COMPDYN 2011
3rd ECCOMAS Thematic Conference on
Computational Methods in Structural Dynamics and Earthquake Engineering
M. Papadrakakis, M. Fragiadakis, V. Plevris (eds.)
Corfu, Greece, 25–28 May 2011

IN-PLANE BEHAVIOR OF REIFORCED MASONRY WALLS: EXPERIMENTALLY BASED MODELLING

F. Mosele[1], G. Guidi[2], L. Nicolini[2] and F. da Porto[2]

[1] University of Padova, Dpt. Of Structural and Transportation Eng.
via Marzolo 9, 35131, Padova, ITALY
e-mail: mosele@dic.unipd.it

[2] University of Padova, Dpt. Of Structural and Transportation Eng.
via Marzolo 9, 35131, Padova, ITALY
{guidi,nicolini,daporto}@dic.unipd.it

Keywords: Reinforced masonry, Analytical Methods, Numerical Methods, Experimentally based modeling.

Abstract. *In the framework of a recent EU funded research project, innovative construction systems for clay unit reinforced masonry walls were developed. In particular, one system was developed for low-rise residential buildings. An extensive experimental program was mainly aimed to understand the cyclic in-plane behavior under shear and compression loads. The tests results were compared with code proposed formulations for the evaluation of shear strength, in order to check their reliability in predicting the ultimate load capacity of reinforced masonry walls. A new calibrated formulation is proposed. A FE continuum micro-model was calibrated on the experimental results and then used to carry out parametric analyses of the reinforced masonry system, to investigate the influence of the axial load level, the aspect ratio and the reinforcement ratio on the global in-plane behavior of the tested walls. A new analytical hysteretic model was also developed and used to carry out non-linear dynamic analyses of SDOF systems, to evaluate the reduction of the elastic response of reinforced walls, for a range of natural periods that characterize the elastic phase of load bearing masonry buildings.*

1 INTRODUCTION

Reinforced masonry (RM) was developed to exploit the strength potential of masonry and solve its lack of tensile strength [1] while significantly improving resistance, ductility and energy dissipation capacity with respect to unreinforced masonry (URM) [2, 3]. In the last few decades, a large variety of RM techniques have been made available. Many RM systems around the world are based on the use of hollow concrete [4, 5] and clay units [6], which are reinforced with steel bars and grouted with concrete. Other RM systems, traditionally developed in Europe, make use of perforated clay units combined with concentrated vertical reinforcement, [see, for example 7, 8, 9, 10].

Generally, RM systems are designed for low rise residential buildings in seismic areas, which resist horizontal earthquake actions with the walls parallel to the seismic actions, according to the box-type behavior [1]. Therefore, the main aim of any experimental and numerical study is to assess the behavior under in-plane cyclic actions. In the case that seismic design of this type of buildings is based on linear elastic methods of analysis, the evaluation of the strength capacity (ULS) and the numerical values of the seismic behavior factor (q-factor) to reduce the elastic design spectrum, are crucial. The shear strength of RM is generally evaluated as the sum of the contributions of unreinforced masonry and horizontal reinforcement [11, 12, 13, 14], where many issues regarding the evaluation of masonry strength and horizontal reinforcement efficiency are still open [15]. On the other hand, the q-factor has been recognized in the Italian code [12] to be implementing an “overstrength” ratio also in the case of masonry buildings [16, 17], and its values can be higher if capacity design principles are pursued, whereas the European code [18] does not provide these possibilities. Furthermore, more rational design methods, based on non-linear analyses, are being developed [see, for example 19]. Nevertheless, to adopt them, it is necessary to give deformation/drift limits that should be used, suitably revised on the basis of the more recent construction systems and available experimental information [20].

In this context, a RM systems for use in low rise were recently developed [21] and tested [22]. The main aims of the experimental and numerical work were to study the behavior in relation to the above mentioned issues.

2 STUDIED REINFORCED MASONRY SYSTEM

The RM system developed for low rise residential buildings is based on the use of concentrated vertical reinforcement, similar to confined masonry. Special clay units are laid with horizontal holes, with recesses for horizontal reinforcement on the bed faces (Figure 1 left). Vertically perforated units are used for the confining columns. Vertical reinforcement placed in the cavities of the confining columns is composed of steel bars (0.130%÷0.173%); horizontal reinforcement may be made of either steel bars or prefabricated steel trusses (0.045% and 0.040% respectively). The main advantages of the system are related to durability and construction process: placing the horizontal reinforcement inside mortared recesses improves reinforcement durability, makes reinforcement positioning easier and more precise, and allows good bond at the interface unit/ mortar and mortar/reinforcement. In addition, this technique is traditionally adopted in Mediterranean countries to improve thermal insulation. As regards mechanical behavior, this system is conceived to perform as RM, provided that units with horizontal holes are effective in bearing the horizontal loads and transferring them to the confining columns, without showing fragile behavior. More details about this system can be found elsewhere in [22] and [23].

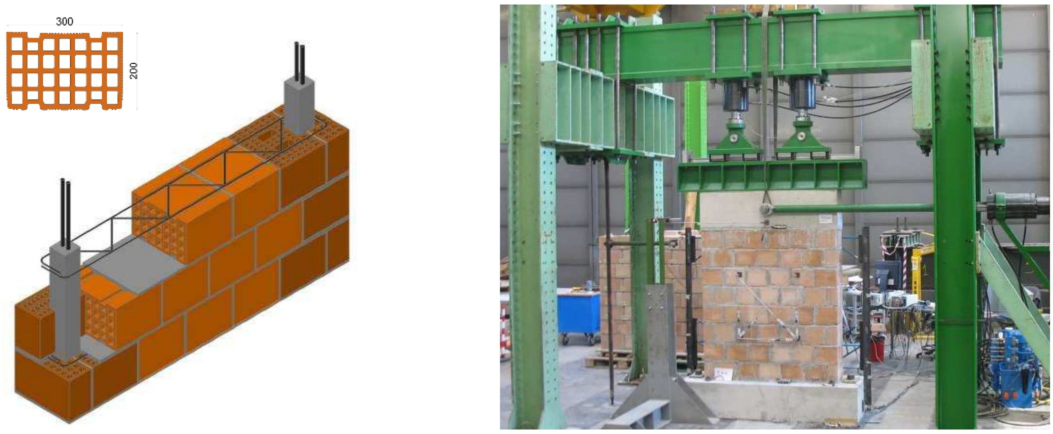

Figure 1: RM system (left) and shear-compression test setup (right).

3 OVERVIEW OF THE MAIN EXPERIMENTAL RESULTS

The seismic performance of the proposed RM system was evaluated by means of in-plane cyclic shear compression tests (Figure 1 right), carried out with cantilever boundary conditions. Fourteen full-scale masonry specimens were tested differentiated by: presence or absence of vertical reinforced confining columns, use of steel bars or prefabricated trusses as horizontal reinforcement, aspect ratio and value of applied axial load, to force both shear and flexural failure modes.

The test results allowed evaluating the influence of the above aspects on the main seismic parameters of RM walls, such as strength and displacement capacity, energy dissipation, viscous damping, stiffness degradation [22, 24, 25].

In general, the failure mechanism strongly influenced all the measured seismic parameters. The tests showed that: the different types of horizontal reinforcement did not cause significant differences in global mechanical behavior, the horizontally perforated units are adequate in bearing the horizontal loads between the confining columns, and the interaction between the inner portion of the wall and the confining columns does not cause premature failure. The ultimate drift θ_u ranged from a minimum value of 0.7% for shear failures to values exceeding 1.7% for flexural failures. These values satisfy the limits associated to ULS for shear (0.6%) and flexural (1.2%) failures of RM walls, adopted by the Italian norms, but the European norms do not provide any drift limit for in-plane response of RM walls. The ductility ratio μ, moves from 2.5 to 4.0 for shear failures and from 3.5 to 6.0 for flexural failures, according to the axial load level. The ratio between dissipated and input energy was around 30%. The values of viscous damping were around 5%, and tended to increase in the post-peak phase.

4 EXPERIMENTALLY BASED MODELING

4.1 Shear strength evaluation

The shear capacity of RM walls is governed by several global and local resisting mechanisms. In general, the combination of vertical and horizontal reinforcement leads to the development of a global mechanism, which lies in between the arch-beam and truss mechanism [1, 26]. While the flexural strength of RM walls is relatively easy to calculate according to theoretical models, the shear strength, due to the complexity of the mechanism, is generally calculated as a sum of contributions, better than on the basis of theoretical models. Four main contributions are usually considered by formulations proposed to predict the nominal shear strength V_R of RM walls: V_m is the shear strength of URM, V_P is the contribution of axial load, V_s is the contribution due to horizontal reinforcement and V_{dw} is the contribution due to dowel-action of vertical reinforcement.

A crucial issue for the shear strength formulation is the efficiency of the horizontal reinforcement, which vary between 30% and 100% according to the various formulations available in literature [see for example 11, 12, 13, 14, 27, 28, 29, 30]. The shear reinforcement effectiveness, evaluated by means of strain gauges, was about 60%, in the present experimental tests, which is consistent with the values provided by codes such as the Italian and American standards [12, 13], and proposed by researchers such as Tomaževic and Anderson-Priestley [1, 28].

As a consequence of an extensive analyses of the shear strength formulations and the comparison with the present experimental data [22, 15], a calibrated formulation for shear strength evaluation has been proposed:

$$V_R = V_m + V_S = \left(\frac{f_t}{b} \cdot \sqrt{\frac{\sigma_0}{f_t} + 1} \right) \cdot td + 0.6 \cdot \frac{dA_{rh} f_y}{s} \tag{1}$$

The equation (1) is based on the Turnšek and Čačovic criterion [31] for evaluating V_m, which implicitly accounts for the contribution of axial load V_P, and which is consistent with the diagonal cracks experimentally observed. The contribution of horizontal reinforcement V_s, is calculated in equation (1) as for stirrups in reinforced concrete members [as in 12, 13], taking into account the number of stirrups, each of area A_{rh}, across the diagonal crack (with 45° slope, d is the effective length of the wall section and s the spacing of the stirrups). The 0.6 reduction coefficient corresponds to the shear reinforcement effectiveness experimentally evaluated.

The comparison between equation (1) and the experimental data is given in Figure 2.

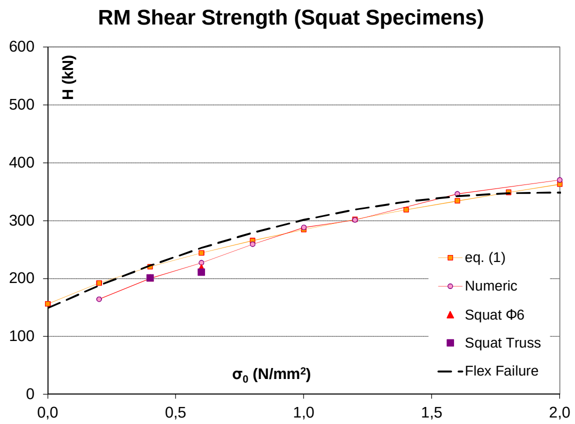

Figure 2: Shear equation and numerical trend vs experimental data.

4.2 Numerical modeling

A simplified micro-modeling strategy with continuum elements and no unit-mortar interface elements was adopted for modeling the envelope of cyclic behavior of the RM walls under study. The model properties were derived from experimental tests. The Total Strain Rotating Crack isotropic damage model [32] was adopted for mortar and blocks. The steel reinforcement was described by means of elasto-plastic Von Mises yield criterion, and had the shape of a line, full bonded and embedded in all the plane stress elements that define the wall geometry. Considering the type of model used, it was not possible to make a distinction between the truss and the bar reinforcement used. The analyses were carried out using the code DIANA. Eight-node isoparametric plane-stress elements with Gauss integration scheme were used in the models. The Newton-Raphson iteration procedure was used with a displacement control and an energetic convergence criterion. The values of fracture energy of masonry in tension (G_{ft}^{I}) and in compression (G_c) were found by means of extensive literature research, summarized into a database valid for masonry structures [33, 34]. Other parameters that were not directly available from the experimental tests carried out are the tensile strengths of the masonry components. The calibration process of the model was carried out starting from uniaxial compression tests, and aiming to solve some defects of the model such as the full-bonded hypothesis used for embedded reinforcement, which is not realistic.

Figure 3 compares the average of the experimental hysteresis loops envelope (H_{med} curve) obtained by the shear compression tests, and the numerical results (Numeric curve), for the specimens failed with shear and flexural mechanism, under axial load of 0.6 N/mm^2. The models slightly overestimate the initial stiffness and reproduce the maximum horizontal load with an average error of about ±5%. Displacements were generally underestimated, but the values of ultimate displacement (when a sudden drop of strength occurs), are in agreement with the experimental ones (average error ±15%), as can be seen in Figure 3.

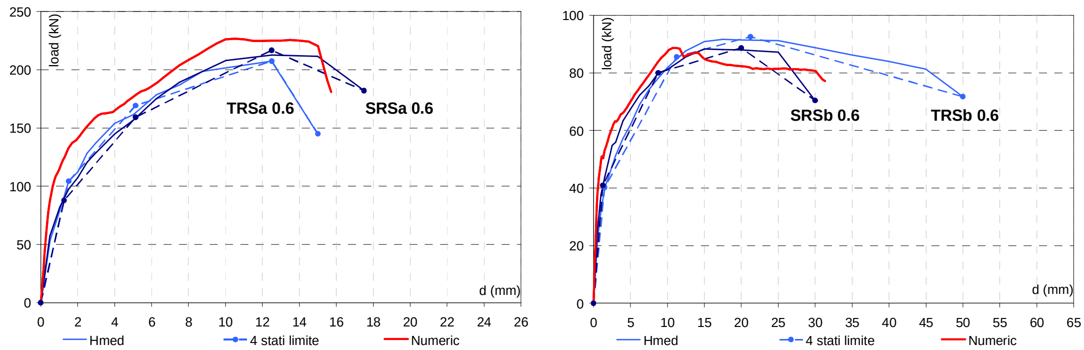

Figure 3: Experimental hysteretic envelopes and numerical pushover analysis: shear failure (left) flexural failure (right).

On the basis of calibrated models, we carried out an extensive parametric study [35, 22] to evaluate the influence of different parameters, such as axial load level, aspect ratio, and amount of vertical reinforcement, on the in-plane behavior of RM walls. The results gave indication about the reliability of the shear strength formulation proposed by equation (1), compared to other formulations available in codes and in the literature. It was possible to confirm the proposed relation between horizontal load and applied axial load, as reported in Figure 2.

In addition, it was found that the contribution of vertical reinforcement is essential for RM walls, since it changes the behavior from rocking mechanism, typical of URM wall, characterized by premature crushing of compressed toe with consequent numerical instability, to a flexural mechanism which leads to higher strength and displacement capacity. When the vertical reinforcement ratio was higher than 0.2%, the walls failed in shear with a limited ductility. This worsening of the wall behavior is more marked for slender rather than for squat specimens.

The parametric analysis on the aspect ratio allowed observing that the maximum shear stress presented a non-linear decrease with increase of H/L ratio.

4.3 Analytical modeling and dynamic analyses

To reproduce the experimental cyclic behaviour, a new hysteretic model was developed. The model was based on the quadri-linear envelope curves defined by the four limit states, and given in [36], and on energy considerations and stiffness degradation rules. Starting from some observation about the shape of the experimental hysteretic cycles [37], the cycles were modelled on the basis of four main points (A; B; C; D) and their symmetrical. These points were found on the basis of the parameters C_1 and C_2, which depend on the amount of the absorbed and dissipated energy during the cycle, and Z, which is a ductility parameter. Figure 4 (left) shows the geometrical scheme for the loops' construction. The slopes of the various loading and unloading phases are given by stiffness parameters, as in [38]. Other two parameters, R1 and R2, are used to model the repeated cycles on the basis of the ratio between input and dissipated energy in the first and, respectively, the second and the third cycle. Overall, the model uses four independent parameters, and the others are all based on those. A more detailed description of the model features is given in [39]. Figure 4 (right) shows the good agreement between the experimental hysteresis loops and those generated by the model.

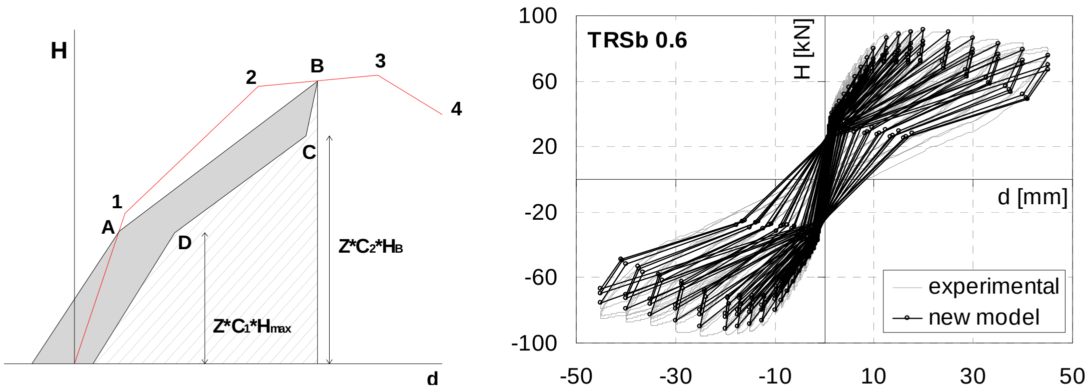

Figure 4: Scheme for the loops' construction (left) and experimental vs numerical hysteresis loops (right).

The developed analytical model was used to carry out non-linear dynamic analyses in order to evaluate the reduction of elastic response of RM walls due to their hysteretic behaviour. They were executed on a SDOF system, using a group of 10 synthetic time-histories, compatible with the spectra of national code. The analyses were carried out for a range of natural periods between 0.10 and 0.5 s, and they were repeated for each soil group classified by the Italian code [12]. Analyses were carried out on the basis of the given value of ultimate ductility factor μ, obtained during tests. The aim was to estimate load reduction factor R_μ due to energy dissipation and non-linear behaviour of the RM system, taking into account shear and flexural failure modes. 2160 analyses were carried out and the obtained values for R_μ were variable with the soil type, axial load level and failure mode. The study of the results, obtained from the dynamic analyses [37], allowed to observe that for natural period of 0.15÷0.20 s, characteristic of masonry buildings, the load reduction factors value is confirmed to be of 2.5 and 3.0 that the Italian norm suggests, respectively for RM failing in shear and in flexure, the latter being associated to the application of capacity design principles. It should be pointed out that the same range of values, regardless of the failure mode, is also given by [18], but as final values of q-factors to be adopted (i.e. neglecting overstrength).

5 CONCLUSIONS

- Extensive experimental and numerical investigations were carried out to improve the knowledge of seismic behavior of RM walls and the available design procedures
- New inputs were provided by the tests about the deformation capacity, to be adopted and implemented in non-linear analyses.
- A shear strength formulation derived from the analysis of the available formulations was in good agreement with the experimental results and consistent with the parametric analyses carried out with the developed numerical model
- The role of vertical reinforcement for RM walls was highlighted and a limitation for vertical reinforcement ratio was identified by means of the parametric analyses carried out with the calibrated numerical model.
- An analytical model derived from experimental results was able to account for the cyclic behavior of RM walls. Adopting this model into dynamic analyses, the capability of RM walls of reducing the dynamic response induced by earthquake, was quantified by the load reduction factor. The results confirmed the values reported in Italian code.
- However, taking into account the intrinsic limitation of the present design procedures [40], further analyses are in progress at University of Padova; mainly an analytical fibre model accounting for the shear/flexural interaction has been developed, to be used in direct displacement based design procedures for RM and URM masonry structures.

REFERENCES

[1] M. Tomaževič, *Earthquake-resistant design of masonry buildings*, Series on Innovation in Structures and Construction, Vol. 1, Imperial College Press, London, 1999.

[2] F. da Porto, M. Grendene, C. Modena, Estimation of load reduction factors for clay masonry walls. *Earthquake Engineering and Structural Dynamics*, **38**(10), 1155-1174, 2009.

[3] F. da Porto, G. Guidi, E. Garbin, C. Modena, In-plane behavior of clay masonry walls: experimental testing and finite element modeling. *Journal of Structural Engineering, ASCE*, **136**(11), 1-14, 2010.

[4] E. Minaie, M. Mota, F.L. Moon, A.A. Hamid, In-plane behavior of partially grouted reinforced concrete masonry shear walls. *Journal of Structural Engineering, ASCE*, 2010.

[5] K.C. Voon, J.M. Ingham, Experimental in-plane shear strength investigation of reinforced concrete masonry walls. *Journal of Structural Engineering, ASCE*, **132**(2), 400-408, 2006.

[6] K. Zilch, D. Schermer, W. Scheufler, Behaviour of reinforced masonry walls made of hollow clay units with concrete infill under combined loading. *14th Int. Brick and Block Masonry Conference*, Sydney, Australia, February 17-20, 2008. (CD-ROM)

[7] A. Bernardini, C. Modena, G. Lazzaro, M.R. Valluzzi, Cyclic behaviour and modelling of reinforced masonry panels. *11th Int. Brick and Block Masonry Conference*, Tongji University, Shangai, October 14-16, 1997.

[8] G. Magenes, In-plane cyclic testing of reinforced masonry shear walls. *11th European Conference on Earthquake Engineering*, Paris, France, 6-11 September 1998

[9] T.P. Tassios, N. Psilla, Reinforced masonry walls under seismic actions. *In: Implications of recent earthquakes on seismic risk. Series on 'Innovation in Structures and Construction', vol. 2*. Elnashai and Antoniou, eds. Imperial College Press, London. 215–228, 2000.

[10] M. Tomaževič, M. Lutman, V. Bosiljkov, Robustness of hollow clay masonry units and seismic behaviour of masonry walls. *Construction and Building Materials*, **20**(10), 1028–1039, 2006.

[11] European Committee for Standardization, *Eurocode 6 - Design of masonry structures. Part 1-1: General rules for reinforced and unreinforced masonry structures. EN1996-1-1*, Brussels, Belgium, 2005.

[12] Ministry of Infrastructures, *DM 14/01/08. Technical Standards for Constructions*. Rome, Italy, 2008. (In Italian)

[13] American Concrete Institute, *Building Code Requirements and Specification for Masonry Structures and Related Commentaries. ACI 530/530.1-08*, January 1, 2008.

[14] Standard Association of New Zeland – SANZ, *Design of reinforced concrete masonry structures. NZS 4230:2004*, January 29, 2004. (and following amendments).

[15] F. Mosele, F. da Porto, C. Modena, Reinforced clay masonry walls: effectiveness of reinforcement and shear equations. *11th Canadian Masonry Symposium*, Toronto, Canada, 31 May-3 June, 2009. (CD-ROM)

[16] G. Magenes, Masonry building design in seismic areas: recent experiences and prospects from a European standpoint. Keynote 9, *1st European Conference on Earthquake Engineering and Engineering Seismology*, Geneva, Switzerland, 3-8 September, 2006. (CD-ROM)

[17] G. Magenes, P. Morandi, Some issues on seismic design and assessment of masonry buildings based on linear elastic analysis. *Michael John Nigel Priestley Symposium, IUSS Press*, 83-94, Pavia, Italy, July, 2008.

[18] European Committee for Standardization, *Eurocode 8—Design of structures for earthquake resistance. Part 1: General rules, seismic actions and rules for buildings. EN1998-1*, Brussels, Belgium, 2004.

[19] G. Magenes, M. Remino, C. Manzini, P. Morandi, D. Bolognini, *SAM II, Software for the Simplified Seismic Analysis of Masonry buildings*. University of Pavia and EUCENTRE, 2006.

[20] G. Magenes, C. Modena, F. da Porto, P. Morandi, Seismic Behaviour and Design of New Masonry Buildings: Recent Developments and Consequent Effects on Design Codes, *Workshop: "Quali prospettive per l'Eurocodice 8 alla luce delle esperienze italiane"*, Università degli Studi di Napoli, Napoli, 3 Aprile, 2009.

[21] *DISWall, Developing innovative systems for Reinforced Masonry Walls*, COOP-CT-2005-018120, 2006-2008. (http://diswall.dic.unipd)

[22] F. Mosele, *In-plane and out-of-plane cyclic behaviour of reinforced masonry walls*. PhD Thesis, University of Trento, Trento, Italy, 2009.

[23] F. da Porto, F. Mosele, C. Modena, Compressive behaviour of a new reinforced masonry system, *RILEM Materials and Structures*, doi: 10.1617/s11527-010-9649-x, 2010.

[24] F. da Porto, F. Mosele, C. Modena, Reinforced clay masonry walls under shear-compression loads: experimental behavior. *11th Canadian Masonry Symposium*, Toronto, Canada, 31 May-3 June, 2009. (CD-ROM)

[25] F. da Porto, F. Mosele, C. Modena, In-plane cyclic behaviour of a new reinforced masonry system: Experimental results. *Engineering Structures, Elsevier*, accepted for publication, 2011.

[26] T.P. Tassios, *Meccanica delle murature*, Liguori Editore, Napoli, Italy, 1988. (in Italian).

[27] Standards Australia, *Masonry structures. AS 3700-2001*, January 1, 2001. (and following amendments)

[28] D.L. Anderson, M.J.N. Priestley, In plane shear strength of masonry walls, *6th Canadian Masonry Symposium*, Saskatoon, Canada, June 15-17, 1992.

[29] P.B. Shing, M. Schuller, V.S. Hoskere, In plane resistance of reinforced masonry shear walls, *Journal of Structural Engineering, ASCE*, **116**(3), 619-640, March 1990.

[30] M. Tomaževič, M. Lutman, Seismic resistance of reinforced masonry walls, *9th World Conference on Earthquake Engineering*, Tokyo-Kyoto, Japan, (Vol. VI), 97-102, August 2-9, 1988,.

[31] V. Turnšek, F. Čačovič, Some experimental results on the strength of brick masonry walls, *2nd Int. Brick Masonry Conference*, Stoke on Trent, British Ceramic Research Association, 149-156, London, UK, 1971.

[32] J.G. Rots, *Structural masonry an experimental/numerical basis for practical design rules*. Balkema: Rotterdam, The Netherlands, 1997.

[33] G. Guidi, *Sistemi di muratura portante in laterizio: calibrazione di modelli numerici sulla base di risultati sperimentali*. Graduation Thesis, Dept. of Structural and Transportation Engineering, University of Padova, Padova, Italy, 2006. (in Italian)

[34] F. da Porto, G. Guidi, E. Garbin, C. Modena, Modelling of in-plane loaded clay unit masonry walls. *14th International Brick and Block Masonry Conference*, Organized by: The University of Newcastle, Sydney, Australia, February 17-20, 2008. (CD-ROM)

[35] F. da Porto, G. Guidi, F. Mosele, C. Modena, Reinforced clay masonry walls under shear-compression loads: parametric analyses. *8th International Masonry Conference*, Dresden, Germany, 04-07 July, 2010.

[36] F. Mosele, F. da Porto, C. Modena, Experimental behaviour of newly developed systems for load bearing reinforced masonry walls. *14th International Brick and Block Masonry Conference*, Sydney, Australia, February 17-20, 2008. (CD-ROM)

[37] F. da Porto, M. Grendene, F. Mosele, C. Modena, In-plane cyclic testing and dynamic modelling of reinforced masonry walls. *14th World conference on Earthquake Engineering*, Bejing, China, 12-17 October, 2008.

[38] M. Tomaževič, M. Lutman, Seismic behaviour of masonry walls: modelling of hysteretic rules. *Journal of Structural Engineering, ASCE*, **122**(9),1048-1054, 1996.

[39] L. Nicolini, Sperimentazione e modellazione del comportamento ciclico nel piano di muratura armata per la definizione di fattori di riduzione delle forze. Graduation Thesis, Dept. of Structural and Transportation Engineering, University of Padova, Padova, Italy, 2008. (in Italian)

[40] M.J.N. Priestley, D.N. Grant, C.A. Blandon, Direct displacement-based seismic design, *New Zealand Society for Earthquake Engineering Conference*, 2005.