

ДГКМ
ДРУШТВО НА
ГРАДЕЖНИТЕ
КОНСТРУКТОРИ НА
МАКЕДОНИЈА

Партизански одреди 24,
П.Фах 560, 1001 Скопје
Македонија

MASE
MACEDONIAN
ASSOCIATION OF
STRUCTURAL
ENGINEERS

Partizanski odredi 24,
P. Box 560, 1001 Skopje
Macedonia

TP-5

mase@gf.ukim.edu.mk
http://www.mase.org.mk

Повикано предавање * Invited Paper

Пауло Б. ЛОРЕНЦО¹, Граца ВАСКОНЦЕЛОС²

**МОДЕРНИТЕ СИДАНИ КОНСТРУКЦИИ И ЗЕМЈОТРЕСИТЕ: АРМИРАНИ
ИЛИ НЕАРМИРАНИ?**

РЕЗИМЕ

Даден е краток преглед на еволуцијата на конструктивната сидарија, од однесувањето на најстари камени градби (линија на притисок) до современо глобално однесување со користење сидови на смолкнување. За модерната конструктивна сидарија, употребата на неармирана, утегната и армирана сидарија е актуелно во сеизмички активни подрачја. Првиот фокус е даден на последните пристапи кон проценка на сигурноста на згради од неармирана сидарија и нивното однесување при тестирање на вибро-платформа. Потоа, презентирани се системи за градење на модерни сидани конструкции неодамна развиени и тестирани на Универзитетот Minho, еден со примена на лесни бетонски блокови и друг со нормални бетонски блокови. Дискутирани се спроведените експериментални и нумерички истражувања и дадени се заклучоци за однесувањето на системите.

Клучни зборови: технологија за градење, конструктивна сидарија, статички тестирања, вибро платформа, нумеричка анализа

Paulo B. LOURENÇO¹, Graça VASCONCELOS²

**MODERN MASONRY STRUCTURES AND EARTHQUAKES:
REINFORCED OR UNREINFORCED?**

SUMMARY

The evolution of structural masonry is briefly reviewed, from old thrust line behavior to modern global behavior using shear walls. For modern structural masonry, the use of unreinforced, confined and reinforced masonry is addressed in seismic areas. A first focus is given to recent approaches towards the safety assessment of unreinforced masonry buildings and their performance in shaking table tests. Subsequently, building systems for modern masonry structures recently developed and tested at University of Minho are presented, one based on lightweight concrete blocks and another based on normal concrete blocks. The experimental and numerical work carried out is discussed and conclusions on the performance of the systems are given.

Keywords: Building technology, Structural masonry, Static testing, Shaking table, Numerical analysis

¹ Prof. PhD, ISISE, Department of Civil Engineering, University of Minho, Portugal, pbl@civil.uminho.pt

² Assist. Prof. PhD, ISISE, Department of Civil Engineering, University of Minho, Portugal, gracia@civil.uminho.pt

1. INTRODUCTION

The lack of masonry codes and norms, on top of technological and architectural motivations, has been an important reason for the loss of market in structural masonry. Currently, norms are widely available and the designers have adequate tools for structural masonry design, even if no consensus exist on how to adequately assess the safety of unreinforced masonry structures subjected to earthquakes. The use of masonry until the 20th century occurred with moderate innovation, based on the principle that it possesses a low tensile strength. The resulting structural form consisted of thick masonry walls with floors made of timber or other. Design based on graphical methods or simple calculations as cantilever walls, without shear walls, lead to increasing thickness from top to bottom. The famous Monadnock building in Chicago is the exponent of this typology with 16 floors and a thickness in the base of 1.82m, see Figure 1.

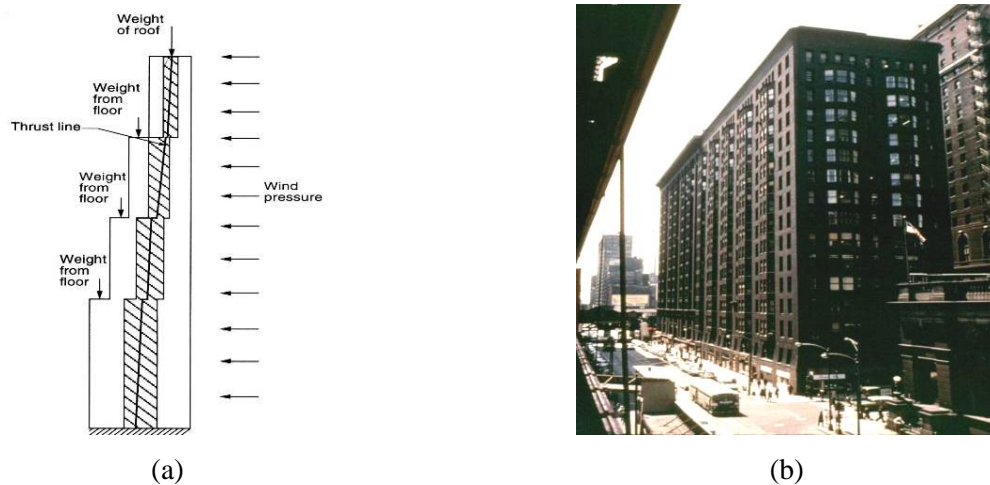


Fig. 1. Traditional (unreinforced) masonry designed for compressive loading: (a) possible actions in an external wall (self-weight, live floor load and wind) and associated thrust line; (b) Monadnock building in Chicago, USA.

Of course, structural masonry design has much evolved and modern design considers the combined behavior of floors and walls. Masonry is still the most used infill material for reinforced concrete frames. Modern engineered masonry became popular as long horizontally reinforced non-load bearing walls in non-residential buildings (Lourenço, 2004). A major challenge that has to be faced by the brick and block producers is the finding of an effective and attractive load bearing masonry system that is able to convince contractors and designers to use it in low and medium-rise buildings, in moderate and high seismicity countries. Besides addressing the safety assessment and performance on unreinforced masonry structures, the paper describes several modern structural systems and details recent research. A first wall system is co-sponsored by the lightweight concrete masonry block industry, where different possibilities of confined masonry walls are envisaged. A second system of masonry walls involves the hollow concrete block masonry industry and deals with the development of innovative systems for reinforced masonry walls. The key aspects under discussion are: (a) the possibility of using unreinforced masonry systems in low and moderate seismicity countries; (b) the adequacy of the safety assessment tools; (c) the possibility of replacing the filling of the vertical joints by unit interlocking and horizontal bed joint reinforcement; (d) the need for filling vertical joints in confined masonry solutions; (e) reinforced masonry systems based on vertical and horizontal truss reinforcement.

2. TYPICAL MODERN STRUCTURAL MASONRY SOLUTIONS

A modern conception of masonry buildings based on shear walls, in which longitudinal walls, transverse walls and slabs resist together against horizontal actions, was introduced in several countries in the years 1950-60. The advantage of this principle is that the walls are used in compression and shear, being possible to make buildings with a high number of floors using unreinforced masonry and walls with moderate thickness. Design was supported by experimental research programs of large dimension and solid structural analysis techniques. The buildings shown in Figure 2 have a height comparable with the Monadnock building but the thickness of the walls varies between 0.15 and 0.30m.



Fig. 2. Modern (unreinforced) masonry high rise buildings, designed for compression / shear.

Due to the significant damage that occurred in large magnitude earthquakes, several “reinforced” masonry solutions were developed through time. In Portugal, the Lisbon earthquake in 1755 promoted a timber-masonry composite system. With time, other solutions were proposed, such as ties and iron cramps or dowels in the masonry units, aiming at increasing the performance of masonry when subjected to a large seismic demand. Still, the devastating effects of large magnitude earthquakes continued to be observed. In the early 20th century, three earthquakes of large magnitude, see Figure 3, contributed to the empirical assumption that masonry structures are unsafe in seismic regions and that the performance of reinforced and steel structures is better. In several countries, e.g. USA, the solution found was to use unreinforced masonry only for low rise buildings and develop reinforced solutions for taller buildings, see Figure 4. Another solution is confined masonry, addressed later in this paper.

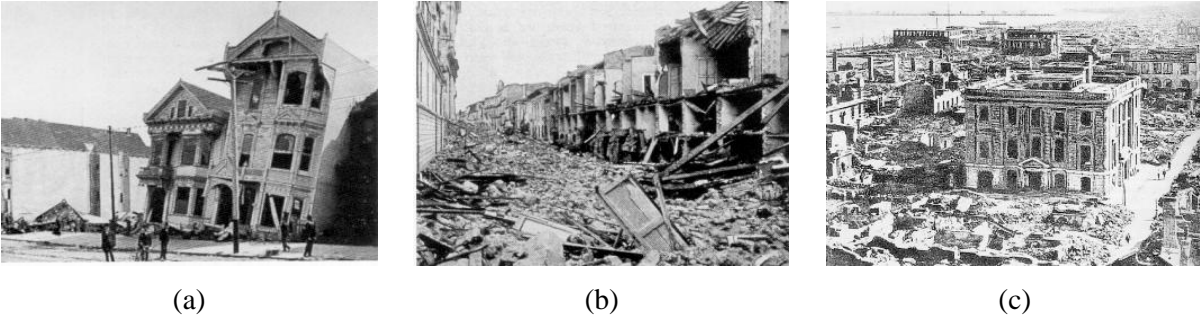


Fig. 3. Images of the devastating effects of earthquakes: (a) San Francisco, USA (1906); (b) Messina, Italy (1908); (c) Tokyo, Japan (1923).

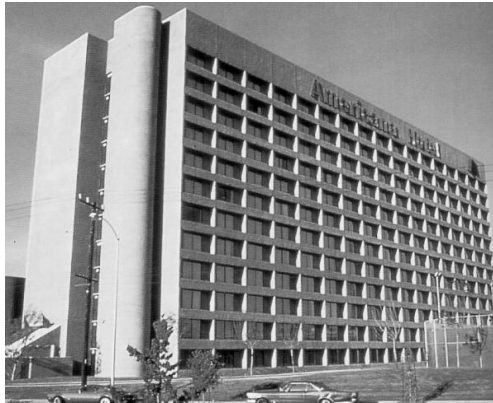


Fig. 4. Modern (reinforced) masonry in high rise buildings up to 30 stories, designed for compression, shear and tension.

2.1 Unreinforced masonry

In Europe, the building solutions using unreinforced structural masonry represent about 15% to more than 50% of the new housing construction, taking as reference countries with low seismicity (e.g. Germany, Netherlands or Norway) but also countries with high seismicity (e.g. Italy). A usual solution is the adoption of masonry units with large thickness in the building envelope to fulfill thermal requirements, see Figure 5. It is stressed that an integrated and complete building technology is needed, including units with different shapes and solutions for floors, see Figure 6.

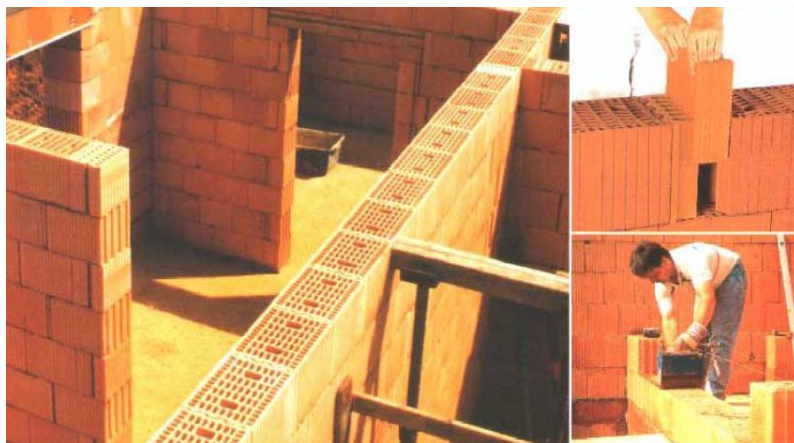


Fig. 5. Details of modern construction in Germany using structural clay masonry.



(a)



(b)



(c)

Fig. 6. Modern unreinforced masonry integrated building systems; (a) calcium-silicate units; (b) clay units; (c) lightweight concrete units.

The solutions shown, typical of countries with low seismicity, are also used in Italy with several additional requirements, namely with respect to robustness of the masonry units (minimum strength and moderate percentage of holes) and the presence of bond beams at floor levels. Figure 7 illustrates examples of Italian structural masonry design, where the combination of load-bearing walls (thicker) with partition walls (thinner) can be observed. It is stressed that the design of unreinforced masonry structures under seismic loading has not yet received general consensus at European level. In particular, the use of elastic design methods and the behavior factors in Eurocode 8 lead, usually, to results different from the ones in the simplified methods and from the results obtained in shaking table tests (Magenes, 2006).

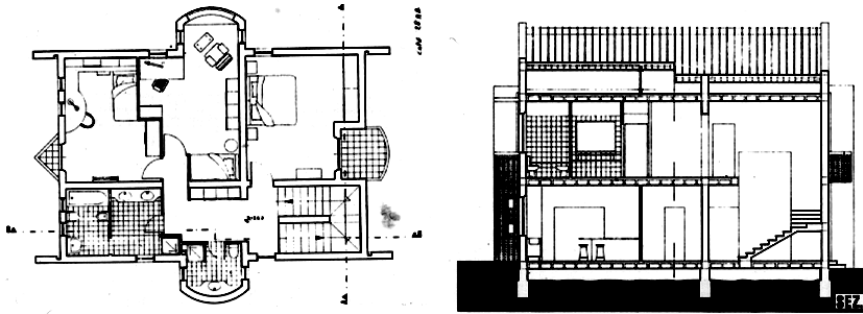


Fig. 7. Unreinforced modern masonry: Design example in Italy.

2.2 Reinforced masonry

Reinforced masonry was developed in different countries as a response to the lower performance of unreinforced masonry buildings under large horizontal loading, but no unified solution was found. Below selected solutions with different levels of success are shown, together with recent innovative solutions. It is common practice to combine prefabricated slabs with load resisting walls, so that formwork, scaffolding and execution times can be significantly reduced. In USA, in the last 30 to 40 years, reinforced masonry became an attractive and efficient solution from a perspective of cost-benefit analysis for buildings in regions of low to high seismicity, including e.g. hotels, residential buildings, office buildings, schools, commercial buildings or warehouses. The standard solution includes reinforced concrete horizontal bond beams, two-cell blocks filled with grout and vertical reinforcement, see Figure 8.

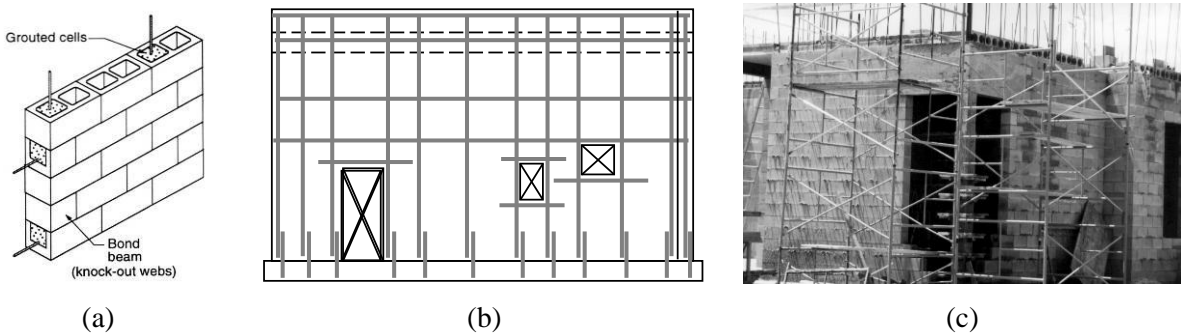


Fig. 8. Modern reinforced masonry (typical American solution). Details: (a) reinforcement and units; (b) reinforcement lay-out; (c) wall execution.

In Italy, in the last 20 to 30 years, a reinforced masonry system was developed incorporating blocks with a large hole for placement of vertical reinforcement and horizontal hoop bars, using the same mortar for the bed joints and for filling the hole, see Figure 9. A prototype building for comparison with a reinforced concrete solution (with masonry infills) was made and several models were tested in a shaking table. The adequacy of the proposed system was demonstrated by tests and prototype, but this reinforced masonry solution had only moderate success in Italy, in comparison to the still used unreinforced masonry solutions for low rise buildings.

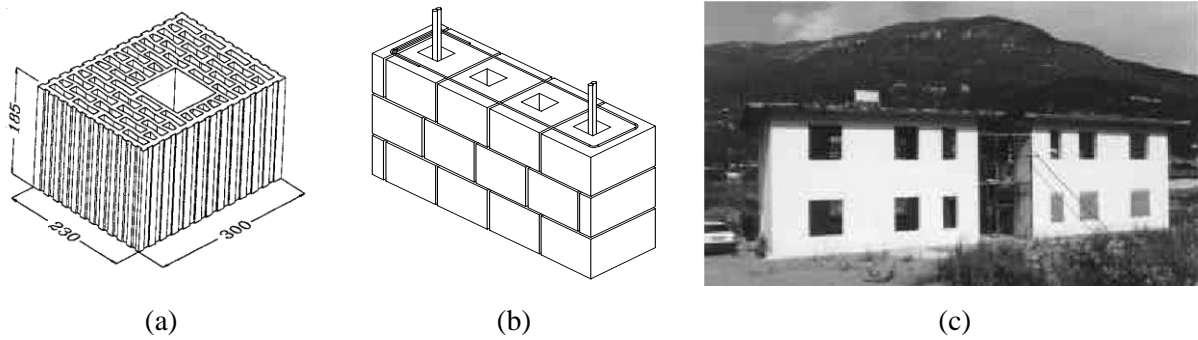


Fig. 9. Modern reinforced masonry (typical Italian solution). Examples of: (a) units; (b) reinforcement lay-out; (c) prototype building (Modena et al., 2004).

In Switzerland, in the last 15 to 20 years, a reinforced masonry system was developed incorporating blocks with two holes of large size, for placement of a complex 3D reinforcement that simultaneously acts as vertical and horizontal reinforcement, see Figure 10a. The same mortar is again used in the bed joints and in filling the holes. The system is used frequently for buildings up to 4 or 5 stories. In Spain, in the last 15 to 20 years, a reinforced masonry system was also developed incorporating truss reinforcement protected against corrosion, horizontally and vertically (Adell, 2000). This system was developed as an alternative to the traditional solution for non-loadbearing walls of large size, see Figure 10b, with horizontal bond beams and vertical elements, made with reinforced concrete. Presently, a similar system is under validation in Portugal, as shown below in the paper. Finally, a recent system is being developed in Germany, consisting of two-cell clay blocks filled with self-compacting concrete, and vertical and horizontal reinforcement, allowing to cast slab and walls simultaneously, see (Mosele et al., 2006) for details.



Fig. 10. Modern reinforced masonry solutions: (a) Switzerland; (b) Spain for infills.

2.3 Confined masonry

Confined masonry is a system in which vertical and horizontal reinforced concrete elements of small section are included in the masonry, see Figure 11. These elements aim at providing an increase of shear and flexural strength, together with a larger energy dissipation capacity and larger ductility with respect to horizontal actions. The system is often used in developing countries, as the changes with respect to unreinforced masonry construction are small. The system received limited attention from the research community.

3. UNREINFORCED MASONRY SAFETY ASSESSMENT

Masonry buildings are often made with unreinforced masonry, constructed without the consideration of earthquake design requirements or reference to any particular design code. The analyses made on

the damages of buildings after several earthquakes through history have revealed the high seismic vulnerability of this type of construction (Bruneau, 1994). It is common that total or partial collapse of unreinforced masonry buildings occurs during an earthquake due to poor quality of materials and construction technology, lack of connection between the intersection walls and between walls and floors and ceilings (Mendes and Lourenço, 2010)..

The low tensile strength, low ductility and low ability to dissipate energy are the main reasons for European codes limiting the use of unreinforced masonry in high seismicity regions, typically a PGA above 0.2g (Magenes, 2006). But more is needed on assessment methods and testing..



Fig. 11. Example of modern confined masonry under construction.

3.1 Structural components models

Modern masonry buildings usually adopt solutions for the slabs that provide considerable in-plane stiffness. This is done by using monolithic systems for the floors, in concrete and steel, and also by establishing an effective connection between slabs and walls. Moreover, many existing buildings originally constructed with timber floors are capable of providing diaphragmatic actions or have been rehabilitated by stiffening the floors and by providing adequate connections. The effect of floor diaphragms combined with the in-plane response of structural walls provides box behavior to the building, which usually leads to good performance of the structure when subjected to earthquakes. The first assessment method for seismic analysis of masonry buildings was developed under this simple hypothesis. This early attempt was then the seed for more sophisticated methods recently developed. Next, a review is made on the development and application of recent analysis methods.

Several methods based on macro-elements have been developed, particularly in Italy. These methods seem the most appropriate for design and assessment of masonry buildings, given their widespread in commercial software, the simplicity of modeling, the straightforward interpretation of results and the accuracy demonstrated in different validations. For a correct simulation of the masonry panels failure mechanism and their behavior different types of macro-elements have been developed, such as the formulations proposed by (Gamberotta and Lagomarsino, 1998) and (Magenes and Della Fontana, 1998) shown in Figure 12, which are incorporated in the 3Muri [www.stadata.com] and ANDILWall/SAM II [www.crsoft.it/andilwall] computer codes, respectively. While the 3Muri formulation is based on the kinematic equilibrium of the macro-elements according to the panel degrees of freedom, the SAM II creates an equivalent frame idealization for a global analysis.

The 3Muri and SAM II computer codes perform the safety verification by a nonlinear static (pushover) analysis (Figure 13) that simulates the evolution of the structural condition during the earthquake, through application of incremental horizontal forces until collapse. The behavior of the structure is represented by the so-called “capacity curve”, which represents the value of the base shear versus the displacement of a control point (usually the mass centroid of the roof slab). Recently, (Marques and Lourenço, 2011) carried out a benchmarking process and different codes were compared. Good agreement of the results was obtained for a pushover analysis on two buildings.

An alternative general purpose software for push-over analysis is SAP2000 computer code [www.csiberkeley.com], based on a frame idealization. In this case, it is necessary to define the possible locations and types of plastic hinges that might develop along each element, to describe

possible failure mechanisms (flexural or shear), as shown in Figure 12c (Pasticier et al., 2008). In addition to 3Muri, SAM II and 3DMacro (Caliò et al., 2012), RAN (Augenti, 2004) is another Italian macro-element method, which can be programmed in a worksheet. RAN allows a global nonlinear analysis of collapse (Figure 12d).

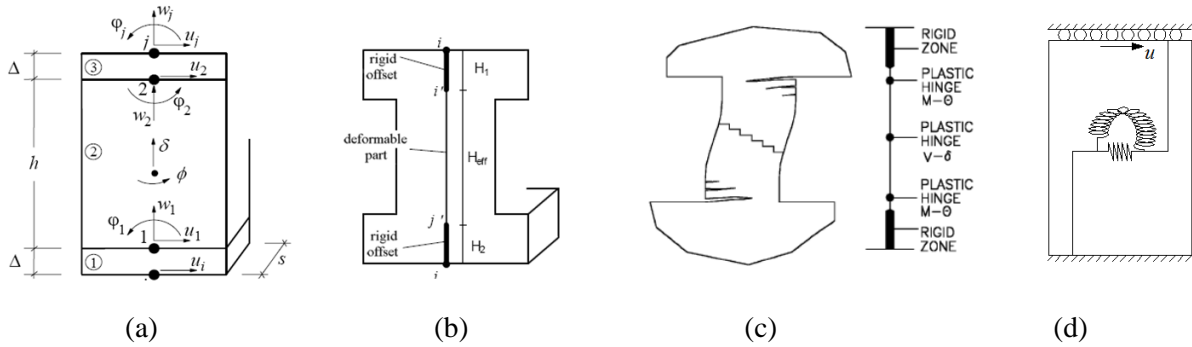


Fig. 12. Macro-element in different methods: (a) 3Muri; (b) SAM II; (c) SAP2000 (d) RAN.

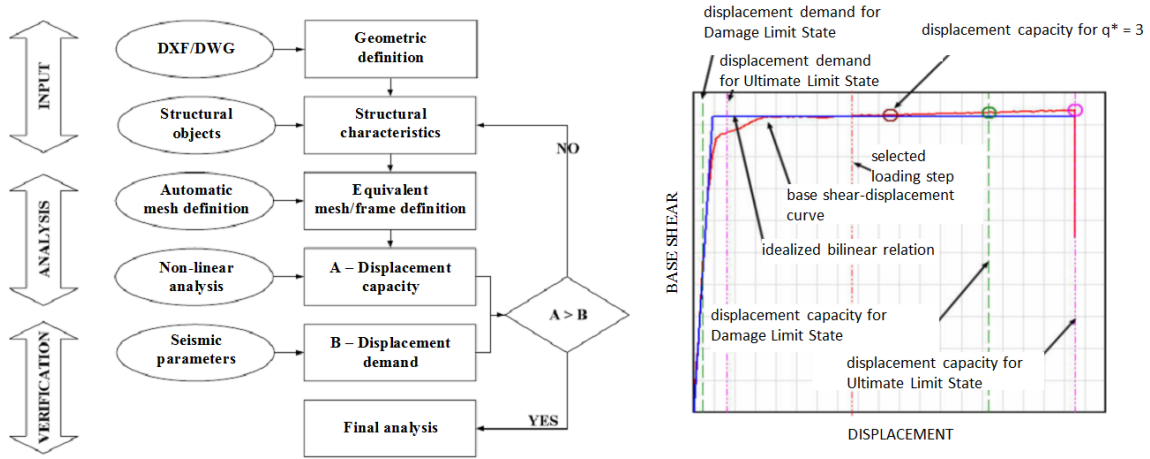


Fig. 13. Seismic verification by nonlinear analysis: flowchart and capacity curve.

3.2 The role of energy dissipation capacity

Despite the recent introduction of methodologies that allow considering the nonlinear reserve capacity of structures in displacements, namely by a pushover analysis, buildings are traditionally designed for earthquakes using force-based approaches and linear elastic analysis. The consequence is that safety assessment of existing structures is often incorrect and behavior factors are required.

The behavior factor q of a given structure is normally defined as the ratio between the F_y strength of an ideal bilinear system equivalent to the true nonlinear, and the maximum elastic base shear $F_{el,max}$. According to (Magenes, 2006), after reaching the strength capacity (shear or flexural) for an element according to a linear elastic analysis, the deformation capacity into the nonlinear regime, even if limited in some cases, is sufficient to allow the system to sustain an increasing seismic load, due to the increase of forces on other structural elements. This force redistribution possibility is already accepted for framed structures in Eurocode 8 (EC8), and for masonry structures in the Italian code OPCM 3431, for which the definition of the behavior factor considers an over-strength ratio (OSR). Thus, the definition of q should be:

$$q = \frac{F_{el,max}}{F_{el}} = \frac{F_{el,max}}{F_y} \cdot \frac{F_y}{F_{el}} = q_0 \cdot \frac{F_y}{F_{el}} = q_0 \cdot OSR \quad (1)$$

where F_{el} represents the base shear at which the first element would reach its strength capacity (shear or flexural) according to a linear elastic analysis.

In the case of EC8, a range of behavior factor values is provided for masonry structures but, in each country, a maximum value of this factor can be defined in its National Annex. EC8 recommended values are the lower limits of the possible ranges. In the case of unreinforced masonry buildings, regular in elevation and with two or more stories, the Italian code allows to adopt an elastic force reduction factor value 2.4 times greater than that allowed by EC8 (Table 1).

Building configuration		EC8	OPCM 3431		
Type of construction	N. of storeys	Behaviour factor q (*)	Basic value q_0	OSR	Behaviour factor q
Unreinforced masonry building; regular in elevation	One	1,5 – 2,5	2,0	1,4	2,80
	Two or more			1,8	3,60
Unreinforced masonry building; non regular in elevation	One	1,5 – 2,0	1,5	1,4	2,10
	Two or more			1,8	2,70
Reinforced masonry building; regular in elevation	One	2,5 – 3,0	2,5	1,3	3,25
	Two or more			1,5	3,75
Reinforced masonry building; non regular in elevation	One	2,0 – 2,4	2,0	1,3	2,60
	Two or more			1,5	3,00

* The upper limit values of q for use in a country may be found in its National Annex. However, the EC8 recommended values are the lower limits of the ranges

Table 1. Behavior factor values for masonry buildings.

In order to obtain additional information on the range of values for the behavior factor distinct researches were performed at University of Pavia (Magenes, 2006) and at ZAG in Slovenia (Tomažević, 2007). The latter tested on shaking table a series of models representing masonry buildings of two different structural configurations, typical for central Europe (a three-story apartment house and a two-story terraced house), and two different types of masonry materials (Figure 14a-b). The former conducted also a numerical analysis using the SAM II method, where nine plan configurations of plain masonry buildings from one to three stories were tested. Then, the OSR factor was computed from the capacity curve obtained for each building. Figure 14c-d reports the histogram of the values of OSR (F_y/F_{el}) that were obtained for the sample of two- and three-story unreinforced masonry buildings.

Based on the shaking table experimental results, particularly on the computed values of ductility and the structural behavior factor q_{μ} calculated on the basis of damage-limitation requirements, and in the OSR values obtained from the numerical analysis, the need to adopt higher values for the behavior factor seems clear, particularly when compared to EC8.

3.3 Case study

To discuss the possibilities of construction with unreinforced masonry in Portugal, the seismic safety of buildings with one up to three stories, based on a pushover analysis carried out in the 3Muri computer code, is considered next. The building configurations studied include a one-story module, and two- and three-story buildings for semi-detached houses. The properties of the materials of the masonry are given in Table 2.

Figure 15 illustrates the ultimate response in terms of deformed configuration and damage of the three buildings, where it can be observed that the collapse mechanisms are essentially induced by flexure, while plastic mechanisms by shear are only found for the three-story building in spandrels adjacent to the first slab. Based on the requirements for earthquake resistance imposed by the Italian code OPCM 3431/2005, and assuming the seismic parameters defined in the Portuguese Annex to EC8, the possibility to construct the studied buildings in Portugal was evaluated using 3Muri, which is given in Figure 16.

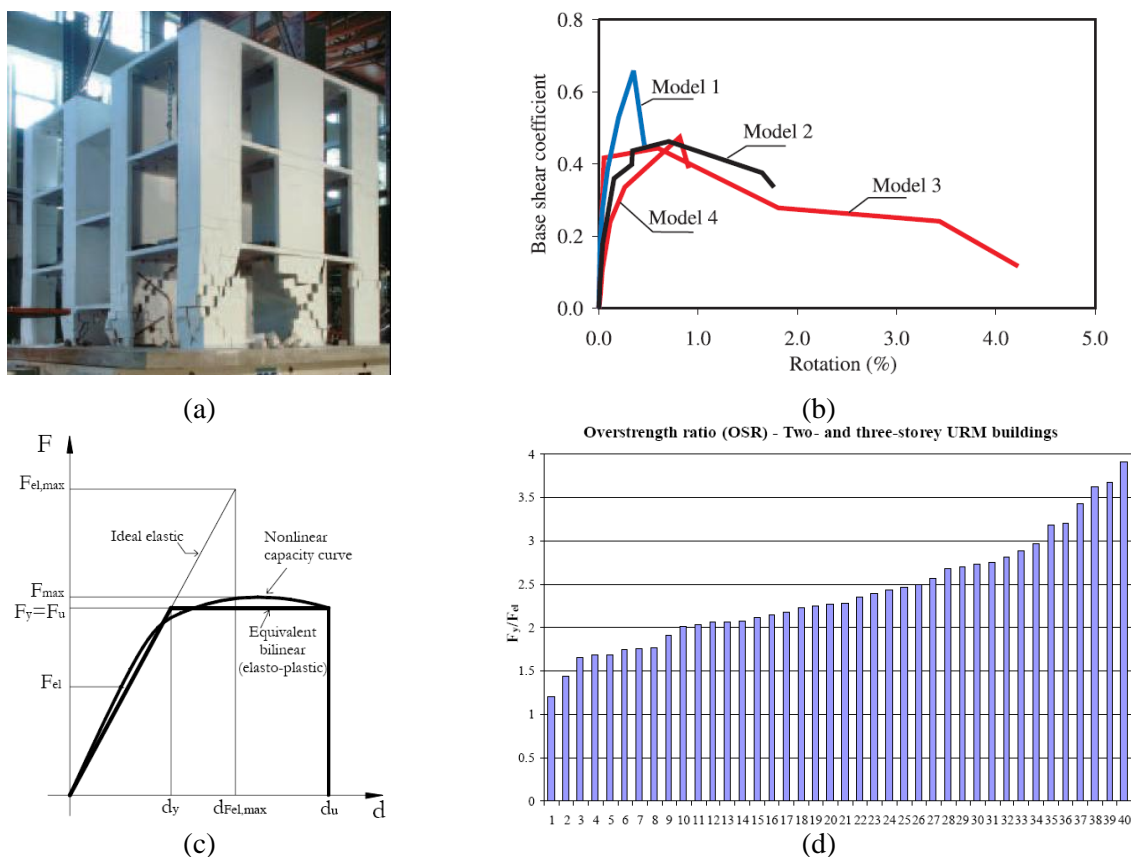


Fig. 14. Behavior factor for unreinforced masonry buildings: (a) shaking table test at ZAG and (b) resistance curves of models of 2- and 3-stories (Tomažević, 2007); (c) reference parameters for q_0 and OSR and (d) calculated OSR values for 2- and 3-stories buildings (Magenes, 2006).

UNITS	Type according to EC6	Clay units of Group 2
		Compressive strength, f_b
MORTAR	Type according to EC6	M10
MASONRY	Specific weight, γ	17.0 kN/m ³
	Compressive characteristic strength, f_k	2.56 MPa
	Pure shear characteristic strength, f_{vk0}	0.15 MPa
	Normal elasticity module, E	2560 MPa
	Tangential elasticity module, G	1024 MPa

Table 2. Building configurations studied.

The proposed buildings can be constructed in most of the country, only with absolute restrictions in Seismic Zones 1.1 in general, and 1.2 for buildings of two- and three-stories. Performing an elastic analysis adopting a behavior factor of 1.5, as recommended by EC8, the safety verification is over conservative, as shown in Figure 17a. A better correspondence between the pushover and linear analysis is achieved by assuming the behavior factor values proposed by OPCM 3431, as shown in Figure 17b. In the case of the regular building configuration adopted, behavior factors of 4.0, 3.0 and 3.5, respectively for the one-to-three story buildings allow a perfect match between the linear and nonlinear analysis.

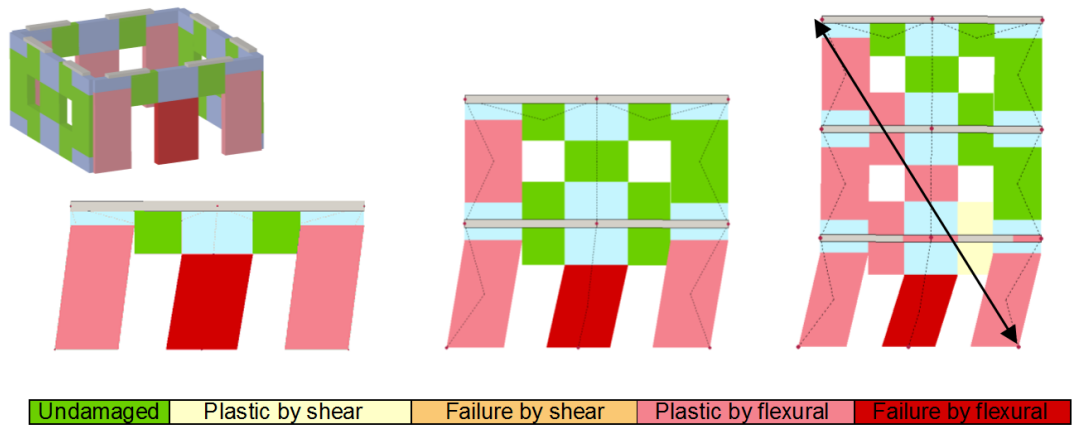
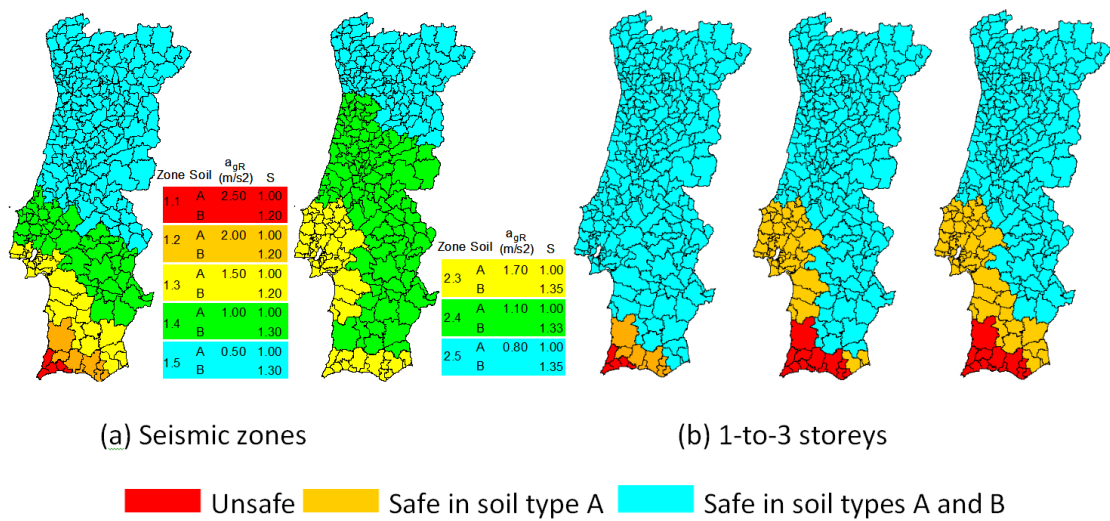


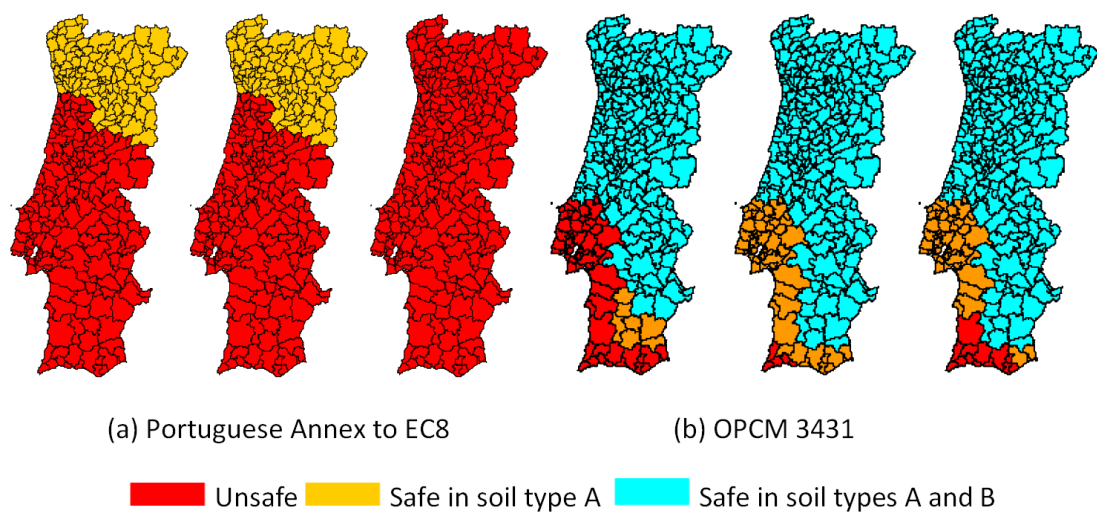
Fig. 15. Assessment of ultimate damage and deformed configurations.



(a) Seismic zones

(b) 1-to-3 storeys

Fig. 16. Safety mapping in Portuguese seismic zones by a pushover analysis.



(a) Portuguese Annex to EC8

(b) OPCM 3431

Fig. 17. Safety mapping in Portuguese seismic zones by linear elastic code based analysis.

3.4 Shaking table tests

Recently, shaking table tests on unreinforced masonry buildings have been carried out by University of Minho (Lourenço et al., 2013). Three-cell concrete blocks and a modified general purpose mortar are used for laying masonry units and for filling the vertical hollow cells (if reinforcement is placed there). The three cell concrete blocks present frogged ends, with 400mm length x 200mm thickness x 190mm height. The adopted residential prototype building is a two-story house with regular geometry as usual in modern residential row buildings, with an interstorey height of 3.0m, two opposite facades with a percentage of openings of approximately 14% and two walls without openings (here, north and south walls), corresponding to the walls limiting neighboring houses. The slab floors are in reinforced concrete and work as a rigid diaphragm.

Due to testing restrictions, such as the size and payload of the table, most shaking table tests are carried out on scaled models that can be considered representative of the prototype structures. The shaking table of the National Laboratory of Civil Engineering (LNEC) in Lisbon, Portugal, is among the largest in Europe with a platform plan dimensions of 4.6m x 5.6m and a payload of 400kN. It was therefore decided to build a reduced 1:2 scale model, taking into account adequate scaling laws. The limitations of the shaking table do not allow the implementation of a Cauchy-Froude law, because the additional masses required plus the model mass are larger than the payload of the table. Therefore, it was decided to consider only the Cauchy similitude law, which has been adopted in many testing programs, e.g. (Mendes and Lourenço, 2010) or (Tomažević, 2009). The tests were carried out using incremental seismic inputs of two uncorrelated signals with a total duration of about 10seg (in true scale), one for each horizontal orthogonal direction of the model, here characterized by the peak ground acceleration (PGA). The accelerograms at 100% input were derived from the proposed elastic response spectrum provided in the Eurocode 8 for Lisbon region (PGA in true scale = 0.15g), considering type 1 seismic action, ground type A and 5% damping.

The crack and damage patterns obtained for the last stage are shown in Fig. 18, with clear diagonal crack localization, damage spread over the entire model (all facades and the two stories), and onset of cracking at about 100% seismic input. When the test was completed, the model was heavily damaged and beyond repair though perfectly standing. Cracking developed at first and second floors as long horizontal and diagonal stepped cracks in the masonry piers and in the north and south walls at the concrete block-mortar interfaces. Damage in the units occurs at the corners (compressed toes), where crushing was found. Sliding movements develop along the continuous bed joint cracks but also along the diagonal cracks.

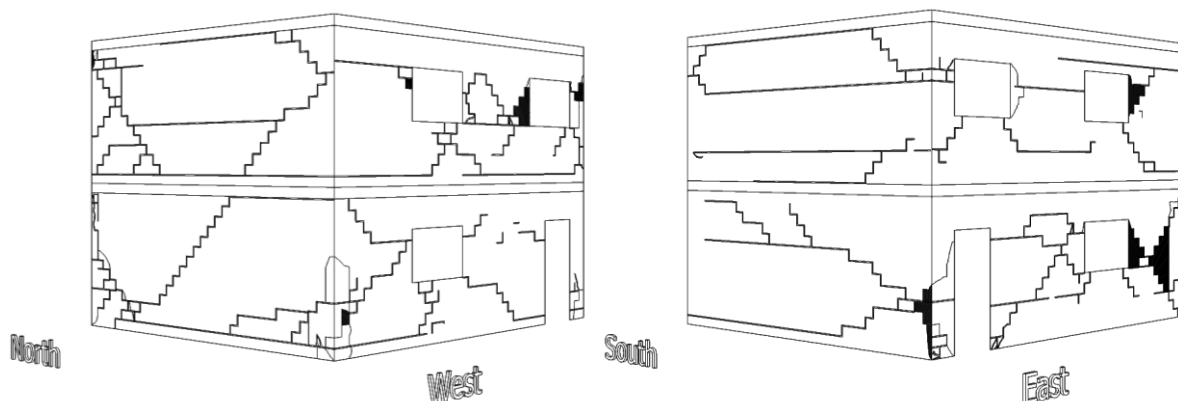


Fig. 18. Final damage patterns for seismic input 250% (about 0.6g).

The monotonic envelopes of the dynamic behavior in which the relation between the maximum base shear coefficients (BSC) developed in the models during each individual test run and corresponding values of total drift are given in Figure 19, where a reinforced masonry (RM) model is shown together with the unreinforced masonry (UM) model. Since inertial forces do not consider the components of damping and stiffness, the final base shear component at the base of the model has been estimated

considering only the masses. Consequently, the calculated forces should be considered as an approximation.

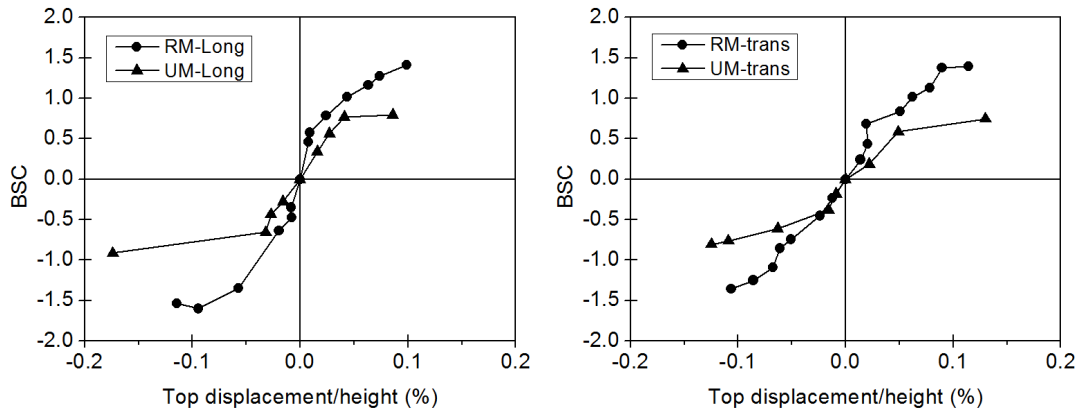


Fig. 19. Hysteresis envelopes.

The comparison of test results clearly indicates the higher resistance of RM model for both directions. Distinct behavior is observed regarding deformation in which the UM model attains maximum values of displacement but with lower resistance. This behavior clearly demonstrates the improvement in resistant capacity, stiffness performance and displacement response given by the reinforcement, which increases the shear strength and gives a more distributed pattern of cracking. The results also indicate clearly that the UM model reached the maximum capacity, with extreme damage, large lateral drifts and a rather ductile response, for a PGA associated with moderate to high seismicity values. Modern regular geometry unreinforced masonry buildings seem therefore to be suitable in low and moderate seismicity zones. Further shaking table tests to evaluate the influence of the geometrical eccentricity have just been performed with also good results, as only moderate torsion was found after cracking and increasing non-linear behavior in the present model.

4. INNOVATION IN MASONRY SYSTEMS USING TRUSS REINFORCEMENT

Next, a research carried out on two different modern masonry systems is briefly reviewed. The first wall system is co-sponsored by the lightweight concrete masonry block industry, where different possibilities of unreinforced and confined masonry walls are envisaged. The second system of masonry walls involves the hollow concrete block masonry industry and deals with the development of innovative systems for reinforced masonry walls. The proposed wall systems should fit the requirements of strength to horizontal loads as the behavior of masonry shear walls is fundamental in the design of masonry buildings subjected to different horizontal actions. On the other hand, the masonry systems should not require major changes in the traditional workmanship. Therefore, two different possibilities were adopted for the wall system: combined vertical and horizontal truss reinforcement and confined masonry.

4.1 Lightweight concrete masonry walls

The lightweight concrete blocks adopted in the testing program are regularly produced by the industry to comply with thermal regulations and have nominal dimensions of 400×320×200mm. A standard half block in terms of height and length was used in the tests. After cutting this half block in two pieces, the resulting half scale block has dimensions of 200×143×100mm, as shown in Figure 20. The adopted mortar is a pre-mixed mortar with 10 N/mm² of compressive strength. The shape of the block's ends enables an improvement on the contact surface in case of absence of the mortar in the vertical joints, which simplifies the construction to a great extent, and reduces possible clearances.

Reinforced walls are built by considering bed joint reinforcement, prefabricated truss type reinforcement Murfor® RND/Z, placed at the horizontal joints, see Figure 21. Note that the bed joint reinforcement is shown in the wall plan section. The horizontal reinforcement aims at increasing the ductility and lateral strength of the walls when submitted to cyclic horizontal loads. For confined

masonry walls, lightly reinforced concrete elements are added, vertically and horizontally. The bed joint reinforcement can be either connected or disconnected to confining vertical elements.

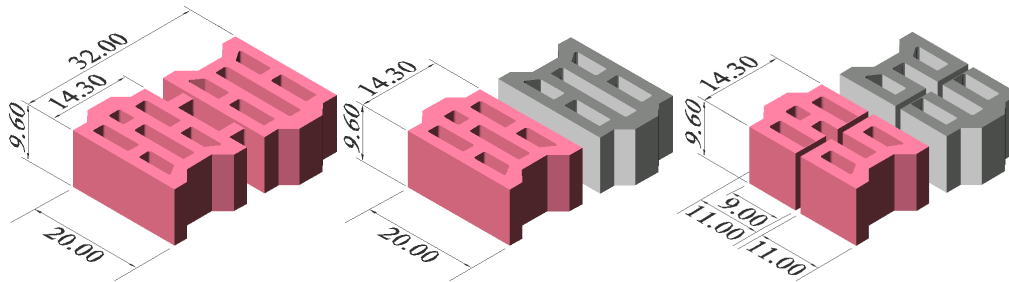


Fig. 20. Half-scale and reduced-size of block.

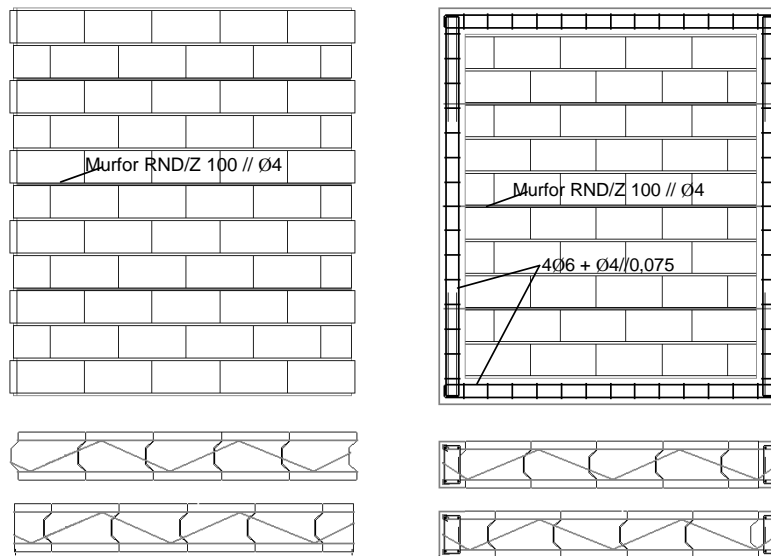


Fig. 21. Examples of unconfined and confined lightweight concrete masonry walls.

4.2 Hollow concrete masonry walls

Within the scope of this project, two distinct building systems are proposed for reinforced masonry solutions. Both systems are based on hollow concrete masonry units, whose geometry and mechanical properties have been adequately specified. Two and three hollow cell concrete masonry units were developed in order to accommodate vertical reinforcement, providing building systems BS1 and BS2, respectively. The concrete block with three hollow cells is designed to accommodate uniformly spaced vertical reinforcement, see Figure 22. In order to allow expedite and economical testing of a large number of masonry walls, it was decided to produce half scale units.

The first building system BS1 is composed by the two hollow cell concrete masonry units, where the vertical reinforcement is placed in a continuous vertical joint, by adopting the masonry bond indicated in Figure 22c, and the horizontal reinforcement is placed in the bed joints. Prefabricated truss type reinforcement is again used for the vertical and horizontal mortar joints. This system enables easy placing of full and half units on the wall after the positioning of the continuous vertical reinforcement, in agreement with the traditional techniques commonly used for the construction of unreinforced masonry walls. An important aspect to be taken into account during the construction is the appropriate filling of the vertical reinforced joints so that suitable bond strength between reinforcement and masonry can be reached, and an effective stress transfer mechanism exists between both materials. Apart from the mechanical requirements of the blocks to be used on structural purposes, this system can be reasonably adopted by the Portuguese contractors since it uses well know masonry units and no additional changes in the building process are needed. It is noted that a possible alternative consists of

placing the vertical reinforcement inside the hollow cells. The second building system BS2 uses the three hollow cell concrete units, see Figure 17b. If traditional masonry bond is used, vertical reinforcement (Murfor RND/Z) can be introduced both in the internal hollow cell and in the hollow cell formed by the recessed ends. Continuous and overlapped vertical reinforcement is possible, using half units or full units. In both solutions above, proper filling of the vertical hollow cells is a major issue since it is intended to substitute grouting of the cells by general purpose mortar used for the bed joints, in order to simplify the system. Therefore, a mortar with adequate workability and flow properties must be adopted (Haach et al., 2011a).

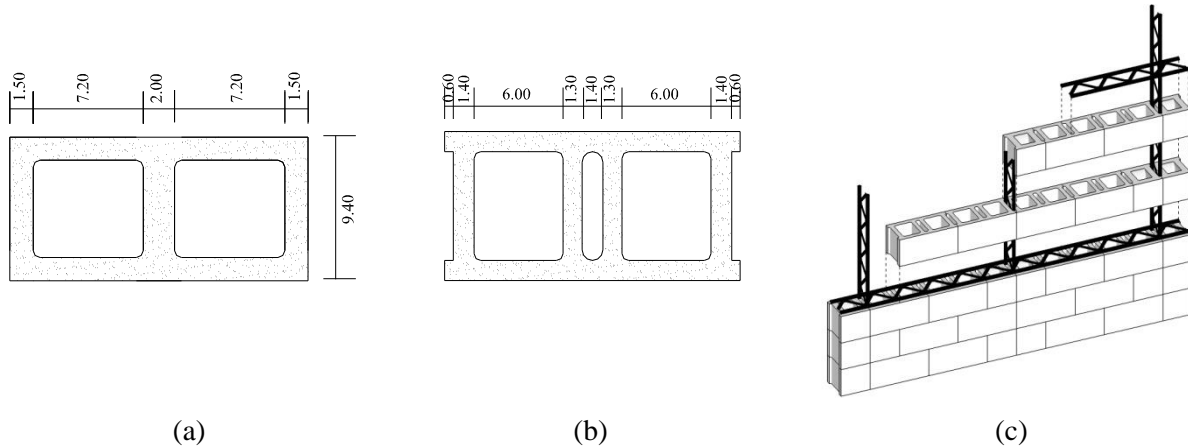


Fig. 22. Half scale concrete blocks: (a) two-cells; (b) with reinforcement pocket; (c) wall.

4.3 Test set-up

The behavior of masonry shear walls is fundamental in the design of masonry buildings subjected to different actions, namely of seismic nature. The performance of each system to seismic actions was evaluated by means of a large experimental program based on in-plane cyclic tests. The tests were performed by following the traditional procedure commonly used on masonry walls under combined vertical-cyclic horizontal loading. Two unreinforced lightweight concrete masonry wall configurations have been considered, assuming filled and unfilled vertical joint. In the latter, the benefit of using bed joint reinforcement was analyzed. Such configurations have been tested again using confined masonry, always assuming unfilled vertical joints. Confining concrete elements have been made using self-compacting concrete. The testing program for the hollow concrete masonry walls included walls built according to systems BS1 and BS2 using different percentage of vertical and horizontal reinforcement, different location for the vertical reinforcement (in continuous vertical joints or also inside a hollow cell) and different vertical pre-compression loads.

The typical test setup used in the in-plane cyclic tests is displayed in Figure 23. The cantilever wall is fixed to a steel beam connected to the reaction slab through steel rods in order to preclude any movement. The pre-compression loading was applied by means of a vertical actuator with reaction in the slab given by the steel cables. A stiff steel beam is used for the distribution of the vertical loading and a set of steel rollers were added to allow relative displacement of the wall with respect to the vertical actuator. The seismic action is simulated by imposing increasing static lateral displacements by means of a hinged horizontal actuator appropriately connected to the reaction wall at mid-height of the specimen. The vertical load was applied with an actuator designed to keep the vertical load constant. Therefore, vertical displacements are allowed in the top steel beam. The horizontal cyclic load was applied to the wall via controlled displacement. Two full displacement cycles were programmed for each amplitude increment, aiming at strength and degradation assessment.

4.4 Test results for lightweight concrete masonry walls

Figure 24 illustrates typical failure modes obtained for the walls tested. In the walls without bed joint reinforcement, initially flexural behavior dominates with horizontal cracks appearing at the bottom and top of the walls. With increasing application of horizontal displacement, a diagonal shear crack

appears, usually well-defined and with sudden occurrence for a given orientation of the loading. With the load increase and inversion of load direction, additional diagonal cracks appear. In the walls with light bed joint reinforcement, the strength deterioration is slow and more distributed cracking occurs. At ultimate stage, cracking is much more severe as the ultimate displacement is much larger. In confined masonry walls, the steel bars of the confining elements are severely stressed, with considerable cracking of these elements. In these walls, masonry crushing was also observed at final stage due to the larger number of cycles applied.

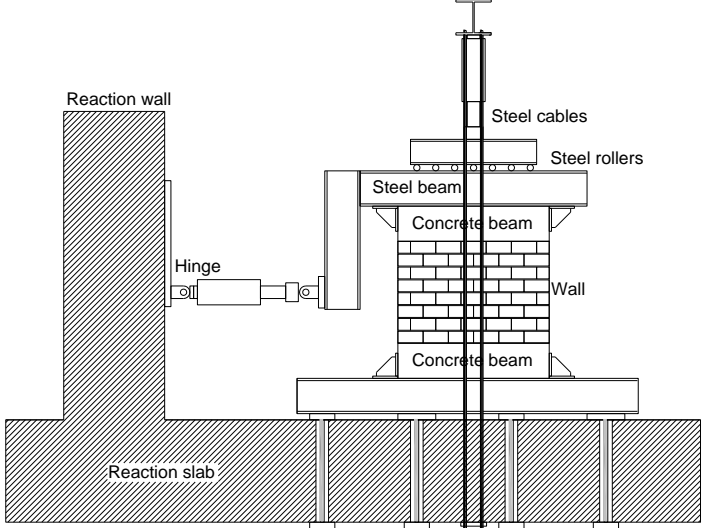


Fig. 23. Front view of the test setup.

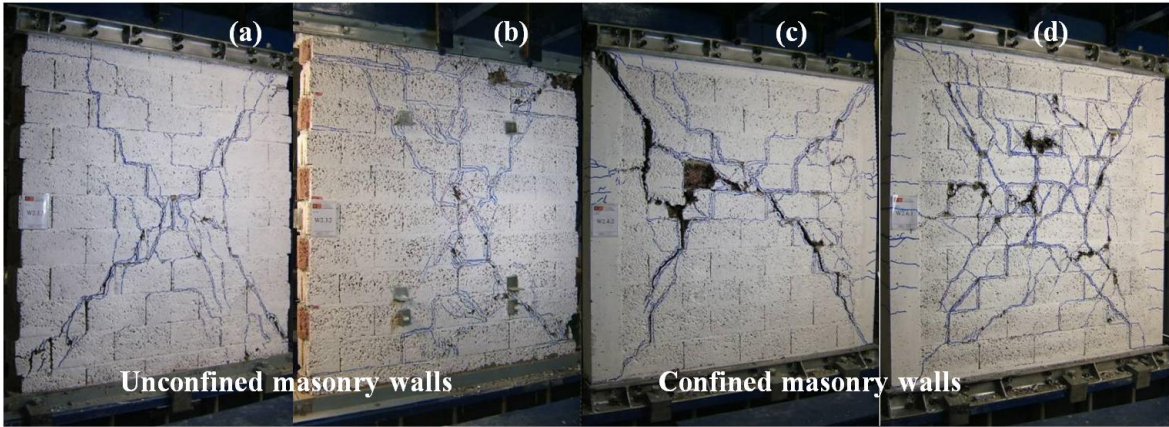


Fig. 24. Failure modes for lightweight concrete masonry walls: (a) unreinforced; (b) lightly horizontally reinforced; (c) confined unreinforced; (d) confined and horizontally reinforced.

From the analysis of the experimental results, the following observations can be made: (a) the addition of bed joint reinforcement in standard unreinforced masonry contributes to a very low increase of the shear resistance (5 to 10%). The horizontal displacements are also increased marginally, with a typical lateral drift at peak of 0.21%. The addition of bed joint reinforcement in confined masonry contributes to a moderate increase of the shear resistance (about 20%). Confined masonry walls have a shear strength increase of about 20%, when compared to unreinforced masonry. The horizontal displacements increase also, leading to a ductility about 20% larger than unreinforced walls. The typical drift at peak is about 0.45%; (b) the theoretical resistance (using the bilinear diagram) is about 75% of the maximum experimental resistance, see (Medeiros et al., 2013).

4.5 Test results for hollow concrete masonry walls

Figure 25a,b illustrates typical failure modes obtained for hollow concrete masonry walls. The walls presented generally a well distributed cracking pattern, with crushing of masonry in the compressed toes. The influence of the amount of vertical load was clear, as higher vertical loads delayed cracking, which appear very close to peak load in this case. Comparing the behavior of the unreinforced masonry with the reinforced walls, it is possible to observe that the reinforcement makes masonry a more homogeneous material. Only the unreinforced masonry walls exhibited localized cracks with considerable opening, which divided the specimen into two parts. After the crack opening, the stress transfer between both parts is achieved almost exclusively at the bottom corners where compressive stresses concentrate. Figure 25c presents a typical experimental load-displacement diagram, where rather good ductility of the response can be observed. The reinforcement increases the wall strength and peak displacement and the increase in vertical load leads to a more brittle response. No significant differences are found between the walls with reinforcement placed inside the hollow cells or in a continuous vertical joint, see (Haach et al., 2010)..

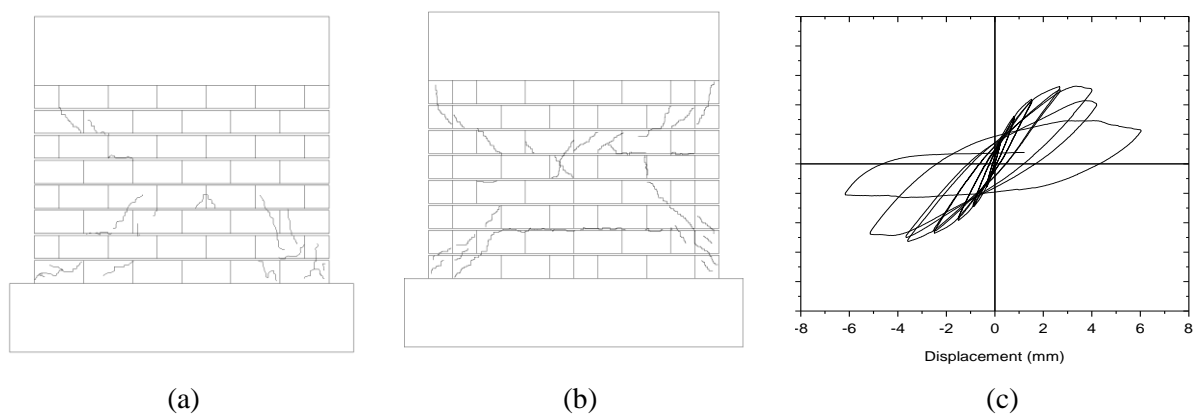


Fig. 25. Typical failure modes for hollow concrete masonry walls with (a) horizontal reinforcement, and (b) vertical reinforcement inside masonry cells or in vertical joints, and (c) typical horizontal force vs. horizontal displacement diagram.

4.6 Numerical analysis

Numerical simulations of the experimental programs aim at carrying out parametric studies that allow the definition of design rules appropriate to be included in the codes. The first step in the numerical simulations includes the validation of the modeling strategy adopted. For this purpose different material models included in DIANA® finite element code were considered, see (Haach et al., 2011b). Figure 26 illustrates typical results of the numerical analyses, which includes a comparison with experimental results and parametric studies taking into account the aspect ratio of the walls, the level of vertical pre-compression and the amount of reinforcement. A proposal for an adequate design approach and design charts have also been prepared, in order to allow practitioners to adopt the masonry systems developed.

5. CONCLUSIONS

Even if structural masonry is used for thousands of years, its use has gradually decreased and new approaches seem necessary in moderate to high seismicity countries. Unreinforced, confined and reinforced masonry solutions coexist but, in the first case, there seems to be incorrect perceptions on the capacity of masonry, in the second case, the amount of information seems limited and, in the third case, no international unified solution could be found. The local practices for reinforced masonry have very different levels of success with respect to market share. It has been shown that adequate methods for safety assessment of unreinforced masonry structures in seismic areas are available and adequate performance can be found in shaking table tests. Different technological systems have been proposed aiming at stimulating the use of modern masonry as an effective alternative to reinforced concrete

structures: confined lightweight concrete masonry and a novel reinforced hollow concrete masonry. Both proposed systems are characterized by minimal changes to the traditional workmanship.

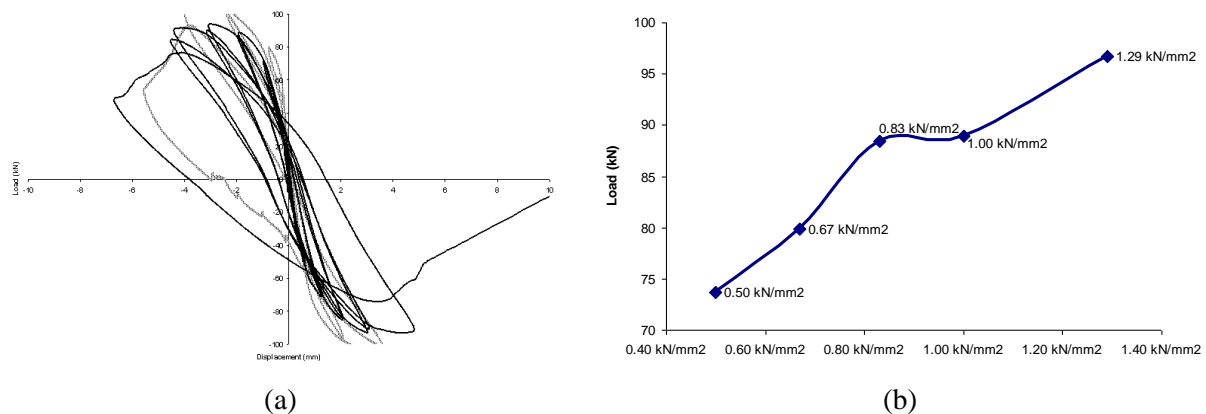


Fig. 26. Typical results for non-linear analysis: (a) validation of modeling through comparison with experimental results; (b) influence of a given parameter in the results (in this case, the vertical pre-compression).

ACKNOWLEDGEMENTS

The authors acknowledge the Portuguese Agency of Innovation (ADI), which partly financed the work presented in the scope of the national project ALVEST (n°5456), “Development of solutions of structural masonry”.

REFERENCES

- Adell, J. M., (2000) “Reinforced masonry”, Editorial Munilla-Leria (in Spanish).
- Augenti, N., (2004) “Seismic design of masonry buildings”, UTET Libreria (in Italian).
- Bruneau M. (1994) “State of the art report on seismic performance of unreinforced masonry buildings”, Journal of Structural Engineering ASCE 120(1), pp. 230-251.
- Ivo Calì, I., Marletta, M., Pantò, B., (2012) “A new discrete element model for the evaluation of the seismic behaviour of unreinforced masonry buildings”, Engineering Structures, 40(7)pp. 327–338.
- Gambarotta, L. and Lagomarsino, S., (1998) “On the dynamic response of masonry walls”, Proc. National Congress “La Meccanica delle Murature tra Teoria e Progetto”, Messina, 1996 (in Italian).
- Haach, V.G., Vasconcelos, G., Lourenço, P.B., Experimental analysis of reinforced concrete block masonry walls subjected to in-plane cyclic loading, Journal of Structural Engineering, ASCE, 136(4), p. 452-462 (2010)
- Haach, V. G., Vasconcelos, G., Lourenço, P. B., (2011a) “Influence of aggregates grading and water/cement ratio in workability and hardened properties of mortar”, Construction and Building Materials, 25(6), pp. 2980–2987.
- Haach, V. G., Vasconcelos, G., Lourenço, P. B., (2011b), “Parametrical study of masonry walls subjected to in-plane loading through numerical modeling”, Engineering Structures, 33(4), pp. 1377-1389.

- Lourenço, P.B., (2004) “Design of large size non-loadbearing masonry walls: Case studies in Portugal. Technical and economical benefits”, Proceedings of the 13th Int. Brick/Block Masonry Conference, July 4-7, 2004, Amsterdam, pp. 661-668.
- Lourenço, P.B., Avila, L., Vasconcelos, G., Alves, J.D., Mendes, N., Costa, A.C., (2013), “Experimental investigation on the seismic performance of masonry buildings using shaking table testing”, *Bulletin of Earthquake Engineering*, 11(4), pp. 1157-1190.
- Magenes, G. and Della Fontana, A., (1998) “Simplified non-linear seismic analysis of masonry buildings”, *Proc. of the British Masonry Society*, 8, 190-195.
- Magenes, G., (2006) “Masonry building design in seismic areas: Recent experiences and prospects from a European standpoint”, Proceedings of the First European Conference on Earthquake Engineering and Seismology, September 4-6, 2006, Geneva, Keynote 9, CD-ROM.
- Marques, R. and Lourenço, P.B., (2011) “Possibilities and comparison of structural component models for the seismic assessment of masonry buildings”, *Computers and Structures*, 89 (21-22), pp. 2079-2091.
- Medeiros, P., Vasconcelos, G., Lourenço, P.B., Gouveia, J., Numerical modeling of unconfined and confined masonry walls, *Construction and Building Materials*, 41(4), p. 968–976 (2013).
- Mendes, N., Lourenço, P. B., (2010) “Seismic assessment of masonry “Gaioleiros” buildings in Lisbon, Portugal”, *Journal of Earthquake Engineering*, 14, pp. 80-101.
- Modena, C., da Porto, F. and Valluzzi, M.R. (2004) “Reinforced and rectified clay blocks masonry”, Proceedings Sismica 2004, April 14-16, 2004, Guimarães, pp. 155-177.
- Mosele, F., da Porto, F., Modena, C., di Fusco, A., di Cesare, G., Vasconcelos, G., Haach, V., Lourenço, P. B., Beer, I., Schmidt, U., Brameshuber, W., Scheufler, W., Schermer, D., Zilch, K., (2006) “Developing innovative systems for reinforced masonry walls” Proceedings of the 7th International Masonry Conference, October 30-November 1, London, 2006, CD-ROM.
- Pasticier, L., Amadio, C. and Fragiaco, M., (2008) “Non-linear seismic analysis and vulnerability evaluation of a masonry building by means of the SAP2000 V.10 code”, *Earthquake Engineering & Structural Dynamics*, 37(3), pp. 467-485.
- Tomažević, M., (2007) “Damage as a measure for earthquake-resistant design of masonry structures: Slovenian experience”, *Canadian Journal of Civil Engineering*, 34, pp. 1403-1412.
- Tomažević M., (2009) “Seismic upgrading of old masonry buildings by seismic isolation and CFRP laminates: a shaking-table study of reduced scale models”, *Bulletin of Earthquake Engineering*, 7(1), pp. 293-321