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URM WALLS WITH THIN SHELL/WEB CLAY UNITS AND UNFILLED HEAD-JOINTS: CYCLIC IN-PLANE TESTS

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ABSTRACT

The use of thin web/shell clay unit masonry as a structural construction system is becoming increasingly widespread in several countries and in particular in many low seismicity regions of Europe for its good thermal and acoustic performance. Nevertheless, clay units with thin webs/shells could be a source of weakness and brittleness when used in structural masonry walls subjected either to high levels of vertical compression or in the case of shear cyclic excitations, especially if the construction is accomplished using thin layer mortar bed-joints. The aim of this research is hence to study the applicability and the use of such innovative types of units in moderate to medium seismic areas evaluating the seismic performance of different types of masonry with thin web/shells clay units through an experimental investigation followed by a numerical research. In this paper the attention is focused on the experimental cyclic response on five masonry walls made up by hollow "tongue and groove" clay units with thin webs/shells, void ratio of about 55%, unfilled head-joints and thin layer mortar bed-joints. The objective is the assessment of the cyclic in-plane response of masonry piers with different in-plane slenderness ratios and different vertical loads. Different failure modes have been observed and the associated force-displacement capacity is reported and discussed, with reference to the seismic performance of similar masonry typologies tested in the past.

INTRODUCTION

The effort of clay unit producers to implement innovative solutions regarding the sustainability and energetic efficiency of masonry products leads to the development of clay units with very thin webs and shells aimed to improve the thermal and acoustic insulation properties. Nevertheless, clay units with thin webs/shells could be a source of weakness and brittleness when used in structural masonry walls subjected either to high levels of vertical compression or in the case of shear cyclic excitations, especially if the construction is accomplished using thin layer mortar bed-joints.

For these reasons some structural codes of the past, like the Italian masonry code DM 20/11/1987 (1987) and the ENV version of Eurocode 8 (CEN, 1995), had included a prescription on the minimum thickness of the webs and of the shells for load-bearing units in order to provide a simplified criterion that, along with the limitation of the void ratio, should guarantee a sufficient robustness of the elements. Currently, the Italian Annex of EC6 (DM 31/07/2012, 2012) and the draft of the Italian technical norms for constructions (Draft NTC, 2012), include a prescription on the minimum thickness of the webs and of the shells for load-bearing clay units, which is set equal to 7 and 10 mm, respectively, and also provide a limitation of the vertical void ratio to 55% (this latter prescription also included in the current Italian norms of the constructions, NTC 2008, DM

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14/01/2008, 2008). In the case of use of structural masonry in seismic areas, these norms limit the void ratio of the units to 45%, prescribe units with continuous and rectilinear webs parallel to the wall plane with interruptions only in the grip holes, require minimum values of compressive strength of the blocks and of the mortar and forbid the use of ungrouted perpend-joints both with and without mechanical interlocking between units.

The aim of this research is hence to study the applicability and the use of such innovative types of units in moderate to medium seismic areas evaluating the seismic performance of different types of masonry with thin web/shells clay units through an experimental investigation followed by a numerical research. In the first instance, the assessment of the seismic performances of rather new structural masonry typologies requires experimental information, which is currently limited, concerning the in-plane failure modes, the associated lateral strength and stiffness, the displacement or drift capacity, the different performance and damage limits, the energy dissipation capacity from the hysteresis cycles which are among the main aspects affecting the global seismic response of a building.

Within this research, the results of an extensive experimental campaign carried out on a masonry typology, called “MA”, constituted by hollow clay “plain” units having thin webs and shells, with a nominal percentage of voids of 45% and bed-joints and head-joints completely filled with general purpose mortar, have already been reported in other works (Morandi et al., 2013a and Morandi et al., 2013b).

In this paper the attention is instead focused on the experimental response of structural masonry walls constituted by hollow “tongue and groove” clay units with thin webs/shells having a void ratio of about 55%, unfilled head-joints and thin layer mortar bed-joints (called typology “MB”). The necessity of experimental information is even more crucial for this type of masonry than for the type “MA” since, in addition to the possible sources of brittleness given by thin shell/web units and by thin layer mortar bed-joints, the elements do not also fulfil the requirements of the Italian norms for constructions in seismic zones regarding the void ratio limitation of the elements, set to be not larger than 45%, and the allowable types of perpend-joints, which must be grouted by mortar.

The experimental study here presented is constituted by pseudo-static in-plane cyclic tests on large scale walls in order to evaluate the in-plane response of the masonry piers under controlled boundary conditions, with different in-plane slenderness ratios and applied vertical loads. The cyclic tests have been anticipated by tests of geometrical and mechanical characterization on units, mortar and masonry wallets.

A critical comparison between the lateral response of the masonry typology “MB” and the typology “MA” is also performed, along with a discussion on the results of past researches on masonry with “tongue” and “groove” clay units and dry head-joints.

DESCRIPTION OF THE MASONRY TYPE AND TESTS OF CHARACTERIZATION

The masonry typology considered in this work is constituted by hollow clay “tongue and groove” units with thin webs and shells, thin layer bed-joints and unfilled head-joints, as illustrated in Fig.1(a) and Fig.1(b).

The nominal dimensions of the units are 225 mm (length) x 350 mm (width) x 230 mm (height) with a percentage of voids of about 55% and a gross dry density of 750 kg/m^3. The webs and shells have a minimum thickness of about 5.0 and 6.0 mm, respectively and the webs parallel to the wall plane are continuous and rectilinear with interruptions only in the grip holes. The units are rectified on their surfaces in order to be used with thin layer mortar bed-joints. Following the classification of Eurocode 6 part 1-1 (CEN, 2004), the blocks belong to Group 3. A M10T pre-batched mortar-glue has been used for the thin layer bed-joints; the mortar contains sufficiently long fibres aimed at limiting the penetration of mortar in the holes of the units; during the construction of the masonry specimens, a special device (a “lay-mortar roll”) has been used for the execution of the bed-joint, with the result of having obtained a mortar film well distributed over the width of the elements and with a constant thickness of about 1.0 mm, as shown in Fig.1(c). The connection along the head-joints is only guaranteed by the mechanical interlocking between adjacent masonry units. In the construction of the masonry specimens, before the placement of the mortar bed-joints, the clay units had been dipped in water in order to prevent excessive water absorption by the units in contact with the bed-joints that could affect the mechanical characteristics of the mortar.

In accordance with the Italian Annex of EC6 and the current draft of the new Italian technical norms for constructions, the units here considered could not be used as structural elements since the requirements related to minimum values of thickness for webs and shells, set equal to 7 and 10 mm respectively, are not fulfilled. Furthermore, the clay elements do not also meet the requirements of the current Italian norms for constructions in seismic zones regarding the void ratio limitation of the elements, set to be not larger than 45%, and the type of the head-joints, which must be grouted by mortar.

Before the execution of the in-plane cyclic tests on large scale masonry walls, a series of tests on clay units, on mortar and on masonry assemblages were performed at the laboratory of the Department of Civil Engineering and Architecture of the University of Pavia, in order to quantify the main mechanical properties of the masonry. The results are summarized in Table 1. Tests for the evaluation of the compressive strength of the units were carried out, both in the load-bearing direction (i.e., under vertical compression) and perpendicular to the load-bearing direction (i.e., under lateral compression); moreover, experiments for the determination of flexural and compressive strength on the hardened glue were executed, according to the European standards. Furthermore, vertical compression tests on six masonry wallets were executed in accordance to EN 1052-1 (2001); in this kind of tests the two bearing surfaces have been capped with a layer of high strength gypsum, in order to avoid strength concentrations. Each specimen was instrumented with six displacement transducers, more precisely one horizontal and two vertical transducers applied on the front and on the back face, in order to measure the deformations. Finally, tests for the determination of the initial shear strength in the plane of the thin bed-joints (f_{vi}) were carried out according to EN 1052-3 (2007). In detail, three samples made up by three units (triplets) were tested, in the direction parallel to the bed-joints, at three different levels of precompression (equal to 0.1, 0.3 and 0.5 MPa, respectively) in the direction orthogonal to the bed-joints. According to EN1052-3, since the normalised compression strength of the units was larger than 10 MPa, higher values of precompression should have been used (0.2, 0.6 and 1 MPa); however, as the execution of a test with a precompression of 1 MPa has provide a failure in the units only, in order to not exclude possible failures in the bed-joints, the lower range of precompression levels was used.

(a) (b) (c)

Figure 1. (a), (b) construction of the masonry walls; (c) device for the execution of the thin mortar bed-joints

Table 1. Results of the tests of mechanical characterization

	Mean values [MPa]	St. deviation [MPa]	c.o.v. [%]	Characteristic values [MPa]
Vertical compression strength of the units, f_b (EN 772-1, EN 771-1)	10.5 (12.0)	0.91	8.7	9.0
Lateral compression strength of the units, f'_b (EN 772-1, EN 771-1)	1.7 (1.9)	0.42	24.7	1.2
Compression strength of the mortar, f_m (EN 1015-11)	13.4	1.18	8.8	-
Compression strength of the masonry, f (EN 1052-1)	6.2	0.60	9.7	5.2
Modulus of elasticity, E	6100	403	6.6	-
Initial shear strength (from triplet tests), f_{v0} (EN 1052-3)	0.49	-	-	0.40
Friction coefficient (from triplet tests), μ (EN 1052-3)	1.04	-	-	0.83
	(in brackets the normalized values)			

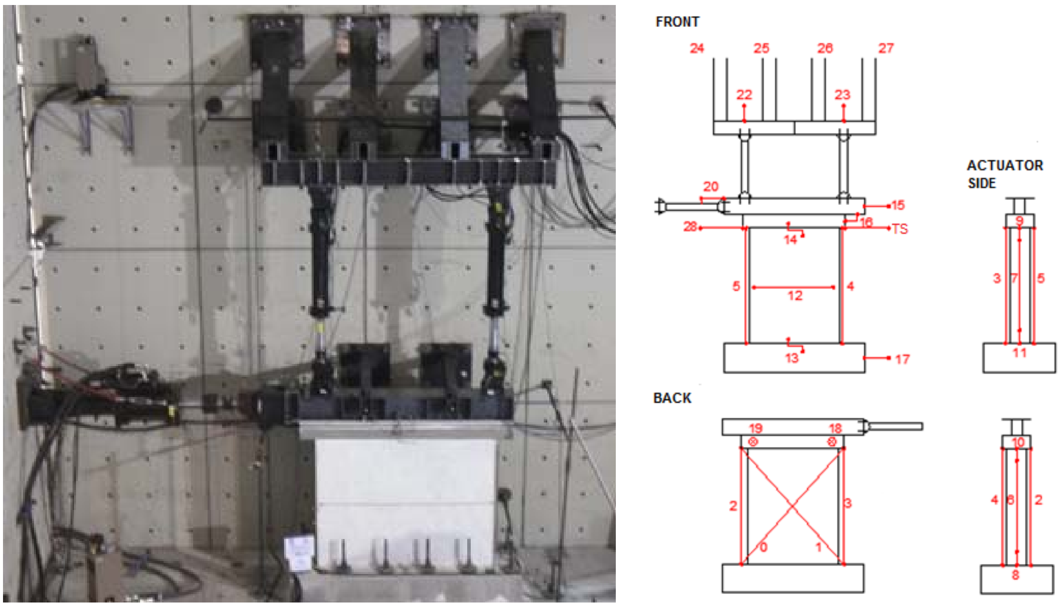
EXPERIMENTAL SETUP AND INSTRUMENTATION ON THE WALLS

The in-plane cyclic tests were carried out at the EUCENTRE Laboratory for Seismic Testing of Large Structures. The test setup took advantage of the three-dimensional configuration of the strong floor and the two orthogonal strong walls of the laboratory.

The walls were built on 400 mm thick reinforced concrete footings, which were clamped to the strong floor by means of post-tensioned steel bars. A horizontally mounted servohydraulic actuator, fixed on the strong wall perpendicular to the specimen, applies a horizontal shear force to the top of the wall through a steel spreader beam; the steel beam is stiffened with steel plates orthogonal to its length and is restrained by a sliding restrainer system that avoid out-of-plane deflections of the wall. Two vertical servohydraulic actuators apply the vertical load on the wall, reacting on a steel frame fixed on the strong wall parallel to the specimen. The test setup is shown in Fig.2(a).

Fig.2(b) schematically shows the adopted wall instrumentation. The horizontal load was measured by a load cell positioned in the horizontal actuator, while the horizontal displacement at the top of the wall is controlled by an external linear potentiometer. Twenty eight displacement transducers (linear potentiometers) were installed on each wall in order to measure the horizontal displacement at the top of the wall, the flexural and the shear deformations of the piers, the relative sliding displacements between the wall and the footing, between the top beam and the wall, between the steel spreader beam and the top beam and between the strong floor and the footing and the out-of-plane displacements.

Additionally, a displacement optical acquisition system was used. The optical measurement system is based on a series of independent high definition cameras able to identify the positions of reflective markers attached on the surface of the specimens and follow their displacements during the cyclic tests.



(a) (b)

Figure 2. (a) Experimental set-up for cyclic in-plane tests; (b) layout of the displacement transducers

PROPERTIES OF THE LARGE SCALE WALLS AND TESTING PROTOCOL

A total of five masonry walls were tested. All specimens were 350 mm thick and 2.14 m high. The testing campaign included three "slender" walls (MB1, MB2 and MB3) with a length of 1.35 m and two "squat" walls (MB4 and MB5) with a length of 2.7 m. Three levels of vertical mean compression stress σ_v were applied: 0.15, 0.45 and 0.75 MPa. The ratios h/l (height over length of the masonry piers, equal to about 1.6 and 0.8) and the ratios σ_v/f (vertical stress over compression strength of the masonry, equal to 0.024, 0.073 and 0.105) were chosen to be the same of the ones used for the experimental campaign on typology “MA” (Morandi et al., 2013a), in order to allow a consistent comparison of the main seismic parameters obtained. These values of vertical stress were chosen to induce different failure modes. The dimensions and the vertical stress levels for the walls are summarized in Table 2.

All walls were tested with double fixed boundary conditions, applying a criterion which involves a mixed force-displacement control, imposing both a constant vertical load and a condition of free translation with no rotation of the top beam.

During the tests, programmed displacements have been cyclically imposed in both directions, with step-wise increased amplitudes up to the attainment of ultimate conditions of the specimens or of large values of horizontal top wall displacements (in this case, up to a drift of 2.0%). At each displacement amplitude, three cycles have been performed, forces acting on the walls and displacements were measured and, for each wall, the hysteresis loops were recorded.

Table 2. Masonry walls subjected to cyclic in-plane tests

Wall	l [mm]	t [mm]	h [mm]	σ_v [MPa]	σ_v /f [-]
MB1	1350	350	2140	0.15	0.024
MB2	1350	350	2140	0.45	0.073
MB3	1350	350	2140	0.65	0.105
MB4	2700	350	2140	0.45	0.073
MB5	2700	350	2140	0.65	0.105

EXPERIMENTAL LATERAL RESPONSE OF THE PIERS

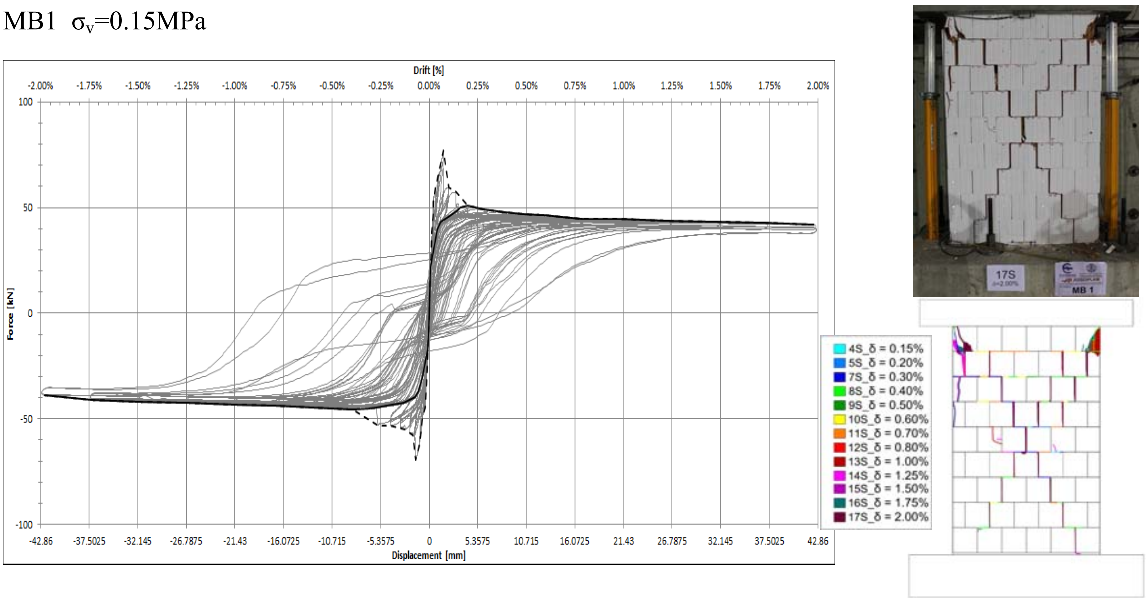
The results of the in-plane cyclic tests on the five specimens are here reported. Fig.3 and Fig.4 show, respectively for the three “slender” and for the two “squat” walls, the hysteretic force-displacement curves along with the overall envelopes, pictures of the damaged specimens and sketches of the cracking pattern at the end of the test.

The tested walls have manifested rather different lateral responses with various cracking patterns corresponding to different failure modes.

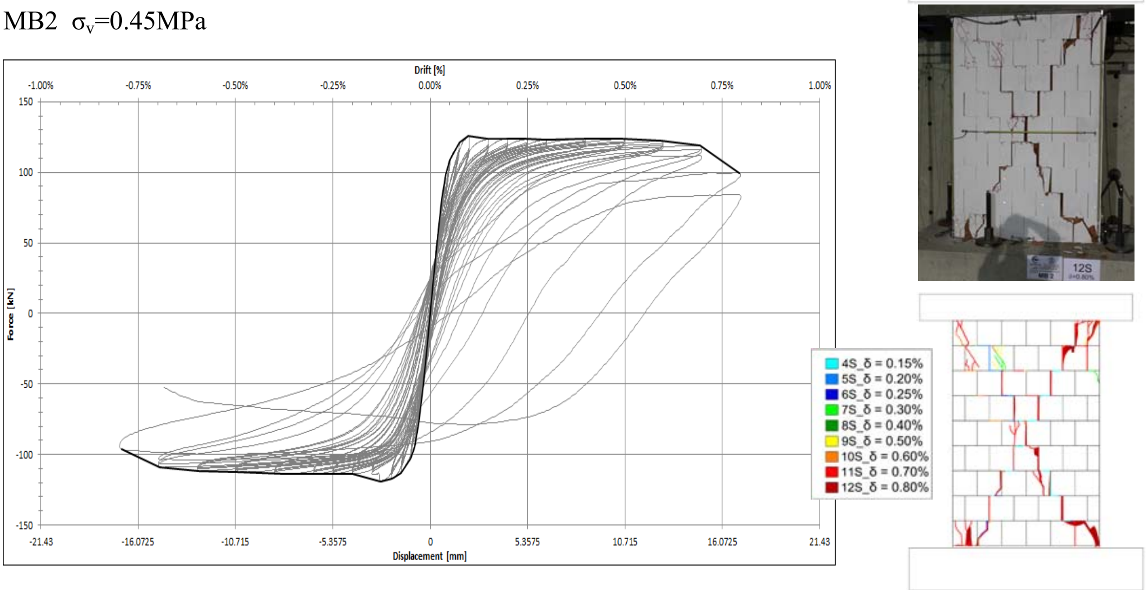
The “slender” wall MB1, subjected to a low level of vertical compression (σ_v=0.15 MPa), has displayed the first light cracks in the bed-joints at a drift of 0.15-0.20%. However, at very small applied drifts (about 0.05-0.10%), on the force-displacement curve it is clearly visible a peak in the force followed by an abrupt strength degradation up to a drift of about 0.20-0.25%, from which the force stays about constant with only a slight degradation up to the end of the test (at 2.0% drift); this peak force is probably related to the contribution of the tensile strength of the mortar bed-joints which, once exceeded, vanishes and produces the steep degradation in the strength. At 0.30% drift, the development of a step-wise diagonal cracking was started to be visible in the bed and head-joints. However, at the following drift level (0.40%), another step-wise diagonal cracking has formed along the diagonal of the wall in the other direction. Moreover, inclined cracks in the units at the two top corners of the wall have started to appear, with the beginning of the spalling of the unit shells at 0.80-1.0% drift; at 1.25% drift, new cracks in the units on one side of the wall have formed. With the increase of the applied top displacements, in the bi-diagonal step-wise cracks the horizontal width of the gap of the head-joints has increased, and the pier has developed an evident “gaping” behaviour with sliding in the bed-joints, also evident in the cycles with drifts larger than 1.25-1.50% where, in proximity of the changes in direction of the applied displacements, the variation of the force is low compared with the applied displacements. At the final drift of 2.0%, further diagonal cracks in the units of the top of the wall have formed, with a significant extent of damage in the corners.

The first visible damage on the 1.35 m long wall MB2, loaded with a vertical stress of 0.45 MPa, has appeared from a level of drift of 0.15%, with the occurrence of bi-diagonal step-wise cracks in the joints, further developed up to a drift of 0.25-0.30%, where also inclined cracks in the units have begun to manifest. After the drift of 0.40%, the extent of the cracks in the clay units grows up both in the centre of the panel and in the corners, along with the increase of the horizontal width of the gaps in the head-joints. At 0.60-0.70% drift, several further inclined cracks in the units have formed spreading out throughout the panel, as shown in Fig.3. At the following applied drift (0.80%), the spalling of

MB1 σ_v=0.15MPa



MB2 σ_v=0.45MPa



MB3 σ_v=0.65MPa

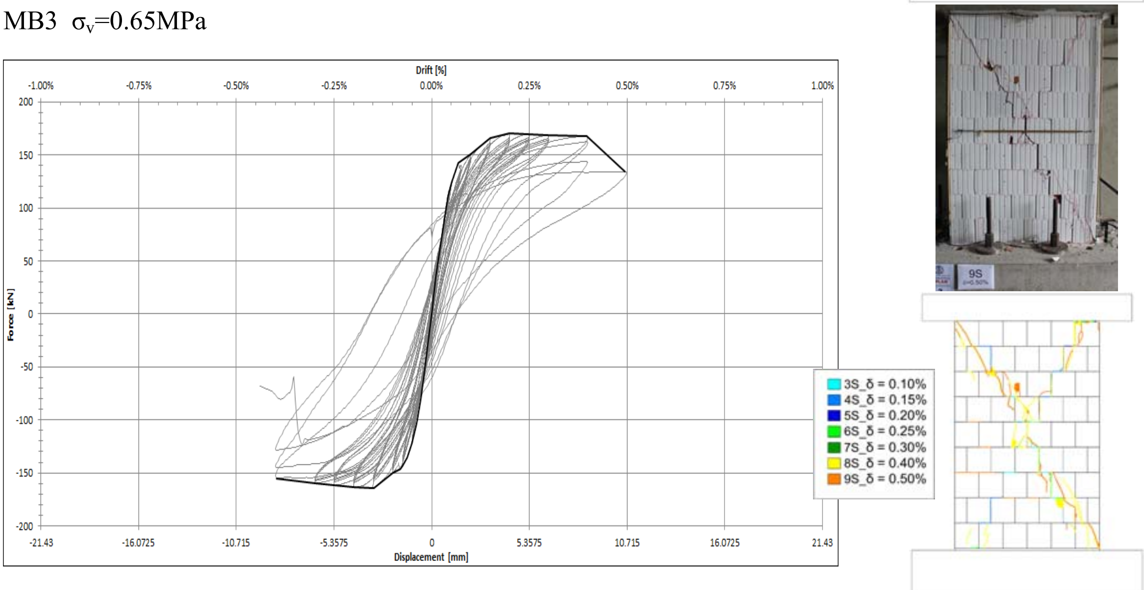
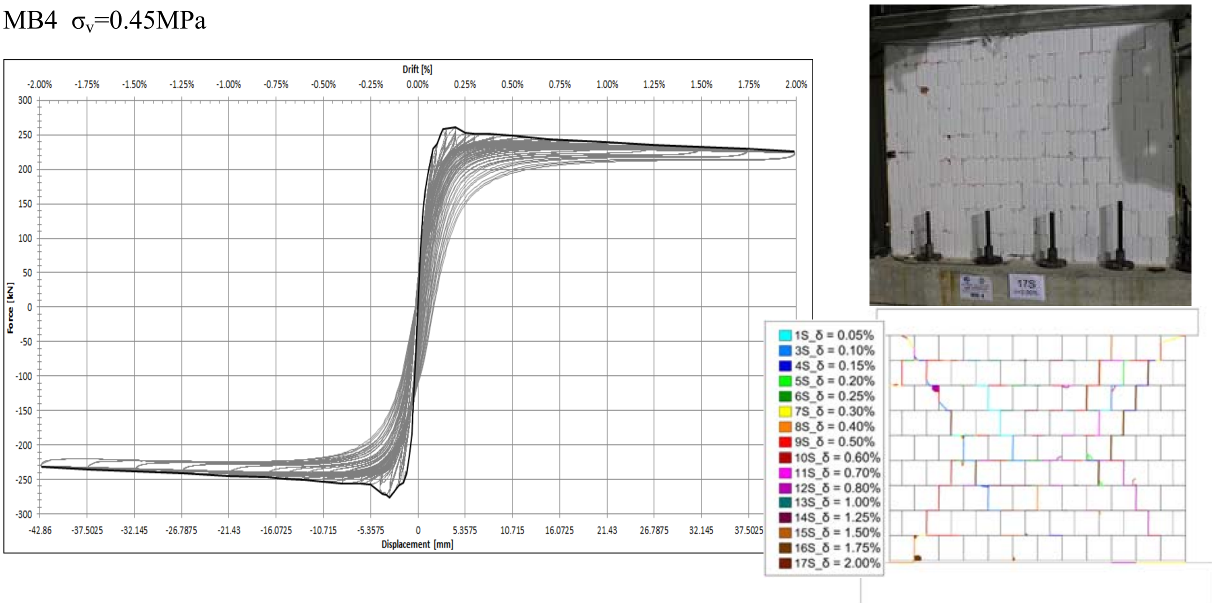


Figure 3. Force-displacement plots, images and sketches of the damage at the end of the tests on “slender” walls

MB4 σ_v=0.45MPa



MB5 σ_v=0.65MPa

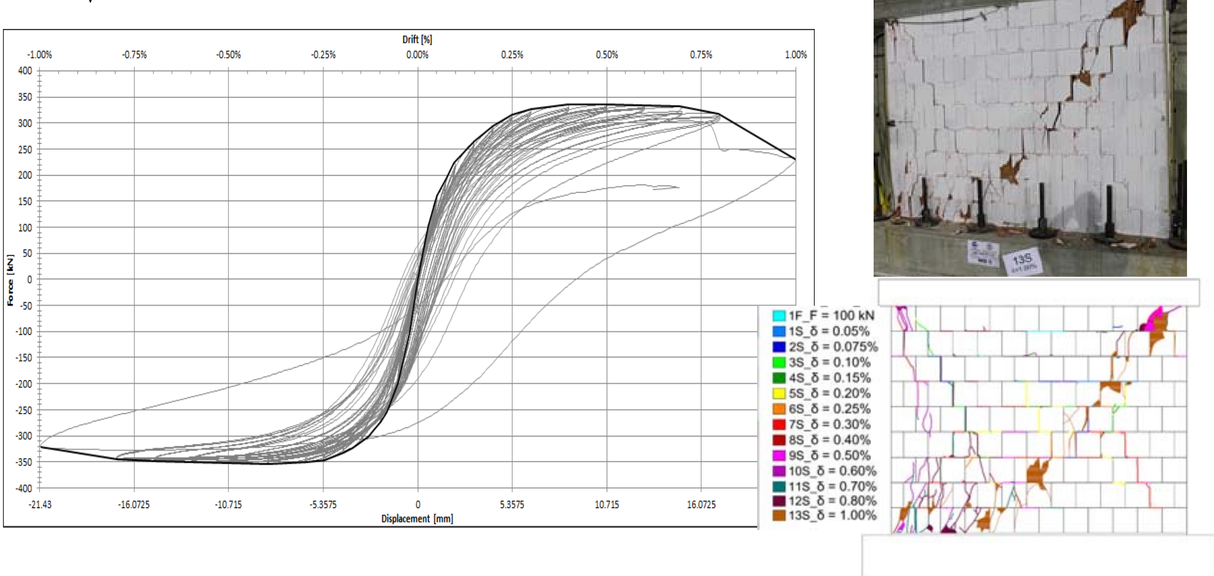


Figure 4. Force-displacement plots, images and sketches of the damage at the end of the tests on “squat” walls

many blocks has occurred and the test was stopped at the second cycle, when an abrupt strength degradation has happened and the wall has resulted to be very damaged. Therefore, this panel has developed an hybrid behaviour, firstly showing a “gaping” response with step-wise bi-diagonal cracking only, and, subsequently (mainly after a drift of 0.40%), with a typical shear failure mechanism with cracks in the units, made also evident examining the increasing energy dissipation of the last cycles of the test.

The most vertically loaded “slender” wall, the MB3, has showed the first visible damage at 0.10% drift; up to a drift of 0.30%, the damage has consisted in step-wise cracks in the joints and inclined cracks in the units. At the 0.40%, a crack from corner to corner along the two diagonals involving the units has occurred with a sudden strength degradation on the force-displacement curve. The test has continued also at the drift of 0.50%, but it was stopped at the first cycle since a very significant drop of the strength and a very extensive level of damage was attained. In the last cycles, the wall has thus shown a typical shear response with a bi-diagonal cracking mainly involving the units, which has produced a large energy dissipation.

The 2.70 m long wall MB4 was subjected to a vertical stress σ_v of 0.45 MPa and has developed a step-wise bi-diagonal cracking involving the bed and head-joints; the first cracks, in the bed-joints, have occurred at a drift of 0.05-0.10% and, increasing the level of applied displacement, the step-wise cracks have spread throughout the wall, in number, in length and in width. Moreover, at large level of horizontal drift, horizontal tension cracks were evident in the lower part and in the upper part of the wall, as in a “flexural-rocking behaviour”. The wall has therefore displayed a “gaping”/”rocking” behaviour with almost negligible damage in the units and small energy dissipation; the test was stopped at a drift of 2.0% without any significant strength degradation compared to the peak force (found between a drift of 0.10% and 0.20%); as in MB1, the reduction of the strength at small displacements could be related to the loss of the contribution of the mortar bed-joint tensile strength.

The other “squat” wall, the MB5, was tested with a lager value of vertical compression (σ_v=0.65 MPa) and has led to a different failure mode as respect to the wall MB4. The first evident cracks in the bed and head-joints have appeared at 0.05-0.10% drift and have become more severe in the following cycles, producing, at relatively low level of horizontal displacements, a step-wise diagonal cracking along the two diagonals of the wall (“gaping” behaviour). Subsequently, at 0.30% drift, some cracks have occurred in the units at the upper corners of the walls and, up to a drift of 0.50%, have kept on growing even with spalling of some shells, but being confined mainly in units of the corners; from a drift of 0.60% new diagonal cracks have formed in the units in other parts of the panel up to the attainment of the ultimate conditions at 1.0% drift, where the spalling of several units along the diagonal was evident and a sudden strength degradation was obtained, deciding to stop the test. The wall has displayed an hybrid behaviour, at the beginning with a step-wise diagonal cracking involving mainly the joints and, further in the test, with a shear response with failures of the units.

EVALUATION OF THE SEISMIC PARAMETERS OF THE TESTED WALLS

A common approach, followed by several studies in the past, to interpret the in-plane experimental response of masonry walls and to evaluate the seismic parameters of a non linear model to be used in non linear static (pushover) analyses, is to idealize the cyclic envelope of the hysteresis loops with a bilinear envelope. In Fig.5(a), a possible definition of the parameters of the bilinear curve is given (see Frumento et al., 2009).

The first step for the evaluation of the bilinear curve, is the construction of a cyclic envelope of the hysteresis loops, considering both positive and negative loading cycles, in order to evaluate the maximum lateral force and its degradation. Subsequently, the elastic stiffness k_{el} can be obtained by drawing the secant to the experimental envelope at $0.70 \cdot V_{max}$, where V_{max} is the maximum shear of the envelope curve. Finally, the ultimate displacement (δ_u) of the envelope curve is evaluated as the displacement corresponding to a strength degradation equal to 20% of V_{max}. The value of the shear V_u corresponding to the horizontal branch of the bilinear curve has been found by ensuring that the areas below the cyclic envelope curve and below the equivalent bilinear curve are equal. Knowing the elastic stiffness k_{el} and the value of V_u, it is possible to evaluate the elastic displacement δ_e as V_u/k_{el}; the ultimate ductility is defined as $\mu_u=\delta_u/\delta_e$.

However, since for each specimen, three loading-unloading cycles were carried out and three positive and three negative envelope and bilinear curves are obtained, it is needed to implement a procedure that allows to get only one bilinear curve for each tested wall. The ultimate displacement $\delta_{u,eq}$ is assumed as the lowest of the ultimate displacements in each of the three positive and three negative cycles, the elastic displacement $\delta_{e,eq}$ is instead assumed as the mean value of the elastic displacement for each of the three positive and three negative cycles and the ultimate ductility $\mu_{u,eq}$ is equal to the ratio between the ultimate displacement and the elastic displacement. The equivalent value of $V_{u,eq}$ has been assumed as the mean value of the V_u for each of the three positive and three negative cycles; the value of the equivalent elastic stiffness is therefore computed as $k_{el,eq}=V_{u,eq}/\delta_{e,eq}$.

It is important to point out that, in the majority of the tests performed in this research campaign, specimens did not reach a strength degradation of 20% of V_{max}. Therefore, in these cases, the ultimate displacement of the envelope curve is assumed equal to the maximum displacement reach at the end of the test.

In Fig.5(b), the comparison between the overall envelopes of the hysteretic cycles of the five walls is reported, along with the equivalent bilinear curves evaluated with the procedure here described. In Table 3, the main seismic parameters with the associated failure modes of the five tests are summarized.

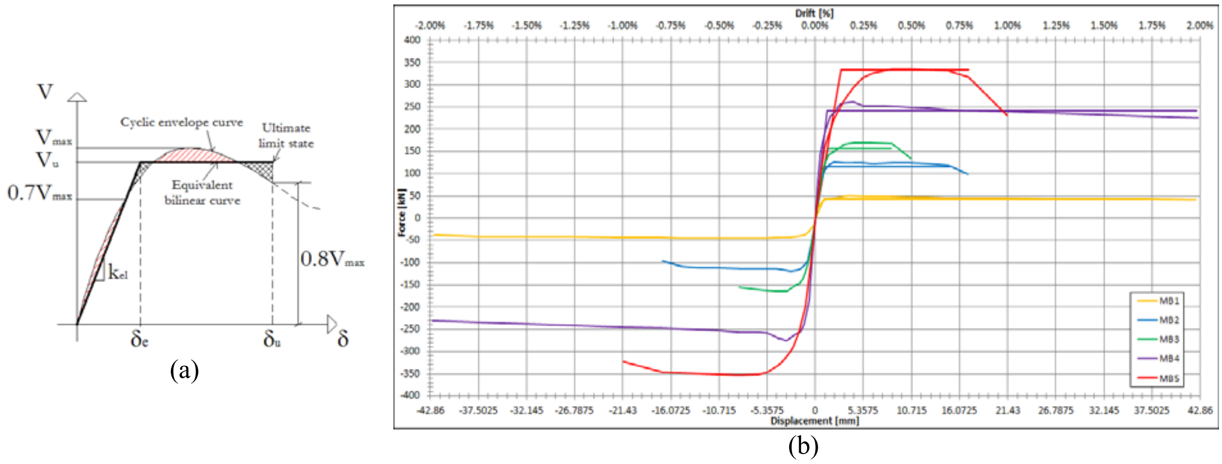


Figure 5. (a) Idealization of the cyclic response; (b) Overall envelopes of the cyclic response of the walls with the equivalent bilinear curves

Table 3. Main seismic parameters evaluated from the experimental results

Wall	l [mm]	t [mm]	h [mm]	σ_v [MPa]	Failure mode	$k_{el,eq}$ [kN/mm]	$V_{max,eq}$ [kN]	$V_{u,eq}$ [kN]	$(\delta_u/h)_{eq}$ [-]	$\mu_{u,eq}$ [-]
MB1	1350	350	2140	0.15	“Gaping”	46	48	44	1.73%	39.1
MB2	1350	350	2140	0.45	Hybrid	120	119	115	0.69%	15.5
MB3	1350	350	2140	0.65	Shear	115	164	157	0.40%	6.2
MB4	2700	350	2140	0.45	“Gaping”	184	263	241	1.98%	32.3
MB5	2700	350	2140	0.65	Hybrid	115	341	333	0.80%	5.9

ULTIMATE DEFORMATION, DUCTILITY AND ENERGY DISSIPATION CAPACITY

The ultimate drift capacities of the five tested walls in terms of $(\delta_u/h)_{eq}$ present a large variability, according to the different failure modes. The lowest value of drift capacity ($(\delta_u/h)_{eq}$=0.40%) has been found for the wall MB3, the only which has displayed a brittle pure shear mode with the occurrence of bi-diagonal cracking with failures mainly concentrated in the units. The wall MB1 and MB4 have instead shown a “gaping” mechanism, respectively associated to sliding and rocking, that has provided very large values of deformation capacity (drift capacity of 1.73% and 1.98% for MB1 and MB4, respectively). In the case of the walls MB2 and MB5, hybrid mechanisms have attained involving, at the beginning and with different extent, a step-wise diagonal cracking in the bed and head-joints (“gaping”) and, further in the tests, shear mechanisms with cracks in the units; in this case, intermediate values of drift between those of pure shear failure and “gaping” mechanism, have been obtained (0.69% for MB2 and 0.80% for MB5). As already pointed out in many other experimental campaigns (i.e., Magenes et al., 2008a; Tomazevic et al., 2012), the increase of the vertical compression level leads to the occurrence of more brittle failure modes and, therefore, a decrease of the ultimate deformation capacity.

Similar considerations to what described for the ultimate drift can be drawn about the ductility. The ultimate ductility provides rather scattered values: the walls MB1 and MB4 have showed very large values of $\mu_{u,eq}$ (larger than 30), whereas the walls involving shear mechanisms (being pure shear or hybrid “gaping”/shear) have obtained lower values. In particular, the ultimate ductility of the wall MB5 has resulted to be the smallest ($\mu_{u,eq}$=5.9) since the wall has provided a large value of the elastic displacement δ_e.

Finally, the dissipated hysteretic energy of the tested walls was examined, in terms of equivalent viscous damping which, given a single load–displacement cycle, can be expressed as a function of the dissipated energy W_d and the elastic energy at peak displacement W_e: $\xi_{eq}=W_d/2\pi(W_e^{+}+W_e^{-})$. In Fig.6 the results of the calculated equivalent viscous damping ξ_{eq} are plotted as a function of the displacement ductility (δ/δ_e) and of the drift (δ/H) of each cycle and considering the first, the second and the third cycle at each target displacement.

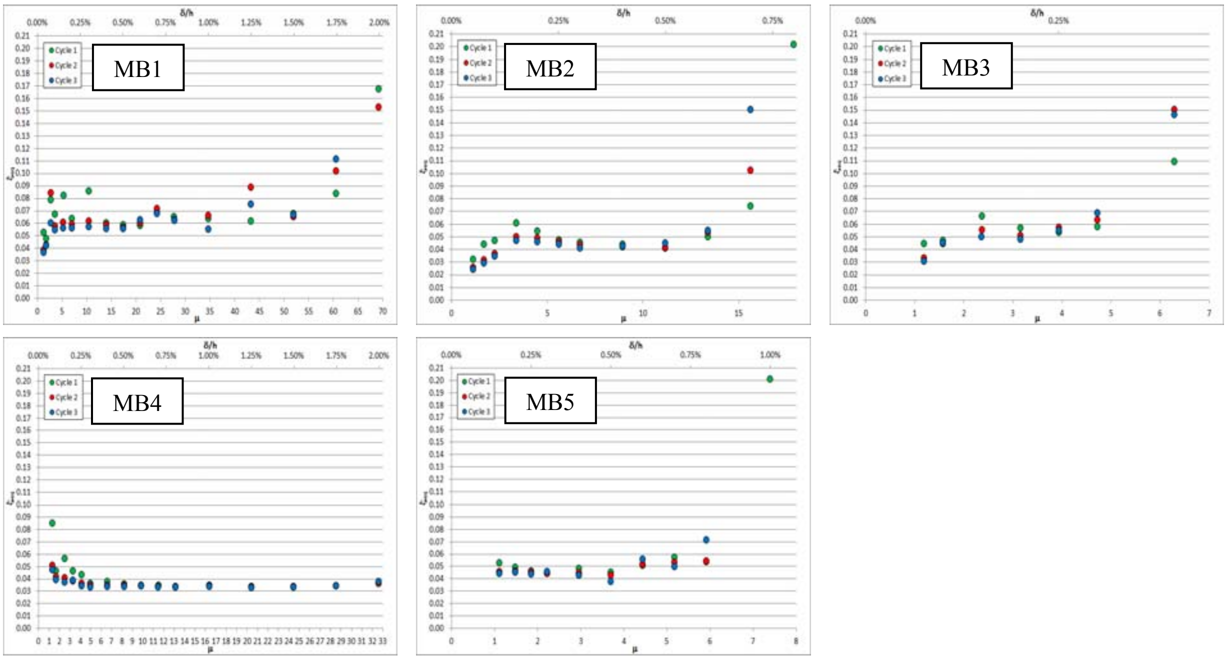


Figure 6. Equivalent viscous damping calculated from the hysteresis loop as a function of ductility ($\mu=\delta/\delta_e$) and drift (δ/H), where δ is the maximum displacement of the cycle.

It may be observed that the cycles have low dissipativity for all failures modes; the equivalent viscous damping ξ_{eq} was found to be less than 7-8% for almost all cycles, except the last ones, when cracking in the units started to develop.

COMPARISON BETWEEN THE MASONRY TYPOLOGY "MA" AND "MB"

As stated in the introduction, within the same project, a similar type of experimental research has been carried out on a masonry typology (called typology "MA") constituted by hollow clay "plain" units having thin webs and shells, with a nominal percentage of voids of 45% and bed and head-joints completely filled with general purpose mortar (see Morandi et al., 2013a and Morandi et al., 2013b). The main seismic parameters of the typology "MA" have been reported in Table 5. Since in the two test campaigns the in-plane slenderness (h/l), the wall thickness (t) and ratio (σ_v/f_m) were chosen to be the same, a consistent comparison of the main seismic parameters can be performed, making reference to Table 3 for the results of the typology "MB".

Table 5. Main seismic parameters of the typology "MA"

Wall	l [mm]	t [mm]	h [mm]	σ_v [MPa]	Failure mode	$k_{el,eq}$ [kN/mm]	$V_{max,eq}$ [kN]	$V_{u,eq}$ [kN]	$(\delta_u/h)_{eq}$ [-]	$\mu_{u,eq}$ [-]
MA1	1250	350	2000	0.50	Shear	113	130	123	0.23%	4.3
MA2	1250	350	2000	0.70	Shear	130	166	154	0.20%	3.4
MA3	1250	350	2000	1.00	Shear	154	198	177	0.24%	4.2
MA4	2500	350	2000	0.50	Shear	325	401	369	0.21%	3.7
MA5	2500	350	2000	0.70	Shear	354	500	472	0.24%	3.6
MA6	1250	350	2000	0.20	Flexure	48	27	24	1.59%	64.0

First of all, the failure modes occurred for the two masonry typologies are very different: all the walls "MA" had showed shear failures with the occurrence of cracks mainly in the units, with the exception of the wall MA6 (tested with a "cantilever" boundary condition) that had provided a flexural/rocking behaviour; on the other side, the specimens "MB" have developed different failure mechanisms, involving a step-wise diagonal cracking in the joints, apart from the most vertically loaded wall MB3 which has attained shear failures in the units as in the walls MA1 to MA5.

The differences between the two masonry typologies are evident also in terms of ultimate drift capacity and ultimate ductility; in fact, all the walls “MA” failing for shear, had manifested very low values of $(\delta_u/h)_{eq}$, between 0.20% and 0.24%, and of ultimate ductility $\mu_{u,eq}$, between 3.4 and 4.3. On the other side, the walls “MB” have provided much larger values of ultimate deformation capacity and ductility, even in the case of pure shear mechanism (values of $(\delta_u/h)_{eq}$ from 0.40% to almost 2.0 % and of $\mu_{u,eq}$ from about 6 to about 40). The fact that masonry systems made up by units with dry head-joints provide larger values of deformation capacity compared to fully mortared masonry systems with completely grouted perpend-joints, has been already considered in other experimental campaigns, as for example, in Morandi et al., 2009.

Furthermore, as already underlined in other experiences (i.e., see Morandi et al., 2009), the values of the lateral strength for fully mortared masonry typologies, as “MA”, is usually larger than for masonry types with dry head-joints, as the ”MB”; this outcome is also evident here, looking at the results on the “squat” walls whereas, for the “slender” walls, it is still valid but in a smaller extent.

Finally, apart from the small differences in the dimensions, the values of the elastic stiffness of the walls ($k_{el,eq}$) are larger for the typology “MA” than for the typology “MB”, given the higher value of the modulus of elasticity (E= 10800 MPa for “MA” and E= 6100 MPa for “MB”); also in this case, the results are noticeable in particular comparing the results on the “squat” walls.

CONCLUSIONS

The paper deals with the results of an experimental campaign on in-plane cyclic behaviour of an unreinforced masonry typology constituted by hollow “tongue and groove” clay units with thin webs/shells, having a void ratio of about 55%, unfilled head-joints and thin layer mortar bed-joints (called typology “MB”). The results have been presented and discussed in terms of cyclic response, deformation capacity and energy dissipation with reference to the specific failure mechanisms.

The interpretation of the lateral response on the five tested walls has not been straightforward, since different failure modes were found: two walls (the MB1 and MB4) have provided a “gaping” mechanism, two walls (MB2 and MB5) have displayed an hybrid “gaping”/”shear” mode and the wall MB3 has manifested a shear failure, mainly involving the units. The failure mechanisms do not appear to be significantly influenced by the in-plane aspect ratio of the panels but are affected by the different applied compression levels. The less loaded walls (the “slender” wall MB1 and the “squat” wall MB4), have showed a step-wise diagonal cracking in the joints with few cracks in the units (“gaping” mechanism) and are characterized by an imperceptible strength degradation and very large deformation capacity (ultimate drift up to about 2%). The walls MB2 and MB5 were instead subjected to larger compression levels, with the occurrence of step-wise cracking in the bed and head-joints followed by cracking in the units which, once widespread, has caused rather marked drops of the strength, resulting in an ultimate drift of 0.69% and 0.80% for MB2 and MB5, respectively. Finally, the most loaded “slender” wall, MB3, was characterized by a more brittle behaviour, with damage concentrated mainly in the units and, when two bi-diagonal cracks from corner to corner were formed, a sudden strength degradation occurred, producing the lowest ultimate drift of all the tested walls (0.40%).

From the comparison with the typology “MA”, tested previously within this project and constituted by fully mortared masonry with thin web/shell units and percentage of voids of 45%, the failure modes for the two masonry typologies were found to be very different, prevailing the more brittle shear mechanisms with diagonal cracking in the units for “MA” and providing much lower ultimate deformation capacity and ductility, albeit with a larger lateral strength and elastic stiffness.

In addition, it has appeared meaningful to put in relation the results of the cyclic tests evaluated here with the ones derived, within the ESECMaSE project (Magenes et al., 2008b), from a similar masonry typology realized with “tongue and groove” clay units (thickness of the webs and the shells respectively larger than 7 and 10 mm and void ratio of about 45%) with dry head-joints and thin M10 mortar bed-joints. The failure mechanisms for this latter masonry were found to be by shear with diagonal cracking from corner to corner of the walls, mainly with damage in the clay units; associated to these failures modes, very low ultimate drifts were obtained (0.21% and 0.33% for the 1.25 m and for the 2.5 m long walls, respectively). The masonry here tested has therefore manifested much larger values of deformation capacity in comparison of a similar masonry system tested in the past.

Further work will be dedicated to the interpretation of the results on the typology “MB” in terms of measured strengths.

Finally, in order to fully comprehend the applicability and the use of such new types of units in low to moderate seismicity areas, the seismic parameters here evaluated will be used for the execution of static non linear analyses on models of unreinforced masonry buildings to identify maximum design seismic levels and to check or revise design parameters as, for example, the behaviour factor q to be used in linear elastic analyses.

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REFERENCES

CEN (2004) Eurocode 6: Design of masonry structures – Part 1-1: Common rules for reinforced and unreinforced masonry structures, EN 1996-1-1:2005, Brussels, Belgium

CEN (1995) Eurocode 8: Design provisions for earthquake resistance of structures – Part 1-3: General rules – Specific rules for various materials and elements, ENV 1998-1-3:1995, Brussels, Belgium

DM 20/11/1987 Norme tecniche per la progettazione, esecuzione e collaudo degli edifici in muratura e per il loro consolidamento, Gazzetta Ufficiale, n. 285 del 5/12/1987 – S. 1987

DM 14/01/2008 Norme Tecniche per le Costruzioni (NTC 2008), Gazzetta Ufficiale, n.29 del 14/02/2008 – Supplemento ordinario n.30

DM 31/07/2012 Approvazione delle Appendici nazionali recanti i parametri tecnici per l’applicazione degli Eurocodici, Gazzetta Ufficiale, n.73 del 27/03/2013 – Serie generale

Draft of the New Technical Norms for Constructions (2012)

EN 771-1 (2011): Specifications for masonry units – Clay masonry units

EN 772-1 (2011): Methods of test for masonry units – Determination of compressive strength

EN 1015-11 (2007): Methods of test for mortar for masonry – Determination of flexural and compressive strength of hardened mortar

EN 1052-1 (2001): Methods of test for masonry – Determination of compressive strength

EN 1052-3 (2007): Methods of test for masonry – Determination of initial shear strength

Frumento S, Magenes G, Morandi P, Calvi GM (2009) “Interpretation of experimental shear tests on clay brick masonry walls and evaluation of q-factors for seismic design” Eucentre Research Report 2009/02, IUSS Press, Pavia, ISBN: 978-88-6198-034-1

Magenes G, Morandi P, Penna A, (2008a) “Experimental In-Plane Cyclic Response of Masonry Walls with Clay Units”, *Proceedings of the 14th World Conference on Earthquake Engineering*, October 12-17 2008, Beijing, China, Paper ID: 12-03-0095

Magenes G, Morandi P, Penna A, (2008b) “Test results on the behaviour of masonry under static cyclic in plane lateral loads”, ESECMaSE project, Department of Structural Mechanics, University of Pavia, report RS-01/08

Morandi P, Albanesi L, Magenes G (2013a) “In-plane experimental response of masonry walls with thin shell and web clay units” *Proc. of the Vienna Congress on Recent Advances in Earthquake Engineering and Structural Dynamics & 13. D-A-CH Tagung*, 28-30 August 2013, Vienna, Austria

Morandi P, Magenes G, Albanesi L (2013b) “Prove sperimentali per la valutazione della risposta sismica nel piano di pareti murarie in blocchi di laterizio a setti sottili” *Atti del XV Convegno di Ingegneria Sismica ANIDIS;* June 30-July 4 2013, Padova, Italy

Morandi P and Magenes G (2009) “Risposta sismica nel piano di pareti murarie in blocchi in laterizio alleggerito” *Atti del XIII Convegno di Ingegneria Sismica ANIDIS*, June 28-July 2 2009, Bologna, Italy

Tomaževič M and Weiss P (2012) “Robustness as a criterion for use of hollow clay masonry units in seismic zones: an attempt to propose the measure” *Materials and Structures*, 45 (4), 541-559