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INFLUENCE OF UNIT STRENGTH ON SHEAR-COMPRESSION BEHAVIOR OF CLAY MASONRY WALLS

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ABSTRACT

Extensive experimental research aimed at defining the mechanical behavior of load bearing masonry walls made with perforated clay units was carried out at the University of Padova. Fifty-one specimens were characterized by means of uniaxial and diagonal compression tests and also by in-plane cyclic shear compression tests. Three types of masonry walls were investigated: masonry made with units with mortar pockets, masonry made with tongue and groove units, masonry made with units with small dimensional tolerance in height and thin layer mortar.
Four types of non-linear finite element models were used in order to simulate the experimentally observed mechanical behavior. From the comparison of the obtained results, it was possible to draw some conclusions about the adopted experimental procedures, the different behavior of the three masonry typologies and the reliability of the modeling strategy to simulate the tests. On these bases, a simplified micro-modeling strategy, with the total strain rotating crack law for the units and interface elements for the masonry joints was chosen to reproduce monotonically the cyclic shear behavior of masonry.
This model was used to carry out parametric analyses of the tested masonry typologies, in order to assess the influence of the unit compressive strength on the tested specimens global shear behavior. A range of strength between 5 and 20 N/mm^2 was investigated. In the present contribution, the results of the parametric finite element analyses carried out are described and discussed.

KEYWORDS: load bearing unreinforced masonry; in-plane cyclic tests; finite element modeling; unit strength.

INTRODUCTION

The development of new systems for constructing unreinforced masonry walls has been driven by the market towards systems which have better thermal insulation properties and allow faster and cheaper construction processes. Because of these developments, the latest construction technologies have replaced traditional head joints, fully filled with mortar, with preformed pockets for mortar infill at the lateral faces (units with mortar pockets) or with mortarless head joints with mechanical interlocking between units (tongue-and-groove units). Traditional bed joints have also been replaced by thin-layer mortar joints, which are laid on edge-ground units (thin-layer joint masonry). One important aspect is that these construction technologies have been developed in countries not prone to seismic risk. Therefore, despite standard mechanical characterization, which can be obtained by means of relatively simple tests (see [1], for a concise literature review), the seismic behavior of these masonry types is still not adequately characterized nor it has been regulated by seismic codes [2].

Several advanced computational approaches are also available for the assessment of seismic behavior of masonry. The non-linear finite element modeling has been recognized as a general and efficient method for the analysis of load bearing and displacement capacity of masonry systems. In general, a numeric representation of masonry can be achieved by separately modeling masonry constituents (units and mortar joints, micro-modeling approach), or following a global approach where the whole structure is schematized as a continuum without any distinction between masonry constituents (macro-modeling). Micro-modeling strategy for masonry has mainly focused on the development of reliable interface models, since the first introduced by [3]. [4] developed an interesting interface model under multi-surface plasticity theory, where not only shear and tensile but also compressive behavior can be taken into account through a cap model. [5], has further developed this interface model with a refined description of the dilatancy phenomenon. An appropriate modeling of cracks through units is also of basic importance to avoid an over stiff response and a considerable higher failure load of the numerical models when compared with the experimentally determined ([4]; [6]). In order to simulate the distributed cracks which develop on the blocks and consequently to better reproduce numerically the global behavior of the structure using a micro-modeling approach, a smeared crack model should be applied to the blocks ([7]; [8] and [9]).

The mechanical parameters can be derived both from experimental data or deduced from homogenization techniques ([10]; [11]; [12]). Micro-modeling strategy is more detailed and is a valuable tool to reproduce masonry assemblages tested during experimental researches. It requires a large number of parameters but facilitates understanding the local behavior of masonry and results in parameterization of experimental campaigns. Conversely, such strategy is not suitable for simulating the global behavior of buildings, since computational burden is usually excessive. Macro-modeling approach is less detailed, but depends on a limited number of parameters. It can be suitable for structures of large dimensions, thus becoming more attractive for practice oriented analyses.

In this scenario, experimental research aimed at defining the mechanical behavior of load-bearing masonry walls made with vertically perforated clay units and various types of head and bed joints has recently been carried out by means of uniaxial compression, diagonal compression and shear compression tests. Three types of clay masonry were studied. According to the experimental results, methodical process of model calibration was carried out in order to obtain one single set of parameters that can be used with different modeling strategies and is able to describe different types of test. Experimental behavior was reproduced by four types of non-linear finite element models. From the obtained results, it was possible to draw some conclusions about the adopted experimental procedures, the behavior of the three masonry types, and the reliability of the various modeling strategies (as presented in [13]), in order to use the most consistent one to carry out parametric analyses.

EXPERIMENTAL PROGRAM

The main aim of the experimental program was to assess the in-plane cyclic behavior of three types of load bearing masonry walls made with vertically perforated clay units and various types of head and bed joints. Masonry made with pockets for mortar infills (Po), according to the Eurocode 6 [14], is classified as having fully filled head joints, as mortar is provided over a minimum of 40% of the unit width. Masonry made with tongue-and-groove units (TG), was built with dry mechanical interlocking between units at the head joints. Thin-layer joint masonry (TM) was built using thin-layer mortar at the bed joints and dry mechanical interlocking between the units at the head joints. Figure 1 shows the three types of unit.

Cross-section design (250x300 mm and shell and web arrangement), percentage of holes (43%) and mean compressive strength (20 N/mm^2) were almost the same in all units. Thin-layer mortar (TM, bed joints 1.3 mm thick) and general-purpose mortar used for specimens with ordinary bed joints (TG and Po, bed joints 12 mm thick), were both pre-mixed products.

More detailed descriptions of test procedures and results are reported elsewhere in [1] and [13].

a) b) c)

Figure 1: Three types of clay units: a) edge-ground unit TM; b) unit with tongue-and-groove TG; c) unit with mortar pocket Po.

NUMERICAL MODELING

To simulate the experimentally observed behavior four types of non-linear finite element models were used. The experimental results were reproduced by means of both macro-modeling and micro-modeling strategies. A plane stress state was assumed in all the adopted models. Eight-node elements with Gauss integration scheme were used in continuum models and for masonry units. In the discrete models, also six-node interface elements with Lobatto integration scheme were adopted.

The macro-modeling strategy implemented two types of constitutive laws: the isotropic total strain rotating crack model [8] and the orthotropic plastic model developed by [15]. The first is implemented with linear softening in tension and parabolic curve in compression, while the second uses Rankine-Hill failure criterion. The micro-modeling strategy implemented continuum elements, adopting both the above mentioned constitutive laws, for the expanded units, and mortar joints collapsed into zero-thickness interface elements. Coulomb friction criterion with parabolic compression cap and tension cut-off describes the interface behavior [15].

The main mechanical parameters were extracted by common tests, used in practice for characterizing materials and simulating actual loading condition in structural masonry walls, these parameters were used without arbitrary corrections. Inelastic parameters, which can be obtained by means of more complex test procedures and are useful for theoretical studies of masonry behavior, were defined on the basis of extensive literature survey [16]. Some simple criteria to obtain orthotropic parameters from isotropic ones and to evaluate mechanical properties of expanded units in micro-models were defined [13]. The first criterion, which influences the masonry anisotropy when perforated units are used, is based on the net area ratio

that characterizes masonry cross sections. The second allows to extrapolate the micro-model parameters and to apply them into the analyses without any correction, by introducing systematically on the numerical models the experimental error related to mortar joints and mortar-unit interfaces irregularity.
To see all the used parameters refer to [13].

VALIDATION OF THE MODELS AND ANALYSES RESULTS

The results showed that, in the case of the uniaxial compression, three out of four models are able to reproduce the experimental behavior, and the orthotropic models seem to be more adequate. The experimental behavior under diagonal compression can be described properly only by means of the interface models, whether they are orthotropic or isotropic. Besides, only the isotropic models adequately describe the shear compression tests. Both these modeling strategies were able to reproduce the observed sequence of failure mechanisms and, in particular, the isotropic micro-models presented crack patterns consistent with the experimental ones (see, for example, Figure 2b and c).
The type of simulated tests was higher influenced on the model accuracy than the type of masonry that composes the specimens. In general, it can be said that any type of test can be well simulated by at least one type of modeling strategy.

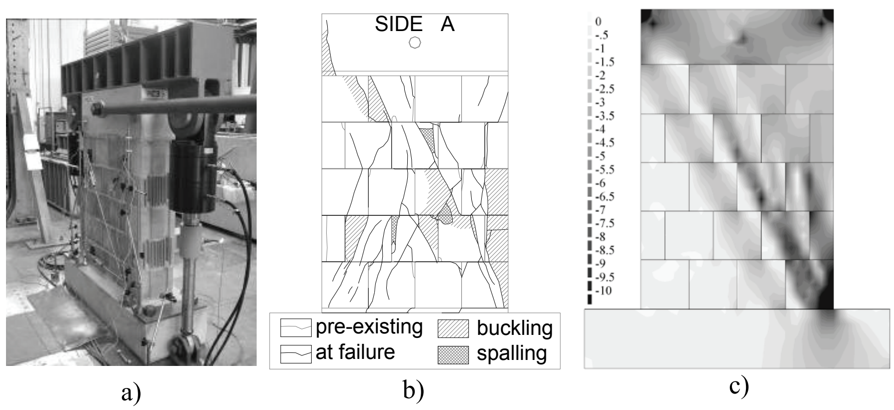

Figure 2: In-plane cyclic shear-compression (TM 27%): test set-up (a); crack pattern (b); principal compressive stresses at ultimate load with interface model TSRCM (c).

PARAMETRIC ANALYSES

The perforated clay units used for experimental tests were designed on purpose for the research, following the principles of 'robustness' given by recent seismic codes (EN 1998-1, 2004; OPCM 3431, 2005; DM 14/01/08, 2008). They had mean compressive strength of about 20 N/mm^2. Compressive strength of perforated clay units used in practice varies in a range from about 20 to 5 N/mm^2. Therefore, to study the influence of unit compressive strength (f_{cu}) on global shear behavior of the three masonry types, the analyses were repeated using units with compressive strength of 20, 15, 10 and 5 N/mm^2. From the results obtained by model calibration, it appeared that simplified micro-modeling strategy, with total strain rotating crack law for units, is the most adequate to reproduce monotonically the cyclic shear behavior of masonry. The parametric analyses thus implemented this model. These criteria, explained more in details in [13], were adopted to define model parameters.

A database of unit properties was created in order to estimate the mechanical characteristics of the theoretical units. This database contains values reported in literature for about one hundredth perforated clay units characterized by similar percentage of holes ([1]; [17]). The fitting process used relations similar to those proposed by Eurocode 2 for concrete [18]. They were adapted to estimate the unit elastic modulus E_u (equation 1) and the unit tensile strength f_{tu} (equation 2) starting from the unit compressive strength f_{cu}. Equations 1, 2 refer to the gross area of units.

$$E_u = 3041 \cdot \sqrt[3]{f_{cu}} \tag{1}$$

$$f_{tu} = 0.087 \cdot f_{cu}^{0.5} \tag{2}$$

For each masonry type and for each value of unit compressive strength, the analyses were repeated applying the same vertical load used in the experimental (and numerical) phases of the research, or a vertical load corresponding to the same ratio between applied load and compressive strength of masonry. In the latter case, it is possible to compare the behavior of each masonry type, varying the unit compressive strength, when stresses inside the walls have comparable intensity.

To evaluate the pre-compression load on the models with different masonry compression strength (with correlation to different unit strength) an adapted version of the Guidi [19] relation (Equation 3) was used, and fitted with the experimental data. The adopted version of the equation used the following values: a=4.52, b=1 and c=2.75. Table 1 shows the parameters used in the parametric analyses that were changed from the models used to reproduce the experimental tests.

$$f_{c,m} = \frac{f_{cu}}{a} \cdot \log(b \cdot f_m + c) \tag{3}$$

Table 1: Parametric analyses. Parameters changed from the used relation.

[N/mm^2]		TM				TG				Po			
f_{cu}		20.42	15.00	10.00	5.00	20.96	15.00	10.00	5.00	20.43	15.00	10.00	5.00
f_{tu}		0.47	0.42	0.33	0.22	0.47	0.42	0.33	0.22	0.58	0.42	0.33	0.22
$f_{c,m}$		6.95	5.16	3.44	1.72	5.67	4.12	2.74	1.37	5.34	4.12	2.74	1.37
σ_0	22%	1.53	1.14	0.76	0.38	1.25	0.91	0.60	0.30	1.17	0.91	0.60	0.30
	27%	1.88	1.39	0.93	0.46	1.53	1.11	0.74	0.37	1.44	1.11	0.74	0.37

All the analyses performed are shown in Figure 3. In Figure 3 the corresponding horizontal load versus drift diagrams are presented. The black line, represents the parametric analyses with the same vertical load applied in the experimental tests, whereas the gray lines indicate the analyses in which the applied vertical load changes in order to keep the ratio with masonry compressive strength constant.

Figure 4 shows a summary, for the three masonry types, of the previously presented parametric analyses results. The dots represent the performance of each numerically analyzed wall in terms of maximum resistance (H_{max}) and drift at ultimate limit state (θ_u). The dash-dots curves correspond to analyses carried out by applying the same vertical load used in experimental tests.

The continuous curves relate to analyses carried out under vertical load corresponding to the same ratio between applied load and masonry compressive strength.

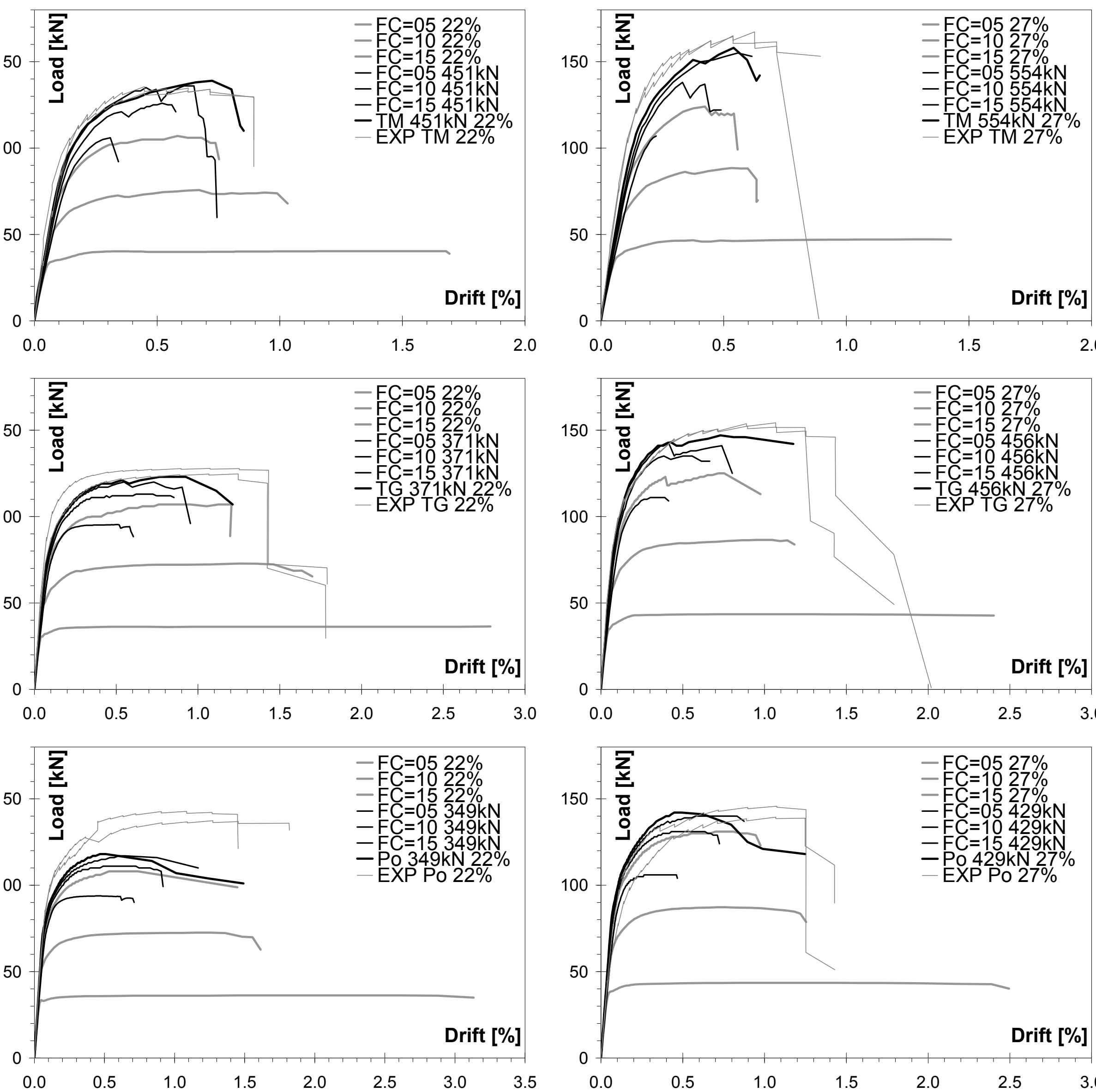

Figure 3: Results of parametric analyses: load-drift curves under the same vertical load (black line) and under the same ratio of vertical load to compressive strength (gray line).

The four dots in each curve correspond to results of analyses carried out on walls made with units of different strength (20, 15, 10 and 5 N/mm^2) and, obviously, decrease in maximum load corresponds to lower unit strength.
When the applied vertical load is the same as in the experimental tests, besides the effect of unit strength, it is possible to observe the behavior of the various masonry types. For TM masonry, maximum horizontal load decreases by -5%, -11%, and -28% for unit strength of 15 N/mm^2, 10 N/mm^2 and 5 N/mm^2 (compared to the case with unit strength of 20 N/mm^2). For TG and Po

masonry, it decreases by about -2%, -7%, and -23% (see Table 2). The differences between masonry types can be related to the higher brittleness of thin-layer joint masonry.

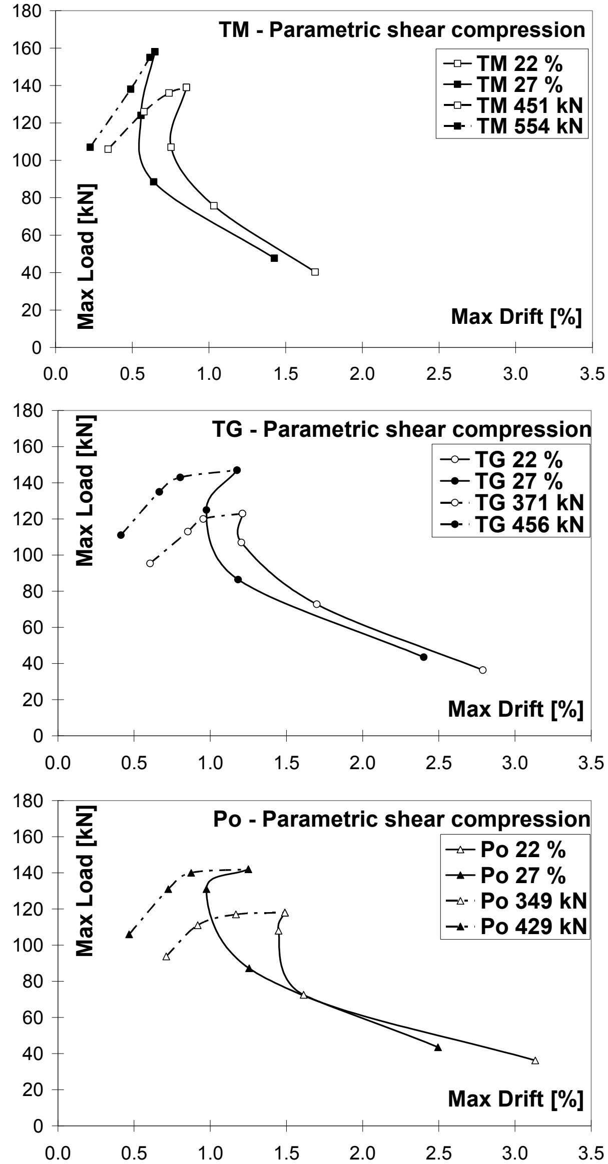

Figure 4: Results of parametric analyses: maximum load-maximum drift diagrams under the same vertical load (dash-dots line) and under the same ratio of vertical load to compressive strength (continuous line).

The most important non-linear maximum load decrease that all the masonry types present at the lowest unit strength show that, for modest unit strength, the wall behavior becomes more brittle independently from the masonry type and this occurs when unit strength is comprised between 10 and 5 N/mm^2. In this regard, it has to be noted that the Italian codes require a minimum characteristic compressive strength of unit of 5 N/mm^2, for the use in seismic areas ([20] and [21]). The European seismic code also recommends a minimum normalized compressive strength of masonry units of 5 N/mm^2 [22]. However, the previous, being a characteristic strength, refers to an average strength which is higher than 5 N/mm^2 (and for sure comprised

between 10 and 5 N/mm^2). Instead, the latter, being a normalized compressive strength, refers to an average compressive strength of unit, which is even lower than 5 N/mm^2 considering the unit dimensions generally found in the construction market and the procedures adopted to obtain this strength from tests [23].
The behavior in terms of drift at ultimate limit state (maximum drift) is different for the various masonry types. As expected, for the lowest unit strength, maximum drift decrease is higher for TM masonry (-63%) than for TG and Po masonry (-57%). However, for higher unit strength, this trend changes, and maximum drift decrease is -28% and -17% for TM masonry and unit strength of 10 and 15 N/mm^2, whereas it is -38% and -26% for TG and Po masonry (as shown in Table 2). Combined with maximum load, this fact is reflected on the dash-dots curves of Figure 4 and depends on the different failure modes of the various masonry types. The curves bend sharply in the case of TG and Po masonry, indicating that failure mode changes for different unit strength. The model results show that rocking is dominating for higher unit strength, while brittle shear failure develops and prevents rocking to occur for lower unit strength. In the case of TM masonry, the dash-dots curves of Figure 4 are almost linear, as shear failure dominates in all the unit strength range and confirms the brittleness of this masonry type.

Table 2: Parametric analyses results.
Experimental and numerical maximum load and maximum drift

TM	f_{cu} [N/mm^2]	22 %		451 kN		27 %		554 kN	
		drift %	load kN	drift %	load kN	drift %	load kN	drift %	load kN
parametric	5.0	1.69	40.3	0.34	106.0	1.43	47.7	0.23	107.0
	10.0	1.03	75.7	0.58	126.0	0.64	88.5	0.49	138.0
	15.0	0.75	107.0	0.74	136.0	0.56	124.0	0.62	155.0
	20.4	0.85	139.0	0.85	139.0	0.65	158.0	0.65	158.0
experim.	20.4	0.89	135.2	0.89	135.2	0.72	166.0	0.72	166.0
TG	f_{cu} [N/mm^2]	22 %		371 kN		27 %		456 kN	
		drift %	load kN	drift %	load kN	drift %	load kN	drift %	load kN
parametric	5.0	2.79	36.4	0.61	95.4	2.40	43.5	0.41	111.0
	10.0	1.70	72.8	0.85	113.0	1.18	86.5	0.67	135.0
	15.0	1.20	107.0	0.95	120.0	0.97	125.0	0.80	143.0
	21.0	1.21	123.0	1.21	123.0	1.18	147.0	1.18	147.0
experim.	21.0	1.43	126.6	1.43	126.6	1.27	153.6	1.27	153.6
Po	f_{cu} [N/mm^2]	22 %		349 kN		27 %		429 kN	
		drift %	load kN	drift %	load kN	drift %	load kN	drift %	load kN
parametric	5.0	3.13	36.2	0.71	93.8	2.49	43.5	0.47	106.0
	10.0	1.61	72.5	0.92	111.0	1.25	87.2	0.72	131.0
	15.0	1.45	108.0	1.17	117.0	0.97	131.0	0.87	140.0
	20.4	1.49	118.0	1.49	118.0	1.25	142.0	1.25	142.0
experim.	20.4	1.64	139.9	1.64	139.9	1.25	142.8	1.25	142.8

When the applied vertical load changes in order to keep constant the ratio with masonry compressive strength (continuous lines), it is possible to observe the effect of unit strength and of the applied vertical load. Through Table 2 is possible to see that for TM specimens, maximum horizontal load decrease is almost linear with unit strength decrease (-22% when f_c=15 N/mm^2, -45% when f_c=10 N/mm^2, -70 when f_c=5 N/mm^2). For TG and Po specimens, maximum horizontal load decrease is less pronounced when unit strength is higher, but it is exactly as in TM masonry for the lowest unit strength (-11% when f_c=15 N/mm^2, -40% when f_c=10 N/mm^2, -70 when f_c=5 N/mm^2). This confirms that the lowest unit strength is critical for all the masonry type. The behavior in terms of maximum displacement is similar for the three masonry types. For unit strength of 15 N/mm^2 displacement is 11% smaller than for unit strength of 20 N/mm^2, due to increased brittleness. For lower unit strength (10 and 5 N/mm^2) the effect of the lower vertical load applied becomes evident and maximum displacement increases of 12% and 110% respectively. Finally, the comparison of the three masonry types in Figure 4 shows that maximum drift, in TM masonry, is always lower than in Po and TG masonry.

CONCLUSIONS

Three types of load-bearing un-reinforced masonry walls, made with perforated clay units and different types of head and bed joints were experimentally tested under in-plane cyclic loads. Four different types of non-linear models, which follow macro and micro-modeling strategies and implement isotropic damage criterion or orthotropic plastic criterion for materials, were calibrated based on tests results. The isotropic models, in particular the isotropic micro-model, adequately described the shear compression tests and allowed to observe the differences in stress distribution and behavior due to the type of masonry bond arrangement.

The parametric analyses carried out showed how the wall performance under combined shear and compression depend on the unit strength and masonry type. In general, maximum horizontal load and drift at ultimate state decrease with the unit strength. For masonry with ordinary bed joint (Po and TG) the decrease of unit strength also corresponds to change in dominant failure mode, from rocking to shear, whereas in the case of more brittle thin-joint masonry, shear is prevailing in all the unit strength range. In all masonry types, critical behavior arises for unit strength between 10 and 5 N/mm^2. The corresponding requirements for the minimum unit strength to be used in seismic area seem to be adequate in the Italian code but unconservative in the European standard. If the ratio of applied vertical load to masonry compressive strength is kept constant, maximum drift can increase for lower unit strength, as vertical load decreases and shear behavior is less prevailing. In any case, displacement capacity of thin-layer joint masonry remains smaller than for other masonry types.

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