An innovative Model for the In-Plane Nonlinear Analysis of Confined Masonry and Infilled Frame Structures

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SUMMARY:
It has been established that the design criteria and construction techniques for confined masonry walls and reinforced concrete (RC) frames with masonry infills are distinctly different. However, the response of these structural systems subjected to in-plane seismic loading is somewhat similar. In both cases, the behavior is mainly controlled by the complex nonlinear response of the masonry wall panels and the surrounding RC elements.
This paper presents an innovative model for the nonlinear analysis of confined masonry and infilled RC frame structures. The model represents a masonry panel using six strut members, which are located in the diagonal direction of the panel, whereas the RC members are represented with a column macro-element. The axial strength of masonry struts is determined according to a general failure theory, by considering the strut inclination and the following failure modes: sliding shear, diagonal tension, and compression failure.
Keywords: confined masonry, infilled frame, macroscopic model, strut mode, nonlinear analysis.

1. INTRODUCTION

Masonry construction is widely used all around the world for residential, commercial and industrial buildings. This material provides subdivision of space, fire protection, thermal and acoustic insulation, durability and aesthetic appeal; many architects value its color, shape and texture. Different construction systems are used for masonry buildings, however this paper focuses on the behaviour and modelling of confined masonry walls and reinforced concrete (RC) frames with masonry infills.

Confined masonry construction is formed by masonry walls which are surrounded by a reinforced concrete frame. The frame is cast after the construction of the walls in order to assure adequate bond between both parts of the structure. This system is applied in lower rise buildings, usually up to three stories, and is widely used in seismic regions of Latin America and Asia. Experience obtained from past earthquakes and experimental results indicate that confined masonry, if properly built, exhibits an adequate seismic response (Brzev, 2008). Consequently, it represents a good alterative in those seismic regions where masonry is widely used due to economical or traditional reasons. For this reason, the Confined Masonry Network was created in 2008, with the main objective of promoting the use of this system in developing countries. The Network currently has financial sponsorship from Risk Management Solutions and is supported administratively by the World Housing Encyclopedia of the Earthquake Engineering Research Institutehttp (http://www.confinedmasonry.org/). Recently, the Network has published the Seismic Design Guide for Low-Rise Confined Masonry Buildings (Meli et al., 2011).

Infilled frame structures have been used since the beginning of the 20th century for low and medium-height buildings. In this case, the sequence followed for the construction is different to that of confined masonry. The masonry infill is built after the frame and, consequently, the shrinkage of the masonry or defects due to inaccurate workmanship usually results in an initial lack of fit. Structural engineers have largely ignored the influence of the masonry panels in the analysis and design of infilled frames, even though unfavorable consequences may occur as result of this criterion. Masonry panels are very stiff, even if the thickness is small, and they can alter drastically the expected response of the structure. The reluctance to consider the contributions of the masonry infills has been due to the inadequate knowledge concerning to the composite behaviour of infilled frames, and to the lack of practical methods for predicting the stiffness and strength. Furthermore, most of the computer programs commonly used by designers do not provide some rational and specific elements for modelling the behaviour of the masonry infills. Infilled frames are scarcely used now for new buildings, however, there is a large stock of existing constructions in different regions of high seismicity that requires evaluation and eventual retrofitting (Galli, 2006; Ozden et al., 2011; Tasligedik et al., 2011).

It has been established that the design criteria and construction techniques for confined masonry walls and RC frames with masonry infills are distinctly different. However, in the author´s opinion, the response of these structural systems subjected to in-plane seismic loading is somewhat similar. In both cases, the behaviour is mainly controlled by the complex nonlinear response of the masonry wall panels and the surrounding RC elements, consequently both structural systems can be analysed with similar models (even though the parameters of the model need to be adjusted for each case).

This paper is divided into two parts. The first section describes the structural behaviour of confined masonry and infilled frames based on experimental data and computer results obtained from finite element models. Particular importance is given to investigate the influence of the conditions of panel-frame interface on the structural response. This description is focused on the in-plane behaviour of both structural systems; the discussion about the out-of-plane behaviour, despite its importance for seismic analysis and design, is out of the scope of this paper. In the second part, an innovative macroscopic model for the nonlinear analysis of confined masonry and infilled RC frame structures is proposed. The model represents a masonry panel using six strut members, which are located in the diagonal direction of the panel, whereas the RC members are represented with a column macro-element. The axial strength of masonry struts is determined according to a general failure theory, by considering the strut inclination and the following failure modes: sliding shear, diagonal tension, and compression failure. The hysteretic response of masonry has been taken into account through a refined model, which was carefully calibrated based on the results of experimental studies.

2. STRUCTURAL BEHAVIOUR OF CONFINED MASONRY AND INFILLED RC FRAMES

2.1. Structural response under in-plane lateral loading

Confined masonry and infilled frames are complex structures which exhibit a highly nonlinear inelastic behaviour. The most important factors contributing to this behaviour arise from material nonlinearity, such as, (i) cracking and crushing of the masonry panel, (ii) cracking of the concrete, (iii) yielding and buckling of the reinforcing bars and local bond slip, and (iv) degradation of the bond-friction mechanism and variation of the contact length along the panel-frame interfaces. The latter issue is very important in order to explain the differences in the structural response of confined masonry and infilled frames.

Before analysing in detail the behaviour of these structures, it must be noted that the number of influential parameters is rather large. Numerous combinations can be considered by changing the materials of the masonry panel and the frame, the constructive techniques used to build the structure and the interface conditions. Despite the numerous experimental results now available, it is rare to find in the literature comparable results from two or more distinct sources in order to draw definitive conclusions about the structural behaviour and the role of the different parameters.

The structural response of confined masonry under lateral loading, at the initial stage, is almost elastic and largely controlled by the characteristic of the masonry wall. The structure behaves as a monolithic element due to the bond strength developed along the structural interfaces. It may be approximately considered that the system is similar to a cantilever wall, as clearly indicates the distribution of principal stresses obtained from a finite element model of a confined masonry wall, see Fig. 1 (a). This conclusion was experimentally verified by Crisafulli (1997), by comparing the measured stiffness of a single confined masonry wall to that obtained from structural analysis considering a monolithic wall.

As the lateral force increases, the masonry or the panel-frame interfaces are not able to resist the tensile stresses and, consequently, cracking occurs in those parts of the wall. Thus, the masonry panel partially "separates" from the surrounding frame, except at the diagonally opposite compression corners, as indicated in Fig. 1 (b). The idea of separation is used in a general sense, including two cases. In some structures the separation physically occurs by cracking and opening of the panel-frame interfaces, whereas in other cases the strength of the interfaces is high and cracking occurs in the masonry adjacent to the interfaces. From the structural point of view, the latter case is equivalent to the first one, and, therefore it is proper to say that "structural separation" occurs between the masonry panel and the surrounding frame. At this stage, the stresses at the tensile corners are relieved while those near the compressive corners are significantly increased, and the masonry is mainly subjected to compressive stresses along the loaded diagonal. This change in the structural response does not affect the resistance of the confined wall, but significantly decreases its stiffness.

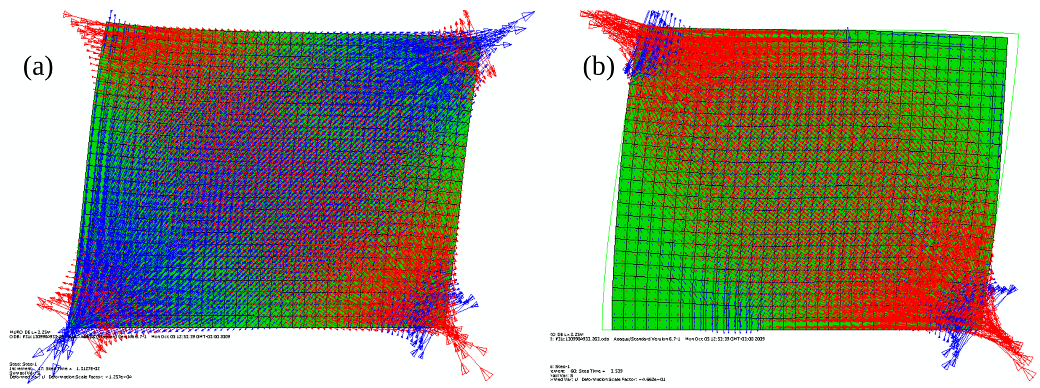


Figure 1. Principal stresses in the masonry wall, (a) before structural separation and (b) after separation (blue arrows indicate maximum principal stresses and red arrows minimum principal stresses).

After structural separation occurs the effect of the masonry panels, even if they are cracked, can be approximately represented by an equivalent diagonal struts, as clearly shown in Fig. 1 (b). It is important to mention that, at this stage, the RC members are subjected not only to axial forces, but also to bending moments and shear forces, particularly at the end of the columns. Fig. 2 shows the moment and forces diagrams obtained from a finite element model of a confined masonry wall. Experimental results (Crisafulli, 1997, Crisafulli et al., 2005) confirmed that the structural response can be represented by the equivalent strut model, both in terms of stiffness and strength. Furthermore, measurements from strain gauges located at the reinforcing bars validated the development of bending moments similar to those shown in Fig. 2 (a).

A further increase of the lateral force results in a large structural separation of the masonry panel and frame, with contact finally restricted to regions adjacent to loaded corners. Cracking of the masonry panel is observed following different patterns, which causes a significant decrease in the stiffness until the maximum lateral resistance is attained. The masonry may exhibit severe damage, and plastic hinges or axial yielding usually develop in the frame due to the increase of the member bending moments. The final behaviour is mainly controlled by the frame, which restrains or confined the cracked masonry wall.

In conclusion, two different stages can be distinguished. During the initial stage the structure behaves as a monolithic cantilever wall until separation occurs. Then, the response is characterized by the composite interaction between the panel and the frame. The induced state of stress into the masonry produces different cracking patterns, with significant damage until the maximum lateral resistance is achieved. Then, the lateral strength decreases and the response is mainly controlled by the frame. Fig.

3 (a) shows the typical force-displacement relationship for confined masonry wall, whereas Fig. 3 (b) presents experimental results under cyclic loading.

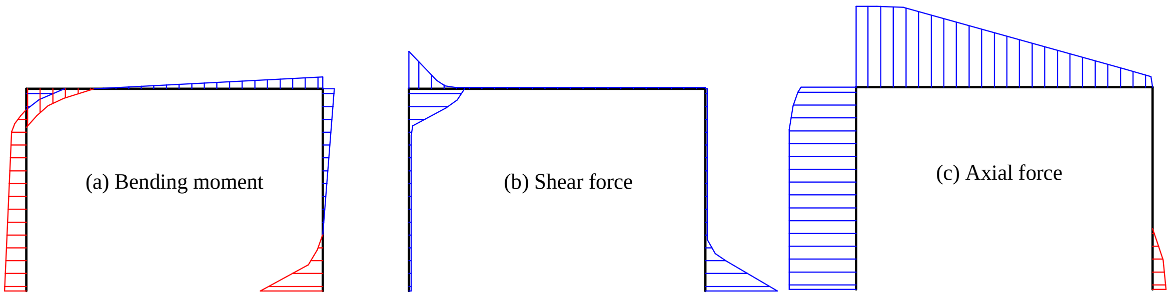


Figure 2. Principal stresses in the masonry wall, (a) before structural separation and (b)after separation.

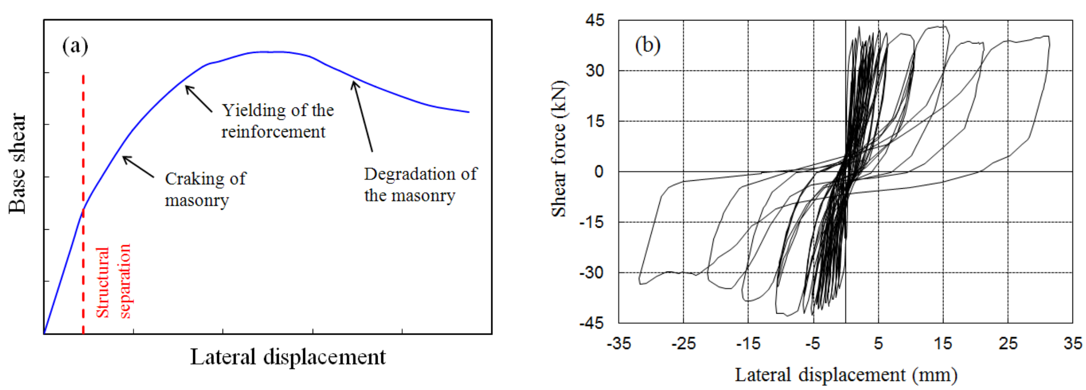


Figure 3. Structural response of a confined masonry wall under in-plane lateral loading, (a) typical monotonic response and (b) experimental response (Crisafulli et al. 2005).

The structural response of infilled frames exhibits similarities, in general sense, to that described for confined masonry. Therefore, the shape of the response curve illustrated in Fig. 3 is also valid for infilled frames. The main differences occur at the initial stage due to the inadequate conditions of the masonry-frame interfaces. As a result, the stiffness of the structure is smaller (compared to a similar confined masonry wall) and the degradation of the masonry wall due to cracking occurs faster. In some cases, depending on the conditions of the interface, the separation can start at very low lateral loads. The final stage of the structural response of infilled frame may also presents some differences compared to that of confined masonry, with a faster degradation of the masonry due to the lack of confinement of the frame. In Section 4, experimental results of an infilled RC frame tested by Meharabi et al. (1996) are presented. It can be observed in Fig. 10 (a) that the structural behaviour is similar to that of confined masonry. Other experimental results available in the literature validate this conclusion (Crisafulli, 1997).

The dynamic behaviour of infilled frames under seismic loading can be dangerous. The initial slackness of the infill panel combined with out-of-plane horizontal accelerations may produce the complete failure of the structure, because the masonry panels are shifted out and fall. This type of failure has been observed in structures subjected to earthquakes or in laboratory tests conducted in shaking tables (Liauw and Kwan, 1992; Lanese et al., 2009).

2.2. Influence of the conditions of the panel-frame interfaces

A large parametric study is under development to investigate the influence of the different conditions at the panel-frame interface, however, only some preliminary results are presented in this paper. The parametric study is based on numerical simulations using refined finite element models with the computer program ABAQUS (Hibbit et al., 2006). In this type of model is very important to properly represent the interfaces between the masonry panel and the RC frame. The interfaces should be able to consider the friction resistance, the bond that may develop between the two materials and the possibility of separation when the interfaces open.

The results presented here were obtained from a model representing a single confined masonry wall, 2.6 m high and 3.3 m wide, with a thickness of 170 mm, which is represented with four node plane stress elements using a concrete damaged plasticity material. The masonry compressive strength is equal to 1.2 MPa and the tensile strength is equal to 0.15 MPa. The RC frame members have a rectangular cross section, 200 x 250 mm, and they are modelled with beam elements. The interfaces are defined as surfaces between the beam elements and the edges of the masonry panel. The conditions at the panel-frame interface were changed (by modifying the tensile strength of the interface, f'_{ti}) in order to considerer to three different cases, see Table 1. Case 1 represents the case of a well-constructed infilled frame and Case 2 and 3 simulates confined masonry walls with usual and improved interfaces, respectively. The coefficient of friction at the interfaces is equal to 0.6 in all the cases (according to the results reported by King and Pandey, 1978).

Table 1. Description of the cases considered to investigate the influence of the interface conditions.

Case	Bond conditions	Observations
Case 1	No bond, f'_{ti} = 0.0 MPa	This case represents the situation of a well-constructed infilled frame, in which there is no bond at the interface, but the panel is in contact with the frame (no gap).
Case 2	Medium bond f'_{ti} = 0.1 MPa	This case represents a confined masonry wall with usual conditions at the interfaces (medium value of the tensile strength)
Case 3	High bond f'_{ti} = 0.3MPa	This case represent a confined masonry wall with a very high value of the tensile strength of the interface

Fig. 4 shows the calculated response of the three models considered in the preliminary study, where it can be clearly observed that the structural behaviour under in-plane lateral loading, despite the differences at the interface conditions, is similar. At the initial stage (range of small displacements), see Fig. 4 (b), the lateral stiffness of each model is significantly different; for example, at a force level of 30 kN the stiffness obtained for Case 3 (high bond) is 2.45 times greater than that for Case 1 (no bond). Structural separation (as described in the previous sections) occurs in all the models at different load levels. In Case 1, separation is observed from the very beginning of the analysis, whereas in Case 3, there is no physical separation, but the masonry adjacent to the frame partially damages, the tensile stresses are relieved and the compressive field develops along the diagonal. This behaviour is certainly observed in Fig. 5, where the plot of minimum (compressive) principal stresses is presented for all the cases. The distribution of the compressive stresses indicates that the width of the equivalent strut is larger in Case 3, due to the improved conditions at the panel-frame interfaces. Fig. 5 also shows the deformed shapes of the model. It must be noted that in Case 1 the separation of the interfaces is clearly observed (beam elements separate from the panel); on the contrary, the frame remains in contact with the masonry panel in Case 3. In the latter case, the analysis of the plastic strains shows that masonry is damaged in those corners where the stresses were relieved.

3. PROPOSED MACROMODEL FOR CONFINED MASONRY AND INFILLED FRAMES

Confined masonry and masonry infilled RC frames can be analyzed using different modelling techniques, ranging from very simple models (such as the equivalent strut model) to very refined finite element models. A new model is proposed in this paper, which includes two macro-elements, namely, (i) a multi-strut formulation to represent the masonry panel and (ii) a refined column element able to capture the bending moment and shear forces that can develop in the RC frame (see Fig. 2). These macro-elements are described in the next sections.

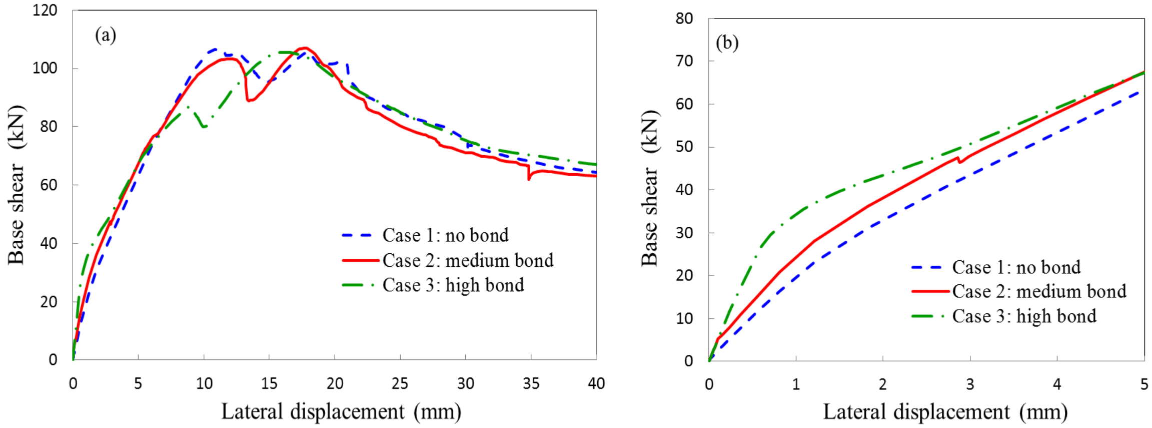
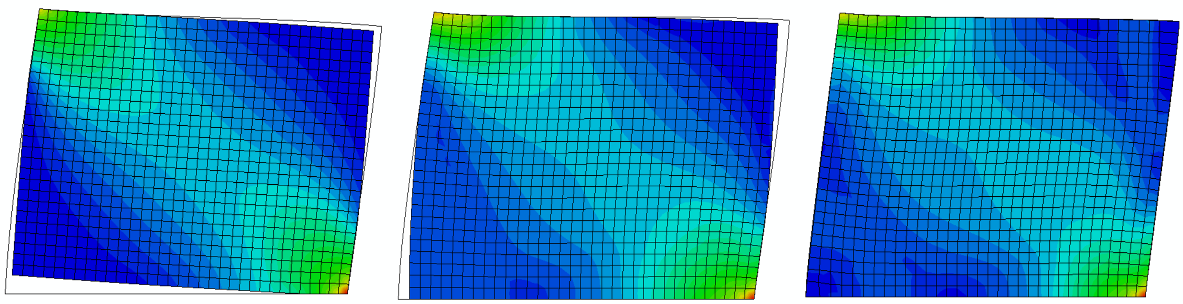


Figure 4. Force-displacement response for the three models, (a) complete response and (b) detail of the response in the range of small displacements.



(a) Case 1 (b) Case 2 (c) Case 3

Figure 5. Compressive principal stresses in the masonry panel at a force level of 90 kN.

3.1. Description of the column macro-element

A column macro-element was developed to take into account the bending and shear effects on reinforced concrete columns. The macro-element, see Fig. 6, has four nodes joining three sub-elements. Each sub-element is an Euler-Bernoulli beam with shear energy, which takes into account the flexural and shear deformations of the element and does not have the problem of Timoshenko beams for large relations of L/d (Torrisi, 2012). The first and third sub-elements have two nonlinear flexural springs at the ends and a nonlinear behaviour in shear, considered by degrading the stiffness of all the sub-elements and represented by a shear spring at the centre of the sub-elements. The central part has an axial nonlinear behaviour, represented by an axial spring. The column element if formulated in term of flexibilities (Filippou and Issa, 1998; Torrisi, 2012) and the complete formulation is described in Torrisi (2012). The flexural springs follow a tri-lineal Takeda hysteretic rule, while the shear springs follow a tetra-lineal model with pinching, as shown in Fig. 7. Finally, the axial spring have a behaviour following a bi-lineal model. The envelope for flexure, (moment vs. curvatures diagram) can be obtained by the classical theory of reinforced concrete and the envelope for shear, (shear force vs. shear strain), can be calculated by the theory of the UCSD or the Xinrong Li model (Torrisi, 2012).

The formulation of this macro-element considers the interaction between the bending moments and axial forces and between shear forces and axial forces (the interaction between bending moments and shear forces is not considered, although they are coupled by equilibrium in the stiffness matrix)

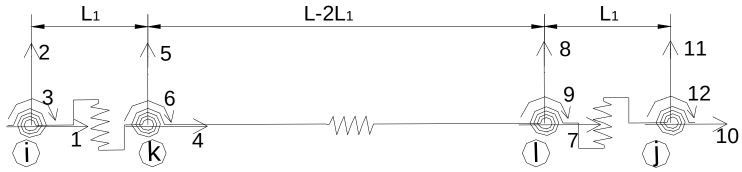


Figure 6. Column macro-element with nonlinear springs and degrees of freedom.

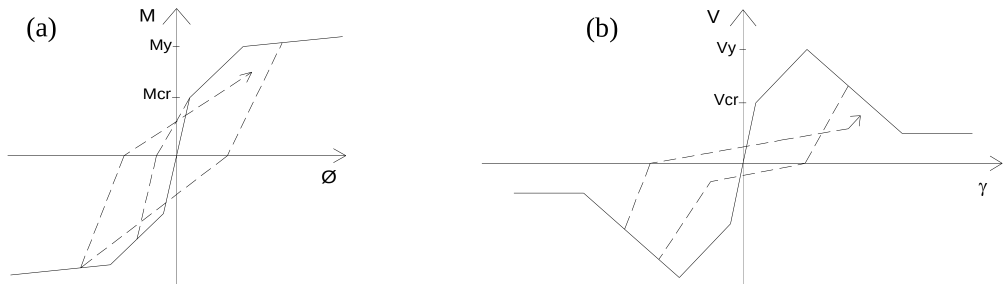


Figure 7. Hysteretic rules for (a) flexure and (b) shear.

3.2. Description of the macro-element panel

The masonry panel is represented with a twelve-node element (with four nodes in each edge in order to allow the proper connection to the column macro-element) and its formulation considers six masonry struts (three in each direction), as shown in Fig. 8 (a). It has been shown that the multi-strut formulation is able not only to estimate the lateral stiffness and the strength of the structure but also to represent the bending moment, shear and axial forces induced in the RC frame as result of the interaction with the masonry panel (Crisafulli, 1997: Torrisi, 2012).

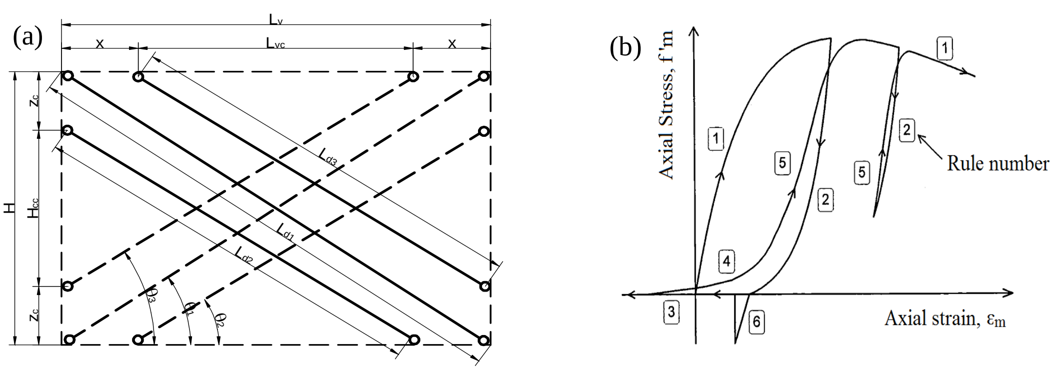


Figure 8. Proposed panel macro-element, (a) geometric and mechanical configuration and (b) hysteretic rule for struts the masonry.

The total area of struts, A_{tot}, in each direction is specified as percent of the diagonal length of the panel (usually $0.15d_m$ to $0.25d_m$, where d_m is the diagonal length), based on the many recommendations available in the literature or in design codes. Also, the percent of area assigned to the central strut is specified (usually $0.35A_{tot}$ to $0.70A_{tot}$). The area of the struts may be degraded to consider the progressive damage in the masonry (in this case is necessary to introduce a residual area for the struts, Crisafulli, 1997).

The behavior of the masonry struts is defined by the hysteretic rule proposed by Crisafulli (1997) and modified by Torrisi (2012), see Fig. 8 (b). The masonry envelope, showed fig. 8(b), requires the definition of the maximum compressive strength for the masonry panel. Different theories have been developed through the years (Mann and Muller, 1982; Dialer, 1991; Crisafulli, 1997), although a new theory has been proposed (Torrisi, 2012) which takes into account the partial or total collaboration of the mortar joints in the strength of the masonry, depending on the workmanship, quality of the materials, condition of the wall, etc. This theory is able to consider different types of failure, such as sliding shear, diagonal tension, and compression failure.

3.3. Implementation and verification of the proposed model

The proposed model, which incorporates the column and panel macro-elements previously described, has been implemented in an in-house program for the analysis of confined masonry and infilled frames. As an example, Fig. 9 presents computed results which describe the nonlinear response of a confined masonry wall and some of its members. It is worth noting in Fig. 9, that the model can represent the nonlinear behaviour due to shear and flexure in the members of the surrounding frame. The model is able to capture the bending moments and shear forces in the frame due to effect induce for the additional strut located off the diagonal, see Fig. 8 (a).

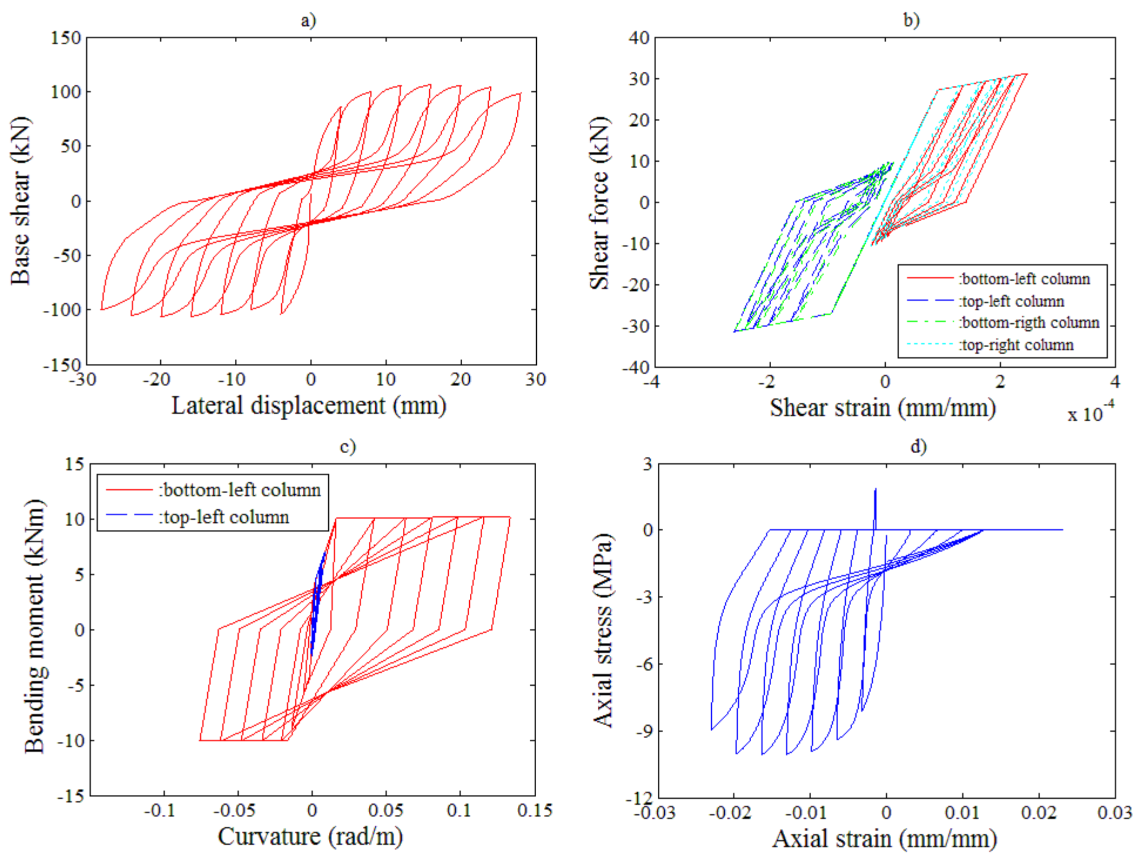


Figure 9. Computed response of a confined masonry wall, (a) global response, (b) shear response at the end of the columns, (c) flexural response of the left column and (d) response of the principal strut.

The proposed model has been tested against experimental results. Fig. 10 (a) shows the experimental response of a specimen tested by Meharbi et. al. (1996), which is compared with the results calculated with the proposed model and also with other programs, such as Ruaumoko (Carr, 2007), FEAP (Taylor, 2005) and ABAQUS (Hibbit et. al., 2006). In figure 10(b), the cyclic response obtained from experimental tests conducted by Crisafulli (1997) is also compared with the proposed model. The comparison between experimental data and numerical simulations indicates, in general, a good agreement. Particularly, in the first comparison, Fig. 10 (a) the initial stiffness and the peak force are well predicted, but for displacements larger than 20mm the strength has a significant difference with experimental results. This is because the analysis was conducted under monotonic loading (according to the experimental data available), even though the experimental curve was obtained as the envelope of a cyclic test. In the second case, Fig. 5(b) the initial stiffness and peak strength are well predicted and also the unloading stiffness and strength degradation is properly captured with the proposed model. These results clearly indicate that the proposed model can reproduce the behaviour of confined masonry and infilled RC frames, even though the parameters of the model need to be adjusted for each particular case.

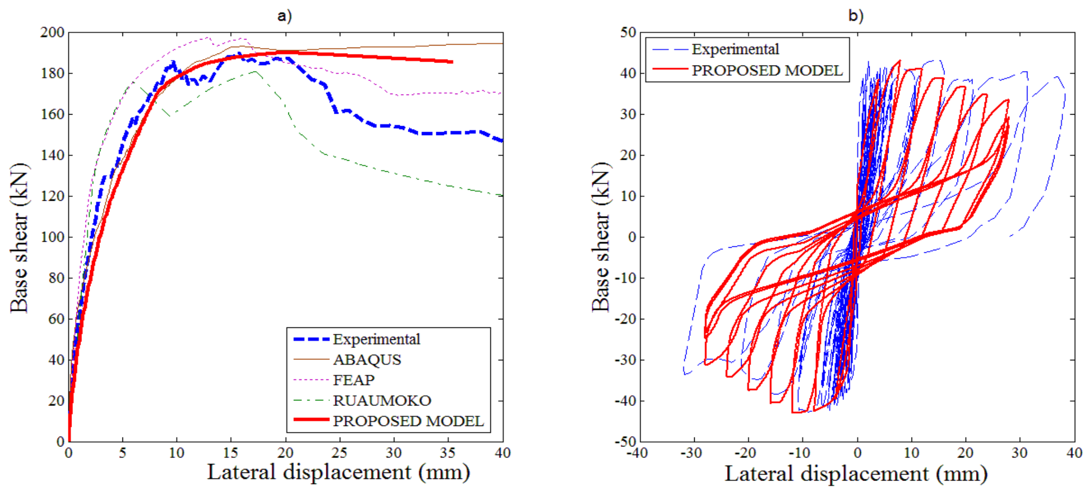


Figure 10. Comparison between experimental data and computed results: (a) infilled frame tested by Mehrabi et.al., 1996, and (b) confined masonry wall tested by Crisafulli, 1997.

4. CONCLUSIONS

The structural response of confined masonry and infilled frames under in-plan lateral loading is similar, despite the different construction techniques. Experimental data and numerical simulations clearly indicate that in both cases structural separation occurs at the initial stage. After this separation, the structural behaviour is characterized by the formation of a diagonal compressive stress field in the masonry and the development of bending moments and shear and axial forces in the RC frame.

A complete parametric study is under development in order to investigate the influence of the panel-frame interfaces, based on the analysis of refined finite element models. Preliminary results, which are presented in this paper, confirm that the computed response of confined masonry and infilled frames is similar.

An innovative model is proposed to analyze confined masonry walls as well as infilled frames, which includes column macro-elements to represent the RC members and panel macro-elements to define the masonry wall. The main advantages of the model are the capacity to predict not only the stiffness and strength of the structure but also to represent the influence of the masonry panel on the surrounding frame. Furthermore, the model can be useful for the analysis of large structures due to its simplicity. The comparison between experimental data and computed results indicates a good agreement and, consequently, validates the use of the propose model for both type of structures.

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