Usually, equations proposed in the literature for the evaluation

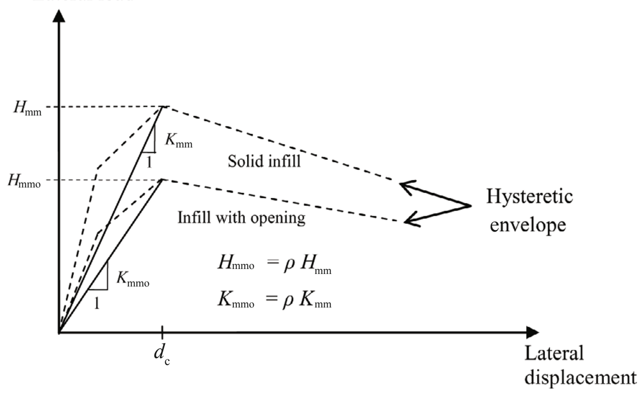


Fig. 3 Schematic representation of the infill lateral load-displacement envelope (not drawn to scale). The hysteretic envelops represent the envelopes of lateral load displacement cycles (e.g. see Fig. 15)

Table 1 Analytical and experimental tests considered in the study

Ref.	N	Range of α_a (%)	Range of α_l (%)	Range of α_h (%)	Frame	Infill	Reinforcement [1]	Type of test
Polyakov (1956)	8	6.00 35.30	8.33 54.70	58.00 66.00	4-hinged steel	Brick masonry	5 NR 3 R	Experimental Scale 1:1, 1:2
Benjamin and Williams (1958)	1	11.44	32.23	35.55	4-hinged steel	Brick masonry	NR	Experimental Scale 1:1
Sachanski (1960)	7	10.96 23.03	18.42 50.00	41.67 83.33	RC	Brick masonry and lightweight concrete	NR	Experimental Scale 1:1, 1:2
Simonici (1978)	1	40.13	55.00	73.00	RC		PR	Experimental Scale 1:2
Liauw (1979)	8	14.16 21.24	20.00 30.00	70.80	Steel	Micro-concrete with and without shear connectors	PR	Experimental Scale 1:10
Utku (1980)	16	4.00 36.00	10.00 60.00	20.00 80.00	-		NR	Numerical
Srinivasa Rao *et al*. (1982)	5	11.11 22.22	33.33	33.33 66.67	RC		PR	Experimental Scale 1:3
Dawe and Young (1985)	3	17.46	22.22	78.57	Steel	Concrete masonry blocks	PR	Experimental Scale 1:1
Decanini *et al*. (1985)	5	12.38	36.36	34.05	RC	Confined masonry	R	Experimental Scale 1:1
Casal (1986)	6	11.11 25.93	33.33 55.55	33.33 77.76	RC		NR	Numerical
Cassis *et al*. (1986)	4	2.96 13.34	13.04 32.61	20.73 40.91	RC		NR	Numerical
Imai (1989)	4	2.24 3.73	20.17 33.61	11.11	-	Reinforced masonry wall, fully grouted concrete blocks	R	Experimental Scale 1:2
Zarnic (1990)	4	22.20 25.00	25.00 39.00	56.92 100.00	RC		2 R 2 PR	Experimental
Decanini *et al*. (1994)	15	4.00 24.00	20.00 60.00	20.00 60.00	RC		14 R 1 PR	Numerical
Pires *et al*. (1998)	3	13.33	21.67	61.54	RC	Brick masonry	NR	Experimental Scale 2:3
Raj (2000)	4	11.25	25.00	45.00	RC	Brick masonry	PR	Experimental Scale 1:2.7
Asteris (2003)	7	4.00 60.00	20.00 77.46	20.00 77.46	RC	Anisotropic material	NR	Numerical
Yáñez *et al*. (2004)	12	13.82 38.10	15.00 63.51	56.25 100.00	RC	Confined masonry, concrete blocks and clay bricks	R	Experimental Scale 1:1
Astroza and Ogaz (2005)	4	19.41 23.00	34.12 38.18	50.00	RC	Confined masonry, clay bricks	R	Experimental Scale 1:1
Singh *et al*. (2006)	2	11.86	43.93	26.99	RC	Confined masonry	1 R 1 PR	Experimental Scale 1:5
Anil and Altin (2007)	5	14.43 50.00	25.00 50.00	57.73 100.00	RC	RC infill panel anchored to the frame	R	Experimental Scale 1:3
Mohebkhah *et al*. (2007)	3	9.92 25.40	27.78 44.44	35.71 57.14	Steel	Concrete blocks	NR	Numerical
Mondal and Jain (2008)	12	3.33 20.00	10.00 40.00	16.67 66.67	RC		NR	Numerical
Kakaletsis and Karayannis (2008)	4	10.31 20.00	25.00	41.25 80.00	RC	Brick masonry	NR	Experimental Scale 1:5
Tasnimi and Mohebkhah (2011)	4	6.15 24.95	22.12 53.10	27.78 80.56	Steel	Solid clay brick masonry	R	Experimental Scale 2:3

(1) Reinforcement condition around openings: NR = unreinforced opening, PR = partially reinforced opening, R = reinforced opening

of stiffness and strength of infills are based on a single experimental or numerical campaign in which few parameters change, e.g., the steel type, or the mechanical characteristics of masonry, etc. The type of frame (e.g., steel or RC), the type of the test (e.g., cyclic, monotonic, dynamic, pseudo-dynamic, numerical, etc.) and the load protocol of the tests (in the experimental campaign) does not change within each study. Therefore, the results and the related proposed formulations should be applied under the same conditions of the test (same kind of frame, type of test, etc.). The aim of the present study is to propose general relationships that take into account the uncertainties arising from the aforementioned different conditions.

The dimensions of the openings in terms of α_a, α_l and α_h, the type of frames and infills considered in each campaign, the presence of reinforcing elements around openings and the type of analysis (experimental or numerical) are reported in Table 1. Considering the whole of the available analyses, the data set consists of 84 experimental tests and 63 numerical tests. The numerical analyses are independent from the experimental ones, because they are not based on experimental results already included in the data base. The distribution of the data according to the opening sizes is shown in Fig. 4. In 87% of the cases examined, the ratio between the opening area and the infill area (α_a) is in the range 0.02−0.25 and in the 84% of the samples the ratio between the opening width and the infill width (α_l) is in the range 0.02−0.40. The distribution according to the ratio between opening height and infill height (α_h) is somewhat regular.

The presence of reinforcing elements around the openings was found to be one of the factors that most affects the seismic behavior of infills with openings (Decanini *et al.*, 1994), noticeably influencing both the strength and stiffness reduction and the crack pattern in the wall. The kind of reinforcement cannot be traced back to a strict classification. However, taking into account the most frequent situations, the following classification have been adopted here:

Unreinforced opening (NR): the opening is not confined by lintel bands or steel reinforcement.

Partially Reinforced opening (PR): at least the upper edge of the opening is reinforced by a lintel band.

Reinforced opening (R): at least two opposite edges of the opening are reinforced by two lintel bands or by a lintel band and a steel bar; if the wall is reinforced the opening is considered reinforced.

Some of these situations are shown schematically in Fig. 5. Considering the whole data set, the openings are unreinforced in 68 tests, are partially reinforced in 25 tests and reinforced in the remaining 54 cases.

3.2 Relation between strength reduction and stiffness reduction

In the following discussion, the relationship between strength reduction and stiffness reduction is analyzed to assess if a unique factor is suitable to evaluate both the strength reduction and the stiffness reduction.

In 63 of the considered tests, both the stiffness (initial and/or secant) and the ultimate strength were available. For these tests, the relationship between the stiffness reduction and the strength reduction due to the openings is analyzed by means of the ratio r:

$$r = \frac{\rho_{\text{stiffness}}}{\rho_{\text{strength}}} \tag{9}$$

where $\rho_{\text{stiffness}}$ is the actual stiffness reduction factor, and ρ_{strength} is the actual strength reduction factor due to openings.

The mean values of r are reported in Table 2. A clear trend is not traceable; however it can be noted that:

The mean values of r range between 0.60 and 1.46 and from 0.59 and 1.28 for steel frames and for RC frames (both confined masonry and infilled frame), respectively.

From the tests performed by Liauw (1979), it appears that the presence of shear connectors between frame and infills produces an increase of r, which means that the stiffness of the system is affected more than the strength

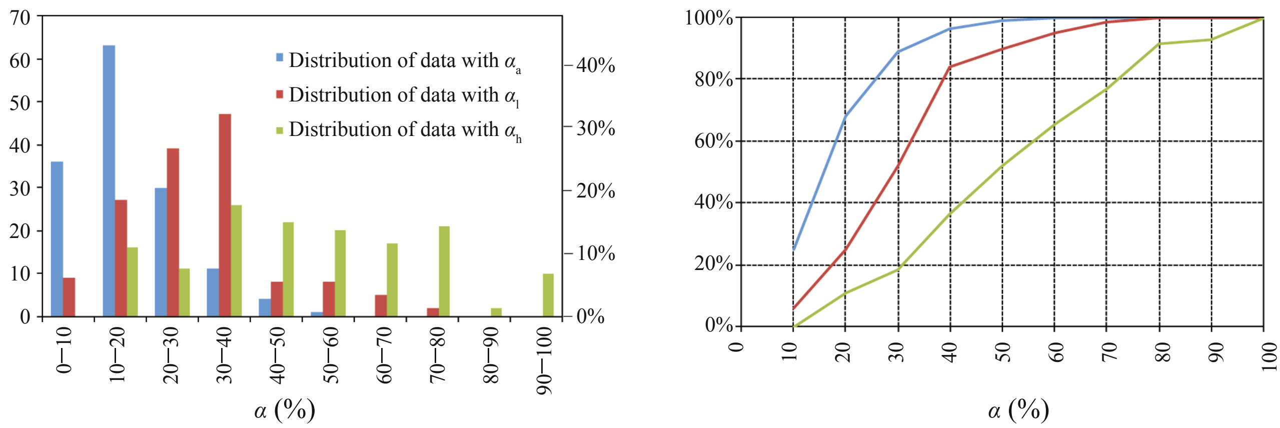


Fig. 4 Distribution of the data as a function of the opening dimensions: α_a = ratio between opening area and infill area, α_l = ratio between the opening width and the infill width, α_h = ratio between opening height and infill height. On the left: number of tests (left axis) and number of tests in % (right axis). On the right: cumulative distribution

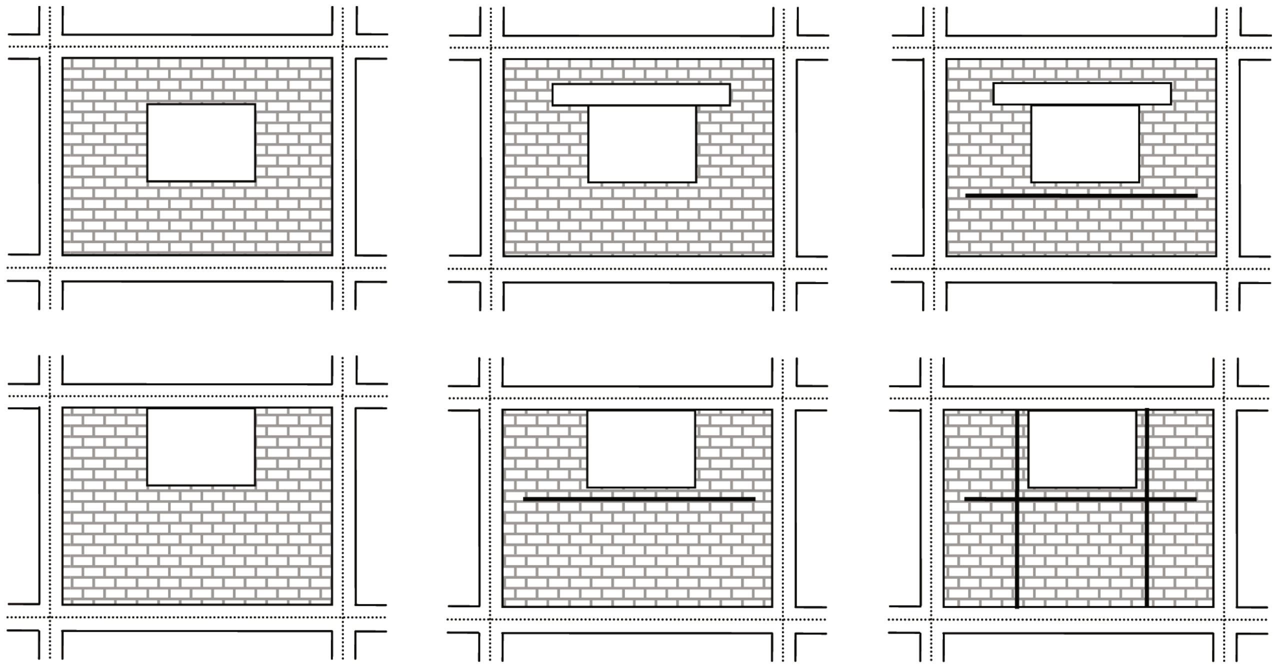


Fig. 5 Examples of unreinforced, partially reinforced and reinforced openings, reworked after Decanini *et al.* (1994)

by the enhancement of the boundary condition.

In the tests performed by Decanini *et al*. (1985), r is greater for the hollow brick masonry than for the solid brick masonry.

Considering the whole data base, the mean values and the standard deviations of the ratio between the stiffness reduction and the strength reduction are 0.98 and 0.26 when considering the secant stiffness and 0.97 and 0.28 for the initial stiffness, respectively.

Moreover, r is not significantly affected by the opening size. From the observations, the use of a unique reduction factor, ρ, seems acceptable for the evaluation of both the stiffness and the strength of the frame-infill systems with openings. However, note that this assumption is derived from the approximation of an average trend and is not always satisfied. In fact, it assumes that the deformation at the attainment of the ultimate strength is the same in the panels with and without opening; however, this condition was not always achieved in the experimental tests.

3.3 Opening reduction factor

The influence of the opening size on the reduction factor (ρ) is shown in Fig. 6, where ρ is reported versus α_a, α_l and α_h. A certain correlation between the reduction factor and the opening area is found; the influence of the opening length is still present, even though less marked, while the parameter α_h alone is not well correlated to the reduction factor ρ. As expected, the scatter of the data is noticeable. The scatter decreases when grouping the samples according to the reinforcing conditions around the openings (Fig. 7); this result suggested evaluating different equations according to such conditions.

The lack of a clear trend in the relationship between the height of the opening and the reduction factor ρ, suggested including only the length and the surface of openings in the proposed equation. However, it is noted that among the three parameters α_a, α_l and α_h, only two are independent (Fig. 2). Among different functions - linear, polynomial, exponential, and logarithmic - the following exponential equation is adequate to fit the observed data:

$$\rho = a\exp\left(b\alpha_{\mathrm{a}}\right) + c\exp\left(d\alpha_{\mathrm{l}}\right) \pm \sigma\,\varepsilon \tag{10}$$

The parameters a, b, c, and d, which are evaluated by means of regression analyses, depend on the reinforcing conditions (Table 3), σ is the standard deviation and ε is 0 for the mean value and 1 for the mean value plus or minus one standard deviation. The mean, the standard deviation and the coefficient of variation of the ratio between the prediction $\hat{\rho}_i$ and the actual value ρ_i, are reported in Table 4 for data grouped according to: i) reinforcing condition; ii) type of analysis (experimental or numerical) and iii) surrounding frame (RC or steel). In the same table, the standard error $\hat{\sigma}$, given by the following equation, is reported:

$$\hat{\sigma} = \sqrt{\frac{\sum_{i=1}^{n}\left(\rho_i - \hat{\rho}_i\right)^2}{n - n_0}} \tag{11}$$

where n is the number of data points and n_0 = 4, being the number of equation parameters to be estimated, i.e. a, b, c, and d.

The mean of the ratio between predictions and

Table 2 Mean values of the ratio *r* = stiffness reduction/strength reduction

Ref.	*N*	*r* (secant stiffness)(1)	*r* (initial stiffness)	Frame-infill system (2)
Polyakov (1956)	8	1.30	-	Steel frame, solid bricks
Benjamin and Williams (1958)	1	1.00	-	Steel frame, solid bricks
Liauw (1979)	4	0.78	0.60	Steel frame, micro-concrete without shear connectors
Liauw (1979)	4	0.97	1.46	Steel frame, micro-concrete with shear connectors
Srinivasa Rao *et al*. (1982)	5	0.90	0.95	RC frame, solid brick
Dawe and Young (1985)	3	1.15	1.30	Steel frame, concrete blocks
Decanini *et al*. (1985)	3	0.59	0.82	RC frame, solid brick masonry (confined)
Decanini *et al*. (1985)	2	0.78	1.28	RC frame, hollow brick masonry (confined)
Raj (2000)	4	-	0.70	RC frame, brick
Yáñez *et al*. (2004)	6	-	0.85	RC frame, hollow concrete blocks masonry (confined)
Yáñez *et al*. (2004)	6	-	0.93	RC frame, hollow brick masonry (confined)
Astroza and Ogaz (2005)	4	0.77	0.88	RC frame, hollow brick masonry (confined)
Singh *et al*. (2006)	2	0.99	-	RC frame, brick masonry (confined)
Mohebkhah *et al*. (2007)	3	0.70	0.78	Steel frame, concrete blocks masonry
Kakaletsis and Karayannis (2008)	2	-	0.85	RC frame, brick masonry
Kakaletsis and Karayannis (2008)	2	-	0.91	RC frame, vitrified ceramic brick masonry
Tasnimi and Mohebkhah (2011)	4	1.38	1.42	Steel frame, brick masonry
Mean value		0.98	0.97	
Standard deviation		0.26	0.28	

(1) The secant stiffness is generally measured at the maximum strength

(2) If no specified the brick are clay brick and the frame-infill system is not a confined masonry

actual values (Table 4) is very close to unity in all the situations investigated, indicating that on average, the proposed equation fits the observed data very well. Considering the data grouped according to the opening reinforcement, the standard error ranges between 0.113 (reinforced opening) and 0.156 (partially reinforced opening); i.e., the residuals are lower for the reinforced opening indicating that in this case, Eq. (10) gives a fairly good estimate of the reduction factor. For the unreinforced and partially reinforced openings, both the standard deviation and the standard error are higher, indicating a greater dispersion of the data and higher differences between actual reduction factors and values estimated by Eq. (10). Note that in the case of partially reinforced openings, the residuals are generally lower than 0.16 with the exception of four cases, which may be considered intermediate between NR and PR cases due to the short length of the lintel band above the opening (Raj, 2000). The last two rows of Table 4 show that the proposed formula fits the observed values better for the steel frames than for the RC frames, since the standard errors are equal to 0.124 and 0.152, respectively.

The proposed equation reflects different experimentally observed aspects: the reduction factor increase due to the presence of reinforcing elements around the opening especially when a complete reinforcement is provided; the influence of α_a and α_l decreases when the level of opening reinforcement increases; when an unreinforced opening with an area greater than 40% of the infill area is present, the contribution of the infill is negligible while if the opening is completely reinforced, the reduction factor is always greater than 0.4.

The limited number of tests with partially reinforced openings and the small differences found with the unreinforced cases suggest the use of the same coefficients for both situations. In this case, the regression analysis made considering 93 tests (NR + PR)

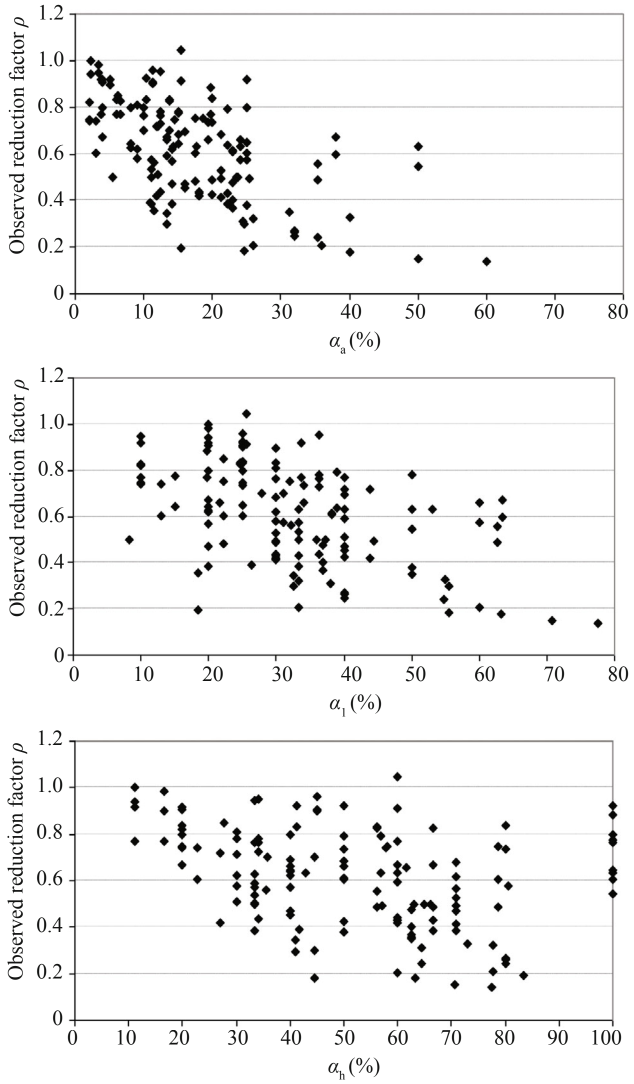


Fig. 6 Observed reduction factors versus α_a, α_l, and α_h, whole data set

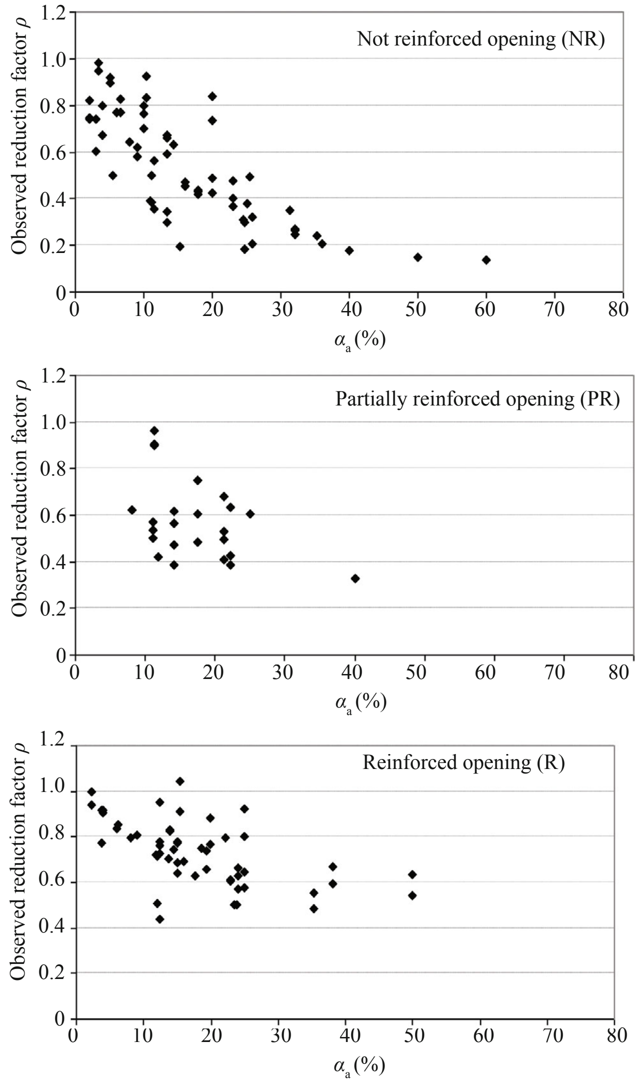


Fig. 7 Observed reduction factors versus α_a, samples grouped according to the reinforcing conditions

Table 3 Parameters for the reduction factor (Eq. (10))

Reinforcement around opening	*a*	*b*	*c*	*d*
NR	0.55	-0.035	0.44	-0.025
PR	0.58	-0.030	0.42	-0.020
R	0.63	-0.020	0.40	-0.010

gives the same equations parameters found for the NR openings (Table 3). The corresponding mean, standard deviation, coefficient of variation and standard error are reported in Table 4.

Taking into consideration the scatter of the data, the equations proposed for the reduction factor are:

NR and PR openings

$$\rho = 0.55\exp\left(-0.035\alpha_{\mathrm{a}}\right) + 0.44\exp\left(-0.025\alpha_{\mathrm{l}}\right) \pm 0.284\varepsilon \tag{12}$$

R openings

$$\rho = 0.63\exp\left(-0.020\alpha_{\mathrm{a}}\right) + 0.40\exp\left(-0.010\alpha_{\mathrm{l}}\right) \pm 0.177\varepsilon \tag{13}$$

In Fig. 8, the above equations are plotted as a function of α_a for different values of α_l. The observed values are reported as well. Each curve is depicted in its range of validity; for example, the curve $\alpha_l = 10\%$ is shown only for $2\% \le \alpha_a \le 10\%$ because values smaller than 2% imply unrealistically small values of α_h while values of α_a greater than 10% imply that α_h is greater than 100% (opening height greater than wall height).

Table 4 Statistical data: number; mean, standard deviation (σ) and coefficient of variation (CV) of the ratio between the prediction and the actual value; standard error ($\hat{\sigma}$)

Opening reinforcement	N	Mean	σ	CV	$\hat{\sigma}$
NR	68	1.083	0.288	0.266	0.136
PR	25	1.045	0.223	0.213	0.156
R	54	1.022	0.177	0.173	0.113
NR + PR	93	1.055	0.284	0.269	0.144
Type of analysis					
Experimental	84	1.002	0.241	0.241	0.150
Numerical	63	1.096	0.250	0.228	0.108
Type of frame					
RC	88	1.019	0.271	0.266	0.152
Steel	29	1.070	0.225	0.210	0.124

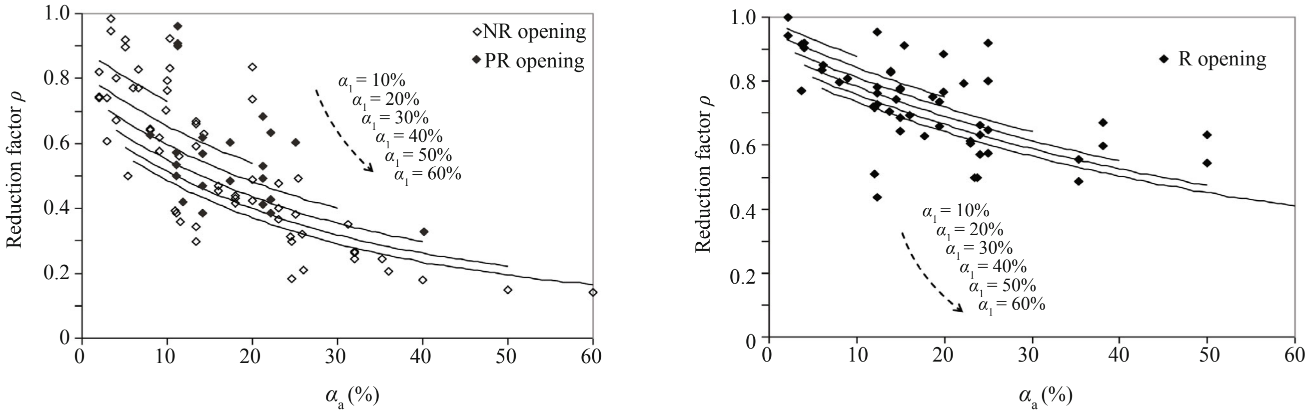


Fig. 8 Proposed reduction factor as a function of α_a for different values of α_l. Comparison with the observed values

The proposed reduction factors are compared with those reported in the NZSEE Recommendations (2006) and to those suggested by Polyakov (1956), Sachanski (1960), Imai (1989), Al-Chaar (2003), Mondal and Jain (2008) and Tasnimi and Mohebkhah (2011). To this aim, the reduction factors proposed by the above researchers are evaluated (using Eqs. (1), (2), (3), (4), (6), (7) and (8)) and compared with the actual values derived from the available experimental and numerical analyses. For each model, only the pertinent tests are considered; for example, for the model of Poliakov, only the tests with $\alpha_h \leq 65\%$ and $\alpha_a \leq 60\%$ are taken into consideration in the comparison.

For each model, the mean values and the standard errors of the ratio $\hat{\rho}_i/\rho_i$ (prediction/actual value) are calculated and shown in Fig. 9 and Fig. 10, respectively. Considering the total number of analyses, different models are suitable for evaluating the average reduction factor; a very good estimation (the mean is close to 1.0) is obtained with the equations of Sachansky (1960) and Mondal and Jain (2008) and reasonable predictions are also given by the NZSEE Recommendations (2006) and by Imai's model. The reduction factor proposed by Poliakov's model tends to underestimate the actual values, while that suggested by Al-Chaar overestimates the actual values. Grouping the data according to the opening reinforcing conditions, the comparison between predictions and actual values shows that the models by the NZSEE Recommendations (2006), Sachansky (1960) and Mondal and Jain (2008) are suitable for the unreinforced and partially reinforced openings (NR + PR) while they underestimate the reduction factor in the cases of reinforced openings. The models proposed by Tasnimi and Mohebkhah (2011) and by Al-Chaar (2003) give a good estimate of the mean reduction factor for reinforced openings.

The standard error, reported in Fig. 10, is generally greater than 0.15 and is higher for the reinforced openings. The model proposed in this study minimizes the errors and adequately approximate the mean reduction factor for both conditions of opening reinforcement.

In Fig. 11 and Fig. 12, the equations proposed by Imai (1989), Al-Chaar (2002), Asteris (2003), Mondal and Jain (2008), Tasnimi and Mohebkhah (2011) and

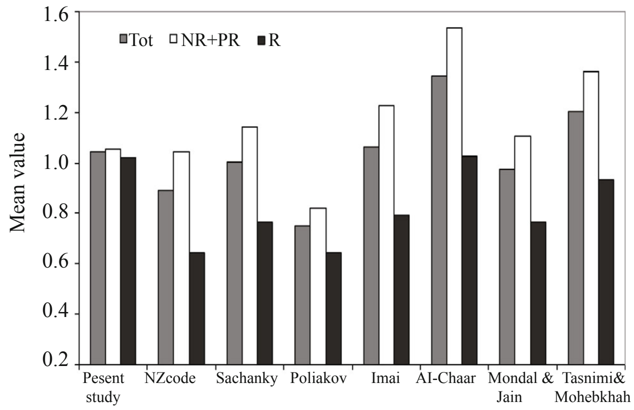


Fig. 9 Mean value of the ratio between prediction and actual value according to different models

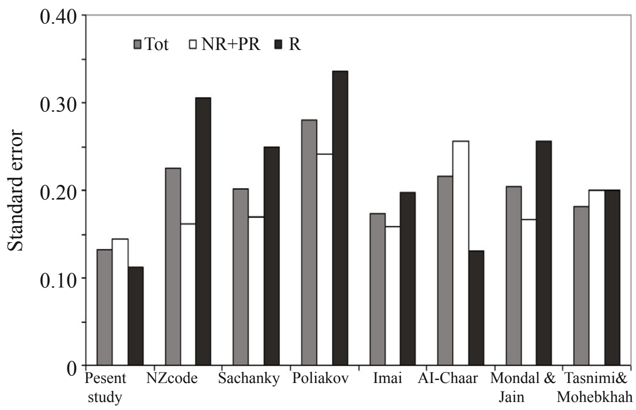


Fig. 10 Standard error of the ratio between prediction and actual value according to different models

NZSEE Recommendations (2006) are plotted together with the equation proposed in the present study. The model of Mondal and Jain gives a good approximation of the mean values in the range $0 \le \alpha_a \le 30\%$, while it underestimates the reduction factor for $\alpha_a > 30\%$; the Al-Chaar expression may give adequate predictions only for infills with reinforced openings. The parabolic expressions suggested by Imai and Tasnimi and Mohebkhah seem to represent the variation of the reduction factor with α_a better than the other models but does not consider the reinforcing condition around the openings. The NZSEE curve (Fig. 12) adequately represents, on average, the actual values in case of unreinforced conditions, while it underestimates the reduction factor for the reinforced openings. The linear expression proposed by Imai (Fig. 12) tends to overestimate the actual values, especially in the case of infills with unreinforced or partially reinforced openings. The equations proposed in the present study satisfactorily match the observed values due to the fact that, besides considering two different reinforcing conditions, they are functions of both the area and the length of the openings.

3.4 Verification of the proposed relationships

To verify the accuracy of the proposed relationships, additional numerical analyses based on a finite element model previously calibrated with experimental tests have been performed.

The analyses used one-bay, one-story frames (3 m tall, 4 m large) infilled with unreinforced masonry. Different situations have been analyzed considering two types of frames (RC and steel) and two types of masonry (hollow-clay-brick and solid-clay-brick with cement mortar). The panel has a central window or door opening. Six different lengths of the opening are considered (l_0 = 0.6-0.8-1.0-1.2-1.4-1.6 m); for each length, both the window configuration (h_0 = 1 m, models from W1 to W6) and the door configuration (h_0 = 2 m, models from D1 to D6) are analyzed (Table 5).

The compressive strength of the masonry is 2 MPa for the hollow-clay-brick masonry and 10 MPa for the solid-clay-brick masonry. The thickness of the panel is 12.5 cm in both cases. Concerning the RC frame, columns are 25 cm wide and 40 cm deep and the concrete has a cylindrical compressive strength of 21 MPa. For the steel frame members, W250×200×67 cross-sections are used, the yield strength is assumed to be equal to 355 MPa. The gravitational loads are due to a distributed load of 20 kN/m and the self weight. Both monotonic and cyclic analyses are performed with the finite element method code ADINA (2002). The *Load-Displacement-Control* method of analysis (Bathe and Dvorkin, 1983) with automatic step increment is used for the monotonic analyses. For the cyclic analyses, horizontal displacements are applied at the beam levels. The *concrete* type material (Bathe and Ramaswamy, 1979) is used to model the masonry because it allows its different behavior in tension and compression to be taken into account and can be used for materials that are characterized by a weak or even negligible tensile strength, to account for cracking once the tensile strength is attained. Columns and beam are modelled as linear elastic frames since preliminary analyses on the bare frames (both RC and steel) has shown that yielding occurs at displacement levels much greater than those corresponding to the attainment of the strength of the frame-infill system.

Examples of typical crack patterns that developed in the perforated panels (window and door) are reported in Fig. 13. In the model with the window, the cracks develop mainly at the opposite corners of the opening; whereas in the model with the door opening, cracks also appear at the base, indicating the development of a flexural behavior of the masonry pier.

The reduction factors are reported in Table 5. They are slightly influenced by the frame type, RC or steel, whereas they are more affected by the masonry type. As a matter of fact, the reduction factor is smaller in the case of hollow-clay-brick masonry. On the average, the differences between the two masonry types are about

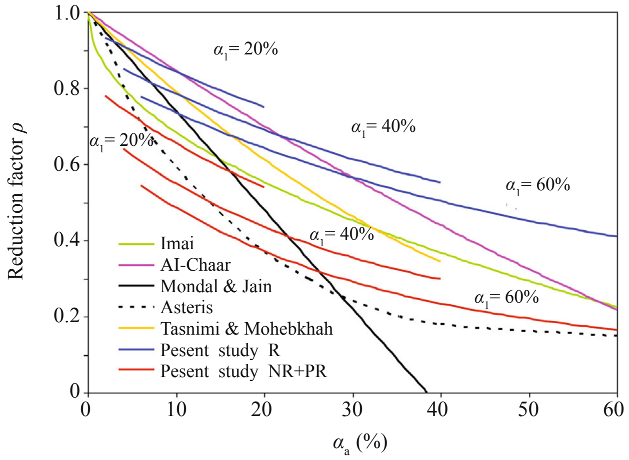


Fig. 11 Reduction factors proposed by Imai (1989), Al-Chaar (2002), Asteris (2003), Mondal and Jain (2008), Tasnimi and Mohebkhah (2011) and the present study

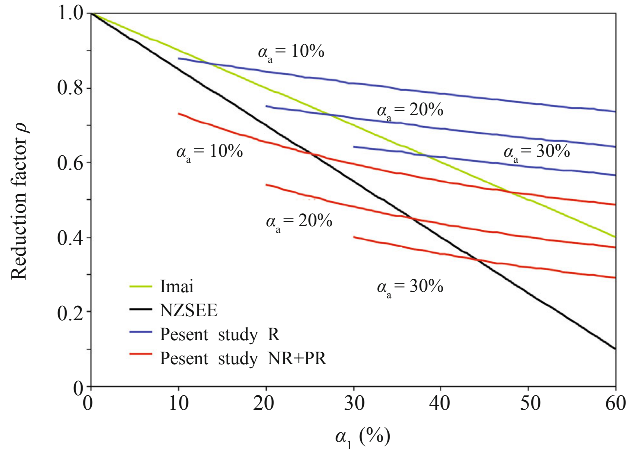


Fig. 12 Reduction factors proposed by Imai (1989), NZSEE Recommendations (2006) and the present study

Table 5 Reduction factors obtained from FEM analyses. Comparison with reduction factors estimated by means of the proposed equation

Model	l_0 (m)	h_0 (m)	α_l (%)	α_h (%)	α_a (%)	RC frame hollow brick		RC frame solid brick		Steel frame hollow brick		Steel frame solid brick	
						ρ_{num} (1)	$\rho_{pred.}$ (2) / ρ_{num}	ρ_{num} (1)	$\rho_{pred.}$ (2) / ρ_{num}	ρ_{num} (1)	$\rho_{pred.}$ (2) / ρ_{num}	ρ_{num} (1)	$\rho_{pred.}$ (2) / ρ_{num}
W1	0.6	1.0	15.0	33.3	5.00	0.73	**1.05**	0.79	0.96	0.74	1.03	0.79	0.97
W2	0.8	1.0	20.0	33.3	6.7	0.69	1.02	0.74	0.95	0.67	1.05	0.77	0.91
W3	1.0	1.0	25.0	33.3	8.3	0.64	1.02	0.69	0.94	0.62	1.04	0.70	0.92
W4	1.2	1.0	30.0	33.3	10.0	0.58	1.02	0.64	**0.93**	0.57	1.04	0.64	0.93
W5	1.4	1.0	35.0	33.3	11.7	0.53	1.04	0.59	**0.93**	0.52	1.06	0.61	**0.90**
W6	1.6	1.0	40.0	33.3	13.3	0.49	1.04	0.54	0.94	0.47	**1.08**	0.54	0.94
D1	0.6	2.0	15.0	66.7	10.0	0.65	1.06	0.69	1.00	0.63	**1.10**	0.70	0.99
D2	0.8	2.0	20.0	66.7	13.3	0.59	1.03	0.64	0.95	0.58	1.06	0.65	0.94
D3	1.0	2.0	25.0	66.7	16.7	0.54	1.01	0.58	0.93	0.52	1.04	0.59	0.92
D4	1.2	2.0	30.0	66.7	20.0	0.50	0.97	0.54	0.89	0.47	1.02	0.55	0.87
D5	1.4	2.0	35.0	66.7	23.3	0.45	0.95	0.49	0.87	0.42	1.02	0.49	0.87
D6	1.6	2.0	40.0	66.7	26.7	0.41	**0.93**	0.45	**0.84**	0.39	0.97	0.44	**0.86**

(1) Evaluated from the numerical analyses. (2) Evaluated by means of the proposed equation.
Minimum and maximum values of the ratio ρ_{pred} / ρ_{num} are reported in bold type character.

8% and 12% for the RC frames and for the steel frames, respectively. Such differences are slightly affected by the opening dimensions. In Table 5, the ratio between the reduction factors evaluated by means of the proposed equation and those obtained from the numerical analyses is reported as well. The prediction for the window opening models is very good; differences between evaluated (numerical analyses) and predicted (proposed equation) values are smaller than 7% and 10% in the cases of RC frame and steel frame, respectively. For the door opening models, differences between numerical and predicted reduction factors are slightly higher, especially in the case of solid-clay-brick masonry, in such situations, the difference increases as the opening length increases with a maximum difference of about 15% for the D6 models (for both RC and steel frames with solid-clay-brick masonry).

As mentioned, the proposed equation (Eq. (10)) does not take into account either the type of frame or the masonry characteristics. The numerical analyses performed here show that there are some differences when considering different types of masonry whereas the influence of the frame type is smaller. The latter conclusion is consistent with the findings reported in other studies, as shown in Fig. 14, where the reduction factors for RC frames and steel frames are reported. Concerning the influence of the wall type, which was specifically investigated by Sachanski (1960) and Yáñez *et al*. (2004): Sachanski (1960) found that the specimens infilled with lightweight concrete behave substantially

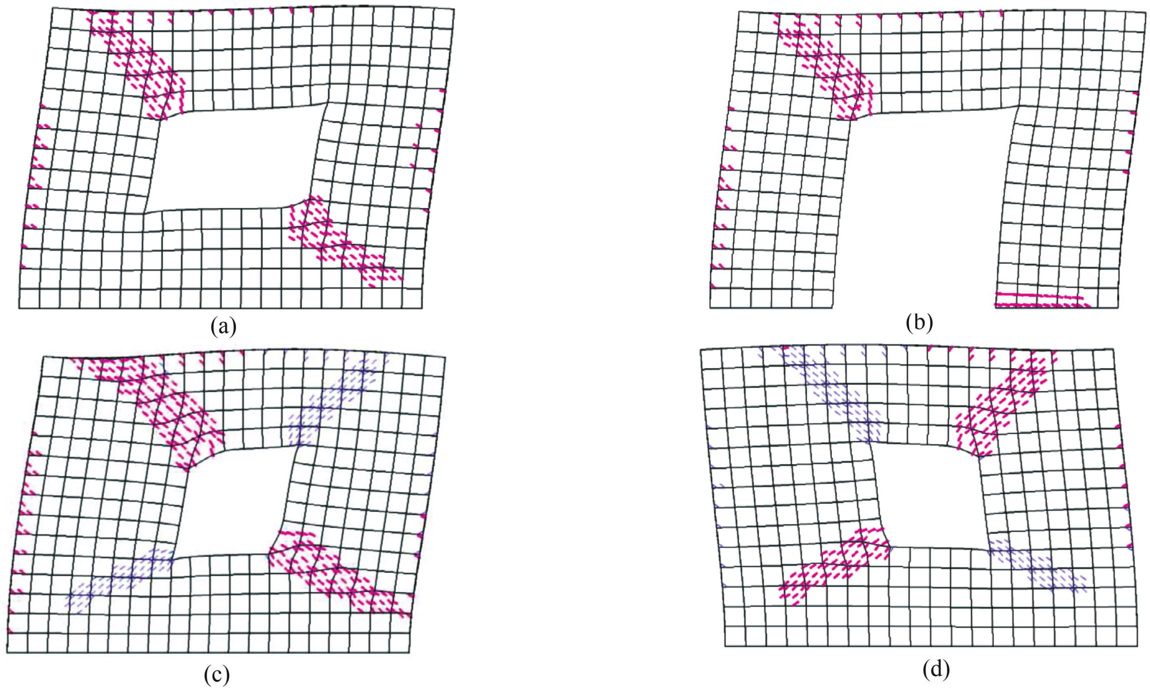


Fig. 13 Typical deformed shapes and crack patterns: (a) model W6; (b) model D6; (c) and (d) model W3 cyclic analysis. Open cracks in pink colour, closed racks in light blue. Displacement are not to scale

like the ones with brick masonry; in the experimental campaign performed by Yáñez *et al*. (2004), the reduction factor was lower (10% on the average) for the hollow-clay-brick masonry than for the concrete masonry; i.e., the effect of the opening is smaller in the case of stiffer masonry, which is consistent with the results of the numerical analyses performed here. However, the influence of the panel characteristics seems moderate, allowing the use of a unique expression for the opening reduction factor for different kind of infill walls.

4 Application of the proposed reduction factor

In this section, a numerical example of an application of the proposed reduction factors is presented. The

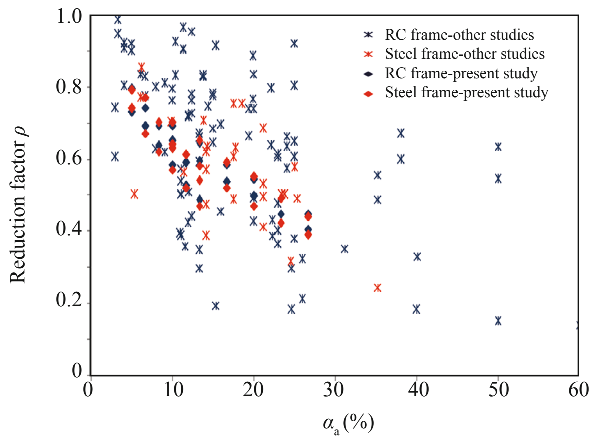


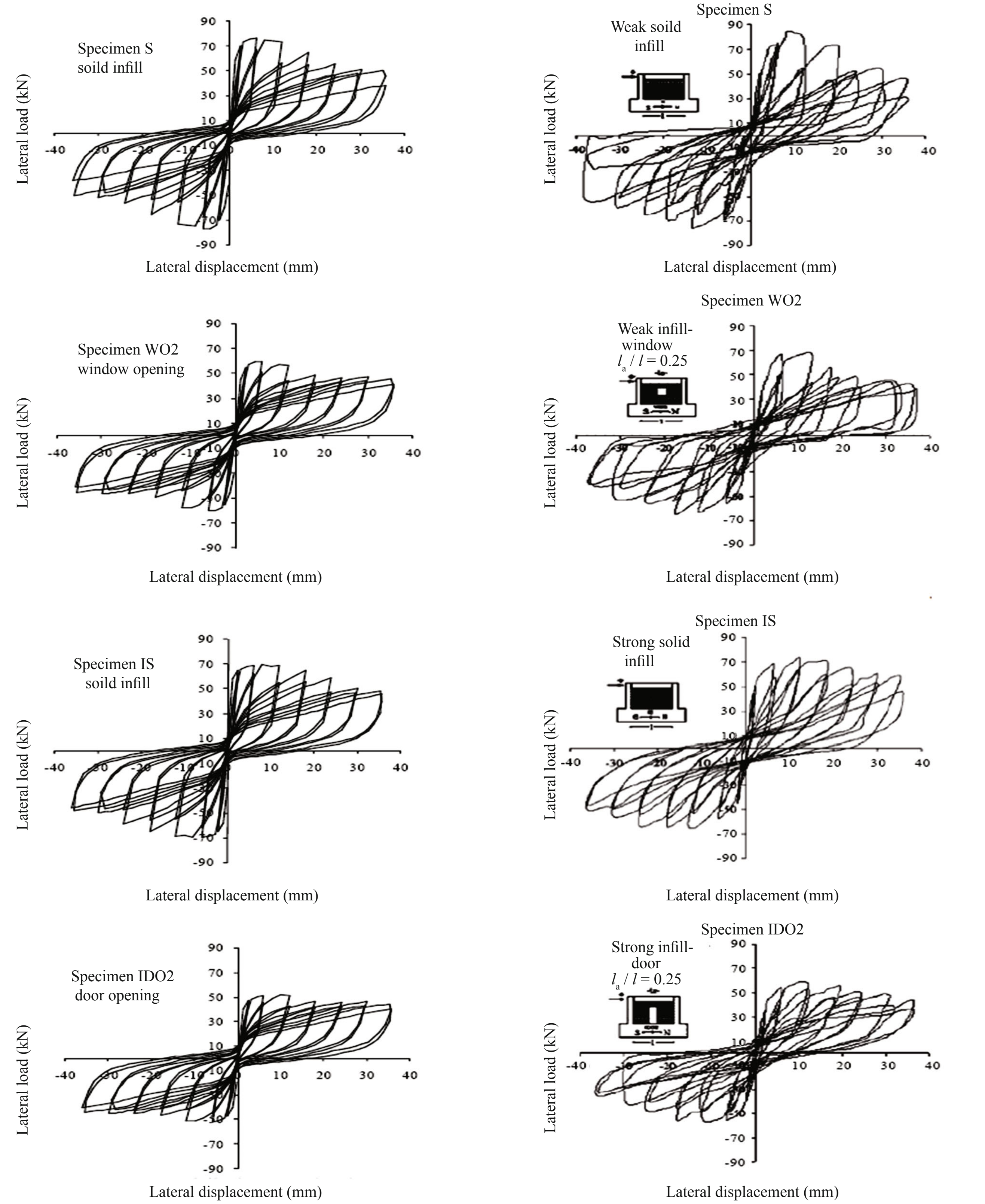
Fig. 14 Reduction factor for RC frames and steel frames. Comparison between the present study (numerical analyses) and other studies (both numerical and experimental, refer to Table 1)

equivalent strut method is used to reproduce the cyclic behavior of the infilled frame specimens experimentally tested by Kakaletsis and Karayannis (2008). Since the analyses by Kakaletsis and Karayannis have been included in the data base and employed for the derivation of the proposed expressions, this section is intended as an example of the application rather than a verification of such expressions.

The experimental campaign by Kakaletsis and Karayannis (2008) concerns seven 1/3-scale, single-story, single-bay RC frames, including the reference bare frame, two fully infilled frames, two infilled frames with central window opening and two with central door opening (Table 6). The openings are not reinforced. Two types of infill masonry are used, one made of common clay brick, and the other of vitrified ceramic brick. A typical mortar mix of cement, lime and sand was used for both masonry types. The loading sequence consists of full cycles of gradually increased displacement; two cycles were applied at each displacement level.

For the numerical analyses, the infills are modelled by means of two diagonal no-tension struts using the model by Decanini *et al*. (Decanini and Fantin, 1989; Bertoldi *et al*., 1993), which supplies the ultimate horizontal strength and stiffness corresponding to a state of steady cracking of the infill. The hysteretic model (Liberatore and Decanini, 2011) takes into account the degradation of stiffness in the unloading branch and the degradation of strength under displacement cycles of constant amplitude. The presence of the opening is taken into account by means of the reduction factor given by Eq. (10).

In Fig. 15 the lateral load-displacement curves of four of the analyzed models are compared with the experimental results. The strength, the lateral force measured at the first cycle of the maximum displacement, and the cumulative energy dissipation evaluated by means of the numerical analyses and measured during the experimental tests are reported in Table 7. The model underestimates the strength of the specimens with differences of about 11% both in the case of the window opening and door opening; whereas generally overestimates the lateral load measured at the maximum



(a) (b)

Fig. 15 Lateral load-displacement curves: a) numerical analyses, b) experimental tests by Kakaletsis and Karayannis (2008) (courtesy of the authors)

displacement, with a difference up to 16%. The cumulative energy dissipation, i.e., the total hysteretic energy dissipated at the end of each analysis, shows differences in the range of 6%–21%, with the exception of model DO2, which presents a larger divergence.

From Fig. 15 it appears that, as expected, the curves derived from the numerical analyses are more smoothed than the experimental ones and therefore they cannot perfectly match the observed behavior. Nevertheless, the model seems to fit the observed shapes reasonably well on the whole and is able to reproduce the stiffness and strength degradation that occurs at each second cycle of constant displacement amplitude. Both in the numerical models and in the specimens, a reduction of energy dissipation capacity due to the opening is observed (Table 7). Both in the numerical analyses and in the experimental tests, such reduction is substantially independent on the type of masonry (clay brick or vitrified ceramic brick masonry) in the cases with a window opening, whereas a greater reduction is observed in the experimental tests with the door opening.

Table 6 Specimens tested by Kakaletsis and Karayannis (2008)

Model	Type	Masonry type	α_a (%)	α_l (%)
B	Bare	–	–	–
S	Fully infilled	Clay brick	–	–
IS	Fully infilled	Vitrified ceramic	–	–
WO2	Window opening	Clay brick	10.3	25
IWO2	Window opening	Vitrified ceramic	10.3	25
DO2	Door opening	Vlay brick	20	25
IDO2	Door opening	Vitrified ceramic	20	25

Table 7 Comparison between numerical (present study) and experimental (Kakaletsis and Karayannis, 2008) results

Spec.	Strength (1) (kN)			Lateral load at max displacement(1) (kN)			Cumulative energy dissipation (2) (kN·mm)		
	Num.	Exp.	Diff (%)	Num.	Exp.	Diff (%)	Num.	Exp.	Diff (%)
S	77	79	-2	50	49	4	14209	13101	8
WO2	60	67	-11	46	41	12	12715	11932	7
DO2	55	62	-11	44	38	16	12276	8498	44
IS	69	68	2	49	55	-12	14378	11834	21
IWO2	55	63	-12	45	46	-2	12440	11740	6
IDO2	52	58	-11	44	40	10	12187	10636	15

(1) Evaluated as the average of positive and negative peak values. (2) Evaluated at the end of the analyses.

5 Summary and conclusions

In the present study, the effect of openings on the lateral stiffness and strength of infilled frames is studied by means of about 150 numerical and experimental tests. The study highlighted that the area and the width of the opening and the reinforcing conditions around the opening, for example the presence of lintel bands or steel reinforcements, significantly affect the seismic behavior of infill-frame systems. The influence of the position of the opening within the panel has not been specifically analyzed here. However, it is worthwhile to point out that openings located in a corner of the panel may produce unfavorable effects, like the formation of short columns in the frame. In seismic areas, openings in the corners should be avoided.

An empirical equation (Eq. (10)) is proposed to take the influence of central openings in infill masonry walls into account in the overall strength and stiffness of the walls. The reduction factor is expressed as a function of the area and the width of the opening. The coefficients of the equation (Table 3) depend on the reinforcing conditions around the opening. The proposed expression is intended to be used with diagonal no-tension strut models and can be adopted both in pushover analyses and cyclic or dynamic analyses. Note that the strut model does not represent the actual stress distribution in the panel when large opening are present, but is a way to take the role of the infill panel into account in the global behavior of the frame-infill system. Alternative methods are the finite element approach, based on a finite element representation of the infill, and the multi-struts approach, based on the use of different struts around the opening. The use of such methods is somewhat complex due to the large amount of information required. The strut model approach is simpler for use in practical applications and, provided that the mechanical and geometrical characteristics of the strut are properly calibrated, it may adequately represent the increase of stiffness and strength due to infills, both solid and perforated.

The proposed equation reflects different aspects that were observed in the experiments: the reduction factor increases when reinforcing elements are present around the opening; the influence of the opening size decreases when the level of opening reinforcing increases; and when an unreinforced opening with an area greater than 40% of the infill area is present, then the contribution of the infill is negligible whereas if the opening is reinforced, the reduction factor is always greater than 0.4.

To verify the accuracy of the proposed relationship, additional numerical analyses based on a finite element model of the infill and the frame have been performed on one-bay, one-story RC and steel frames with window and door openings considering two types of masonry (hollow-clay-brick and solid-clay-brick). The reduction factors obtained were influenced more by the masonry type than by the frame type. A very good prediction is given by the proposed equations for all the models with window openings. For the door opening models, differences between numerical and predicted reduction factors are slightly greater but do not exceed 15%.

The proposed reduction factors are compared with those reported in the NZSEE Recommendations and those suggested by different authors. Some of these equations satisfactorily match the observed values for unreinforced openings but underestimate the reduction factors for reinforced openings.

Finally, an example of an application of the opening reduction factors is presented. For this purpose, some experimental tests on infilled frames with and without openings available in the literature were selected. The cyclic behavior of the test specimens was reproduced by modelling the masonry infills as diagonal no-tension struts and using the proposed reduction factors for the infills with openings. As expected, the numerical analyses and the experimental tests do not match perfectly; however, the model reproduces the strength, stiffness and energy dissipation capacity degradation due to the presence of openings fairly well.

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