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Seismic assessment of brick URM buildings: latest findings and future perspectives

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Abstract

The paper gives an overview of recent developments in the field of seismic assessment of unreinforced masonry (URM) buildings, drawing mainly from researches and coordinated projects recently carried out mostly in Italy. In particular, this work presents the latest findings on the available models and programs for the seismic assessment of masonry buildings, on the definition of rational criteria for the application of different strength expressions and on the evaluation of drift capacity of walls, specifically for the case of brick masonry. In order to investigate such aspects, a systematic comparison with past experimental data derived from in-plane cyclic tests on brick piers and its interpretation has been carried out. A recent benchmark study on different commercial software for seismic assessment of URM buildings has also been briefly presented. The findings of this study may provide a basis for the improvement of the codified approaches and a useful tool for professionals.

Key words: seismic assessment, clay brick masonry, URM buildings, codified procedures, global analysis, strength criteria, wall drift limits

1 Introduction

The large number and variety of Unreinforced Masonry (URM) buildings present in seismic-prone areas around the world requires the need of finding rational approaches for their seismic assessment, which should be well supported and validated by numerical and experimental research. In fact, most of such buildings were constructed before the development of rational engineering design procedures and usually exhibit high seismic vulnerability, as demonstrated by several past seismic events, e.g. the 2020 Zagreb [1] and Petrinja earthquakes in Croatia, just to mention two of the recent strong ones. Within the historical masonry building stock, the clay brick masonry surely represents one of the most common.

Typical brick URM buildings are likely to be composed of several load-bearing masonry walls arranged in orthogonal planes, with relatively flexible floor diaphragms. Observed seismic damage in URM structures often includes out-of-plane failures of walls, driven by excessive deflections of diaphragms and insufficient connections between them.

When the out-of-plane failure is prevented by proper measures, like reinforced concrete ring beams, steel ties at the floor levels and, in general, suitable connections between walls and diaphragms, the in-plane response of the walls may be exploited, providing robustness and stability to collapse.

Indeed, the attention is here focused on the global seismic assessment of the masonry buildings, in particular on the modelling procedures and the structural programs currently available (and their reliability), and on the issues regarding the choice of suitable in-plane strength criteria and displacement capacity levels of brick walls to be adopted in the structural analyses.

In fact, although several studies and correlations between different existing strength formulations on brick walls have been performed in the past and are available in the literature (e.g., [2, 3, 4]), an unanimous consensus is still lacking. This topic becomes even more relevant after the recent release of new structural codes (e.g., the Italian NTC2018 [5] and the draft of EC8 part 3 [6]), which propose, for existing buildings, different alternative formulations for the calculation of the in-plane shear strength of URM piers, making the choice of the more suitable formulation to adopt rather intricate, above all when dealing with brick masonry.

An even more complex issue regards the evaluation of the in-plane displacement capacity of brick walls as a function of difference performance limits, where the current European codes define a unique value of drift at ultimate limit state, without differentiating between different masonry typologies, e.g., regular/irregular masonry, bond pattern.

Therefore, this paper aims to present the latest findings on these aspects, in particular on the available models and programs for non-linear seismic assessment of masonry buildings and their validation, on the definition of rational criteria for the application of the different strength expressions and on the evaluation of drift capacity, specifically for the case of brick masonry walls; these two latter topics have been investigated thorough a systematic comparison with experimental data of in-plane cyclic tests on brick piers. A very recent benchmark study on different commercial software for seismic assessment of URM buildings has also been briefly presented. The findings of this study can provide a basis for the improvement of the codified approaches and a useful tool for professionals.

2 Overview on the available models for seismic assessment of masonry buildings and benchmark study on different commercial software

When the out-of-plane failure is prevented by proper measures and the in-plane strength of walls is exploited, in the seismic assessment of masonry buildings attention should be paid mostly to methods of global analysis. In particular, EC8 and other modern seismic norms (e.g., the Italian NTC2018) consider four main methods of structural analysis: linear static (or simplified modal), linear dynamic (typically multimodal with response spectrum), non-linear static ("pushover") and non-linear dynamic. The methods of analysis that are used in common practice are essentially elastic linear (static or dynamic, usually through equivalent frame and FEM-based software) or non-linear static methods, originally mainly based on "storey mechanism" models [7] but in the last two decades characterized by equivalent frame or macro-element idealizations of the entire 3-D buildings (e.g., [8]). Despite macro-element procedures in the field of dynamic non-linear analyses have been recently developed, the use of such tools still requires high expertise and large computational burden and therefore are not suitable yet for everyday practice.
In the following lines, considerations on non-linear static analyses and related models are reported, along with some results of a recent benchmark study on different commercial software for the seismic analysis of URM buildings.

2.1 Non-linear static analyses (pushover)

The last two decades have been characterized by a significant progress in non-linear methods of analyses of masonry structures, to the extent that now a rather reliable nonlinear pushover analysis of buildings is a real possibility also for practice. The need for non-linear seismic analysis of masonry buildings had been recognized in Italy and Slovenia as early as in the late 1970s, after the 1976 Friuli earthquake, after which an equivalent static, simplified non-linear assessment method was proposed and developed in Slovenia by Tomaževic ([7]), based on the so-called "storey-mechanism" approach. On the other side, recently, refined nonlinear finite element modelling has made significant progress (e.g., [9, 10]), although does not constitute yet a suitable tool for the analysis of whole buildings in everyday engineering practice. For this reason, several methods based on macro-element discretization have been developed, requiring a low to moderate computational burden.

As a development of several basic ideas of the "storey-mechanism" approach, a non-linear method based on an equivalent frame idealization of multi-storey walls was developed and implemented at the University of Pavia and EUCENTRE (called SAM, see [8, 13]), followed by other programs, such as Tremuri [11] and 3D-Macro [12]. In this method, masonry buildings are modelled by a three-dimensional equivalent frame with walls, ring beams and masonry spandrels considered as beam-column elements placed in the centroid of the structural elements (see Figure 1a)). The walls and the horizontal elements are supposed to have an elastic-plastic behaviour with limited deformation expressed in terms of chord rotation θ, as illustrated in Figure 1b) and Figure 1c). The elements have a linear elastic behaviour until one of the possible failure criteria (flexure or shear) is met. This idealization can yield effective results on quite regular structures, also when compared with more refined nonlinear FEM analyses or experimental results [13].
Several software packages for nonlinear pushover analyses of masonry buildings have also become available to the public since mid-2000s above all in Italy (e.g., [14, 15, 16], among the others). These tools have been made available only for a limited time so far, therefore, there is a justified concern among professionals on their reliability, and on the type of results that such tools would produce in the hands of an average professional. However, it is the author's opinion that the main problem lies not much in non-linear modelling procedures, but in how the engineer defines its model, as it would be also for a linear elastic analysis, understanding the meaning of the input parameters and of the default values suggested by the software and its input interface.

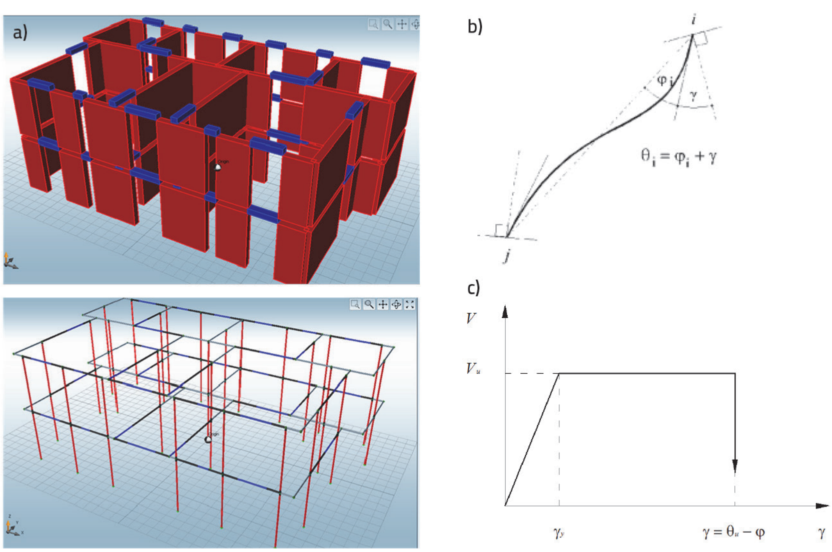

Figure 1. Non-linear method SAM: a) equivalent frame idealization; b) definition of chord rotation; c) bilinear behaviour of the walls for shear failures

2.2 Benchmark study on different software for pushover analyses

If the model is consistently defined using different reliable and validated softwares, the differences in the results become minimal, in particular in terms of initial stiffness and global building strength prediction. In this context, a wide research program (named "*URM nonlinear modelling* - Benchmark project") has been carried out by several Italian Universities involved in the Italian Network of Seismic Laboratories (ReLUIS) projects, funded by the Italian Department of Civil Protection ([17]). The final objectives of this research, started in 2014 and still in progress, are to support the professionals in acquiring awareness in the use of commercial software packages available for the seismic assessment of masonry buildings and to provide them analytical tools for the critical evaluation of the obtained results. In particular, the research activity has been organized by defining various benchmark structures of increasing complexity (single panel, trilith, 2D wall, two floor single-cell building, real 3D building), each one accompanied by data sheets containing all the input data necessary to reproduce the structures by third parties, too. The analyses have been performed through different software, based both on equivalent frame, discrete and finite element modelling approaches, assuming, when possible, same assumptions in the definition of the numerical models, to reduce the dispersion of the results due to the intrinsic characteristics of the programs and to the arbitrariness of analysts.
As an example of the obtained results, the comparison of the predicted numerical responses of some of the case studies are here reported. In this paper, two benchmark cases are considered (respectively named as "BS4" and "BS6"), inspired by a single-unit two-story brick unreinforced masonry building tested at the University of Pavia in mid 1990s ([18]) and by an existing multi-storey masonry buildings, struck by the Central Italy earthquake sequence in 2016-17 (the Pizzoli Town Hall (AQ), [19]).
For each structure, two configurations have been considered, respectively called "A" and "C", the former characterized by weak spandrels (i.e. without tensile resistant elements coupled), the latter by spandrels coupled to RC beams. In Figure 2, 3D views of the two case studies are sketched, referring to configuration C.
In Figure 3 and Figure 4 the global capacity curves obtained by the different adopted commercial software packages (*"SWi"*) based on equivalent frame models are reported, referring in particular to the analyses performed assuming a "uniform" distribution of the lateral forces acting in the positive X-direction (see Figure 2). A quite good agreement in the predicted numerical response of each structure can be observed, in terms of global initial stiffness, maximum shear strength and ultimate deformation capacity, despite the difference in the definition of some parameters like the deformation capacity of masonry elements (in some software expressed in terms of chord rotation in others in terms of drift). These effective results were also found on all the considered different benchmark structures, proving that, if engineers make use of validated software with awareness, understanding the meaning of the input parameters, the results of the pushover analyses are sound and reliable.

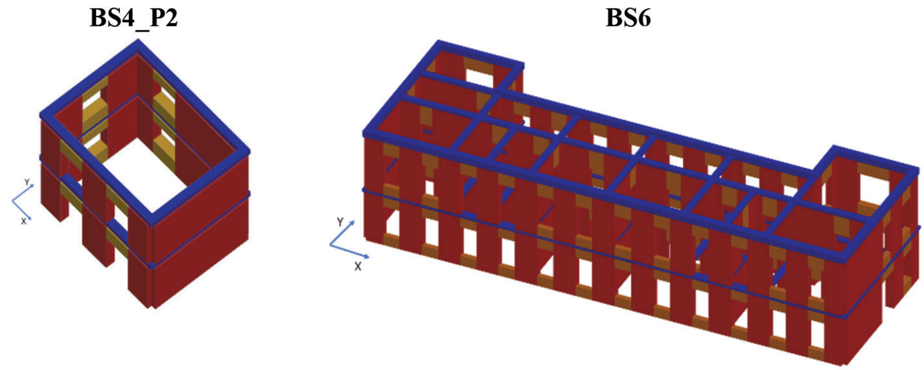

Figure 2. 3D view of the considered case studies

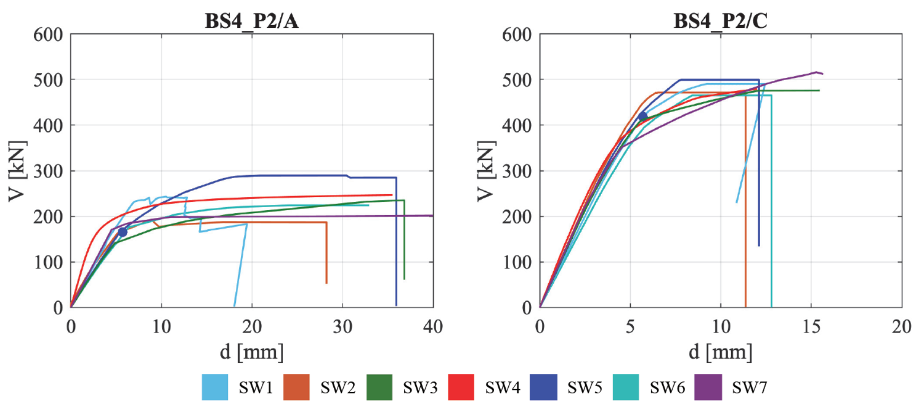

Figure 3. BS4: Comparison of the numerical results in terms of global capacity curves

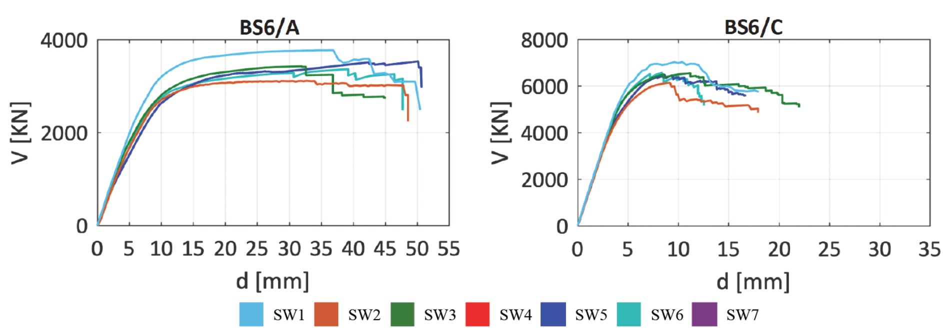

Figure 4. BS6: Comparison of the numerical results in terms of global capacity curves

3 In-plane cyclic tests on brick masonry walls

It is clear that, in the global assessment approach, besides the reliability of the calculation tools, two of the parameters that play a fundamental role are the strength and deformation properties of the members. However, despite the explicit possibility given by EC8 to use non-linear static procedures, little guidance is, in general, given for existing buildings in the current version of EC8 part 3 [20] and very few indications are reported on both the strength criteria and the drift limits of brick masonry walls. Therefore, to fill this gap, an investigation on the more suitable strength expressions and an evaluation of the drift capacity has been carried out through a systematic comparison with experimental data of in-plane cyclic tests on masonry piers. The experimental results on brick walls have been extracted from a dataset recently published by Morandi et al. [21] and by other tests ([22][23][24]), leading to a total of 51 specimens. An example of in-plane cyclic test on brick masonry pier is reported in Figure 5.

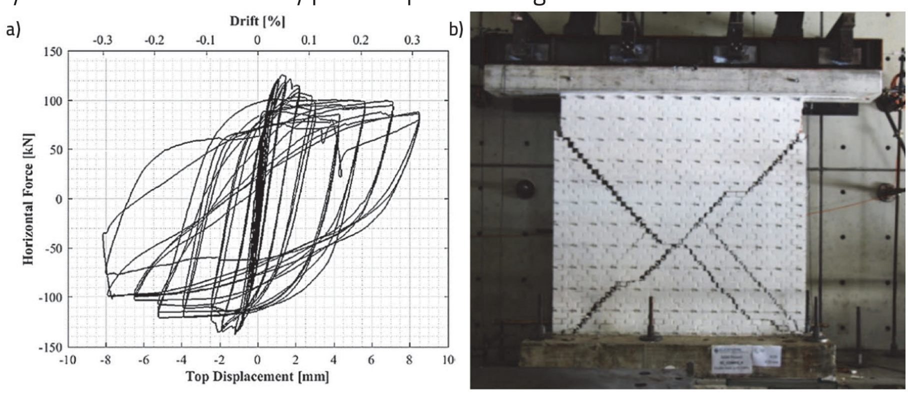

Figure 5. Example of an in-plane cyclic test of brick masonry wall: a) force-displacement hysteretic curve; b) shear failure mechanism ([25])

The tests were carried out on specimens having different heights and include both tests on real scale panels (between 2.5 and 3 m), and on reduced dimensions (around 1.5 m), as summarized in Figure 6(a). Twenty-eight specimens are realized with a "running/stretcher bond" brick pattern, and twenty-three with an "English bond". The tests cover a wide range of applied vertical load values, as reported in Figure 6(b) in terms of normalized stress (σ_v/f_d), such to include the values of mean stress acting on the walls in most of the real buildings with common height and categories of use. Considering the boundary conditions of the specimens (i.e., the static scheme assumed during the cyclic tests), there are 28 walls tested with double-fixed condition (DF), i.e., without rotation of the top beam, and 23 walls with cantilever configuration (C), i.e., with the top beam free to rotate. This information is essential for the choice of the shear span h_0 to be used in some of the proposed formulations, equal to half of the height of the wall in the case of double-fixed and equal to the height of the panel in the case of cantilever. Regarding the characterisation of the material, the mechanical properties of units, mortar and masonry were measured according to the relevant Standards.

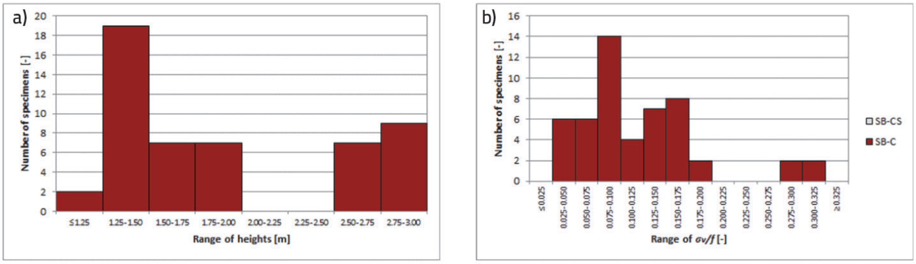

Figure 6. a) Number of specimens at different wall height ranges (xinf < x ≤ xsup); b) number of specimens at different intervals of $\sigma v/f$ (xinf < x ≤ xsup)

4 Rational application of different strength criteria on brick walls

The lateral strength of the 51 brick masonry specimens here considered has been evaluated, for the estimation of the shear resistance, according to the approaches included in EC6 [26] and EC8 [20] ("Coulomb approach"), by Turnšek and Čačovič ([27] and [28]), Mann and Müller [29] and Magenes and Calvi [2]. The lateral resistance determined by the Turnšek and Čačovič and Mann and Müller criteria has been estimated multiplying the unit shear strength by the area of the wall (thickness t x length l) divided by the parameter b, equal to the in-plane slenderness h/l of the panel and limited, as proposed by Benedetti and Tomaževič (1984), between 1.0 and 1.5. For the prediction of the flexural/rocking resistance, the flexural criterion in NTC2018 [5] has been adopted. Such expressions have been properly applied on the cracked and whole wall sections according to the different approaches and boundary conditions. More in-deep information on the adopted strength approaches are reported in [30].

The actual geometrical parameters and vertical load values of each tested specimen have been considered in the calculations, adopting the mechanical properties of units, mortar and masonry obtained by the performed tests of characterization, without any estimation of the parameter to avoid other uncertainties in addition to those due to the intrinsic experimental variability. The values of the compression and shear masonry strength and of the unit strength have not been reduced by any partial material factor (g_M = 1).

The so-calculated values of resistance were then compared with the experimental results obtained from the in-plane cyclic tests, in order to evaluate the suitability and applicability of the different shear strength formulations in the case of brick walls. The comparisons are illustrated in Figure 7(a) to (e) in terms of ratio between calculated (V_t) and experimental ($V_{max,exp}$) shear strength for the different expressions; a ratio equal to 100 % provides a perfect coherence between the estimated and the experimental value, while lower or higher ratios indicate respectively under- and over-estimated predictions. The coloured marks show the failure modes attained in the tests (flexure, hybrid flexure-shear, hybrid flexure-sliding, hybrid shear-sliding, shear and sliding), whereas at the bottom of each graph, "F" or "S" indicate the expected analytical failure, respectively for flexure or for shear, applying the different strength expressions.

Focusing on the criteria for the evaluation of the resistance associated to shear mechanisms, a summary of the analytical vs experimental comparison applying the different approaches is reported in Figure 7(f). For each formulation, the mean value of the ratios between calculated and experimental shear strength, considering only the specimens characterized by experimental shear failures, is indicated with a marker, while the two values obtained by adding/subtracting to this value the mean of the absolute values of the positive or negative deviations from 100 % are represented with two dashes.

First of all, the expression for the calculation of the lateral strength corresponding to a flexural failure, besides being reliable in the case of "pure" flexural/rocking mechanisms, can safely also be extended with a good approximation to panels that exhibit hybrid mechanisms/failures involving flexure.

The application of the criteria by EC6/EC8, Turnšek-Čačovič and Mann-Müller for the shear strength leads to non-conservative estimate as respect to test results; the first two provide a mean calculated/experimental ratio of 108 % and the Mann-Müller expression a ratio of 122 %, with an upper bound error of further 28 %.

On the other hand, the method proposed by Magenes and Calvi seems to be safe-sided, providing a mean calculated/experimental ratio of 88 %, which also allows a conservative prediction when the related deformation capacities are considered. Moreover, this approach is certainly more suitable conceptually than all the other criteria, because it applies more rationally the expressions involving the mortar joints failures on both cracked and whole sections and it considers the actual global parameters of the masonry for the "Coulomb"-sliding approach on cracked sections (and) consistently to the Mann-Müller formulation, which implies the assumption of weak head-joints.

Based on the results of the comparison with the experimental tests, a refinement and optimization of the resistance criteria to be used for the assessment of brick walls in masonry buildings is presented in Table 1, which is also somehow in line with the procedure reported in the last draft of the new EC8-3 [30]. Regarding the shear mechanisms, for irregular brick masonry, the only formulation by Turnšek and Čačovič should be contemplated. For regular masonry instead, the minimum strength value between the expressions that involve joint and brick failures is adopted, according to the approach proposed by Magenes and Calvi, which derives by a rational application of the "Coulomb" and Mann-Müller approaches. The strength criterion for flexural mode is identical for both the irregular and regular brick masonry. The strength of a wall undergoing sliding, under seismic excitation, along a horizontal joint is sometimes expressed as mN, where *m* represents the sliding coefficient of friction of the masonry joint, and cohesion is neglected invoking the fact the joint is already cracked in tension due to flexure. In the case of regular masonry, if *m* is put equal to the residual friction of a sliding bed-joint, mN tends to underestimate rather significantly the load, which corresponds to the onset of sliding, since the sliding resistance of a joint cracked in tension is higher than the residual sliding strength of a bed-joint failing in shear. The expressions in Table 1 for regular masonry and shear involving joint failure could be considered more appropriate ([2]).

For each wall, the safety verifications need to be performed at the top, at the base and at the centre of the panel according to the applied criteria. The minimum lateral resistance between flexural and shear modes defines the lateral resistance and the failure mechanism of the wall. If, in a structural analysis of a building, the verifications are carried out only at the end sections of the walls, the more critical section of the two should be considered, anyhow taking also in account the expression in the uncracked section.

More information on this study on the application of the different strength criteria for brick masonry walls is included in [30].

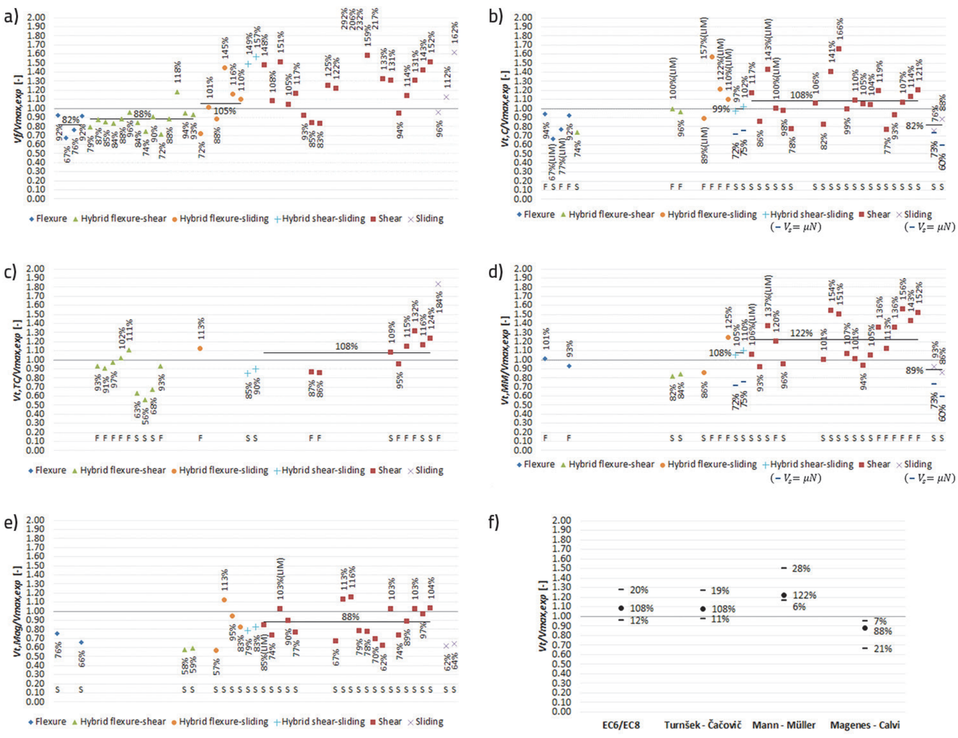

Figure 7. Calculated/experimental ratios of lateral resistance between the different strength criteria: (a) flexural criterion. Shear criteria: (b) EC6/EC8, (c) Turnšek and Čačovič, (d) Mann and Müller, (e) Magenes and Calvi. (f) Averaged calculated/experimental ratios for the different shear strength criteria

Table 1. Proposal for the lateral resistance criteria for brick masonry walls for assessment procedures

<table>
<tr><th>Brick masonry</th><th>Flexural</th><th colspan="2">Shear</th></tr>
<tr><td rowspan="2">Irregular masonry</td><td rowspan="2">$V_f = \frac{l^2 t \sigma_0}{2h_0}\left(1-\frac{\sigma_0}{0.85 f_d}\right)$
(relevant to cracked sections)</td><td colspan="2">Shear involving masonry diagonal cracking</td></tr>
<tr><td colspan="2">$V_t = lt\frac{f_{td}}{b}\sqrt{1+\frac{\sigma_0}{f_{td}}}$
(relevant to whole sections)</td></tr>
<tr><td rowspan="2">Regular masonry</td><td rowspan="2">$V_f = \frac{l^2 t \sigma_0}{2h_0}\left(1-\frac{\sigma_0}{0.85 f_d}\right)$
(relevant to cracked sections)</td><td>Shear involving joint failure</td><td>Shear involving brick failure</td></tr>
<tr><td>$V_d = lt\tau_{min(\tau_{cs};\tau_{ws})}$
$\tau_{cs} = \frac{1.5\tilde{f}_{v0d} + \tilde{\mu}\sigma_0}{1+3\frac{\tilde{f}_{v0d}\ \alpha_V}{\sigma_0}}$
(relevant to cracked sections)
$\tau_{ws} = \frac{\tilde{f}_{v0d} + \tilde{\mu}\sigma_0}{1+\alpha_V}$
(relevant to whole sections)</td><td>$V_{d,b} = lt\frac{f_{btd}}{2.3(1+\alpha_V)}\sqrt{1+\frac{\sigma_0}{f_{btd}}}$
(relevant to whole sections)</td></tr>
<tr><td colspan="4">Notes l: wall length; t: wall thickness; h_0: span ratio; α_V: shear ratio (h_0/l); b= 1.0≤h/l≤1.5; σ_0: axial load stress; f_d: masonry compression strength; f_{tb}: tensile strength from diagonal compression; $\tilde{f}_{v0d}$: equivalent initial shear strength; $\tilde{\mu}$: equivalent friction coefficient; f_{tb}: brick tensile strength.</td></tr>
</table>

5 Deformation capacity of clay brick walls

Regarding the displacement capacity of clay brick walls, the values of drift at peak force (θ_{Vmax}), at 20 % of strength drop after the peak force ($\theta_{20\ \%drop}$) and at the maximum attained drift (θ_{max}) have been derived from the cyclic tests, since they may be associated respectively to the Damage Limitation (DLLS), Life Preservation/Severe Damage (SDLS) and Near Collapse Limit States (NCLS), as suggested by some authors (see, e.g., [31]) and inferable from the damage pattern of walls observed during the tests. As an example, Figure 8 reports an experimental envelope curve and the sequence of damage at corresponding points of the envelope for a clay brick wall failing in shear.

The drift values θ_{Vmax}, $\theta_{20\ \%drop}$ and θ_{max} are plotted in Figure 9 for all the considered clay brick wall specimens, with the indication of the experimental failure mode. Figure 10 reports the cumulative distribution functions (log-normal distributions) associated to the three drift levels applied to all specimens ("ALL") and also to shear ("S") and hybrid ("H") mechanisms separately (the specimens with flexural and sliding modes were too limited to make any statistical processing). Table 2 finally reports the main statistical parameters, where e^m (median) and β are the parameters of the lognormal curves. The fragility curves in Figure 10 on drift capacity derived by the experimental results can be used in probabilistic seismic assessment.

More information on the evaluation of the in-plane displacement capacity of masonry walls is reported in [21].

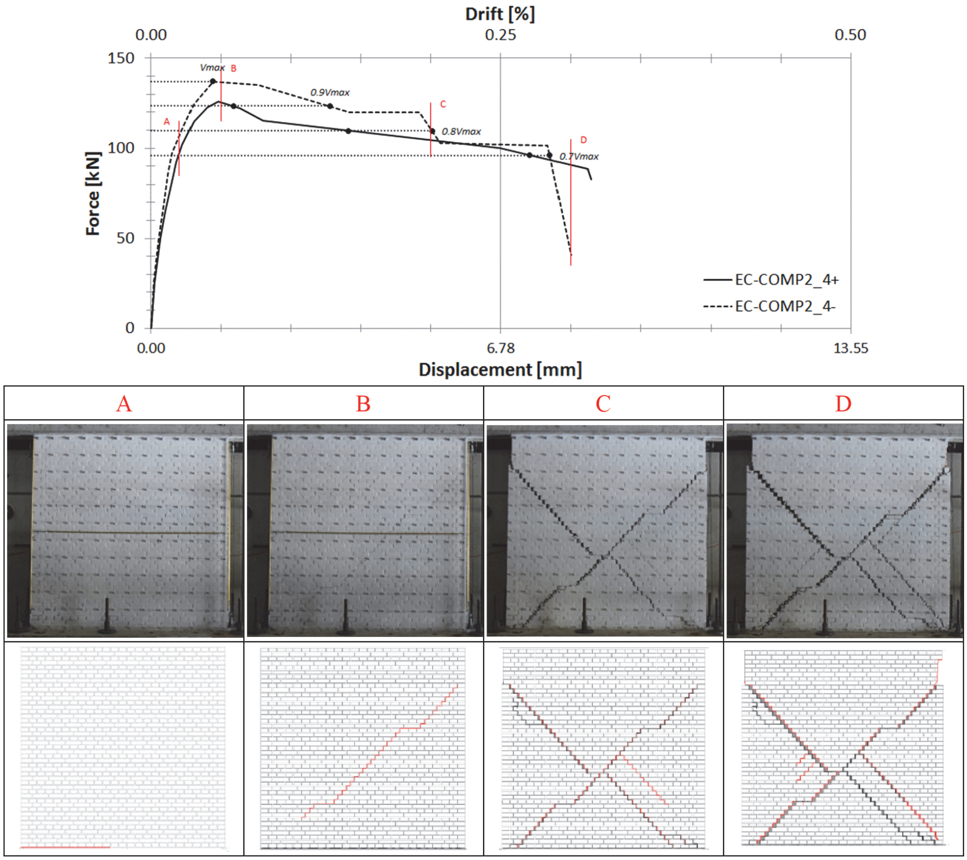

Figure 8. Example of damage pattern sequence ([25]) at corresponding points on the experimental envelope

The values of drift at peak force θ_{Vmax} have ranged in a wide interval between approximately 0.10 % and more than 1.00 %, with a mean value of 0.46 % (median=0.36 %), with lower values for shear modes (mean=0.28 %, median=0.24 %)).

The values of drift $\theta_{20\ \%drop}$ differ significantly as a function of the different experimental failure modes; in particular, for pure shear failures, drifts between about 0.20 % and 0.80 % have been obtained, with a mean value of 0.46 % and a median of 0.43 %. Conversely, walls characterized by flexural/rocking mechanisms and pure sliding have provided higher values of drift $\theta_{20\ \%drop}$, but only few tests are available for these mechanisms. Specimens with hybrid modes have instead obtained intermediate values, with a mean drift of 1.07 % and a median of 0.95 %.

The same trend of $\theta_{20\ \%drop}$ was found for the values of the maximum drift capacity achieved at the end of the test, θ_{max}. In the case of pure shear mechanisms, the mean and median value of θ_{max} were equal to 0.50 % and 0.47 %, respectively. Larger values of drifts were found for flexural and sliding, being the drift capacity for specimens with hybrid modes lying between shear and non-shear mechanisms (mean and median of θ_{max} equal to 0.99 % and 0.91 % respectively).

The scatter of the drift values $\theta_{20\,\%drop}$ and θ_{max} for specimens failing in shear is found to be more limited than for hybrid mechanisms and for all the modes considered together. It is finally important to note that some results of drift, in particular in terms of $\theta_{20\,\%drop}$, are not available for lack of information in the original publications, as shown in Figure 9; this clearly affects the results as reported in Figure 10 and Table 2, in particular for the case of hybrid failures.

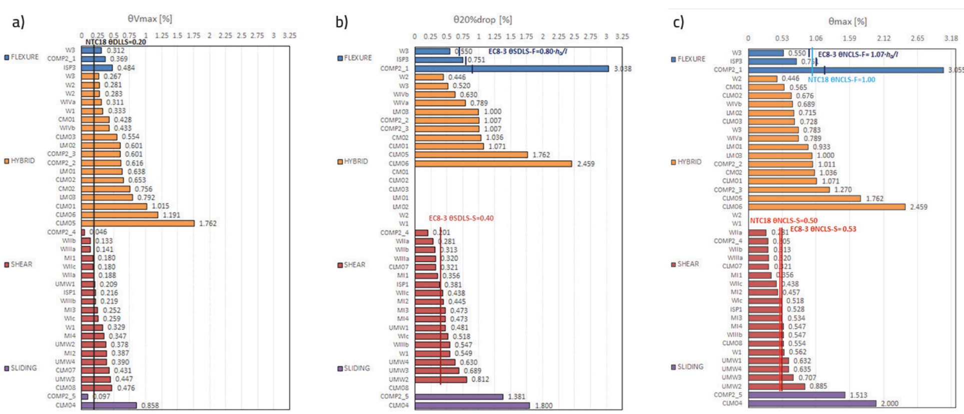

Figure 9. Experimental drift values for different failure mechanisms: at peak force θ_{Vmax} (a), at 20 %-drop of V_{max} $\theta_{20\,\%drop}$ (b), and at the maximum θ_{max} (c)

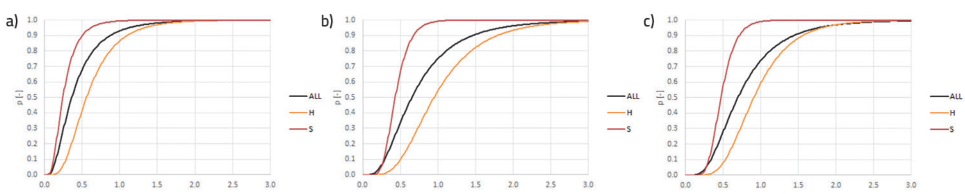

Figure 10. Fragility curves of the experimental drift values for different failure mechanisms: at peak force θ_{Vmax} (a), at 20 %-drop of V_{max} $\theta_{20\,\%drop}$ (b), and at the maximum θ_{max} (c)

Anyhow, it is important to recall that the values of drifts at ultimate conditions (SDLS and NCLS) clearly depend on the axial load ratio on the walls and on the shear ratio (h_0/h or h_0/l) and expressions able to match the experimental test results as a function of such parameters are surely needed. Moreover, the question whether associating the drift limits to the failure mechanisms (i.e., for shear or for flexure) or not is still unsolved and needs to be further investigated.

Finally, it is also useful to point out that the drift capacity determined in Figure 9 and in Table 2, in particular the ones failing in shear, should be considered conservative estimates of the actual limits. This is due to many reasons, for example to the very large number of cycles carried out in the tests, much higher than an actual ground motion in real cases [32], or to the actual occurrence of hybrid mechanisms instead of pure shear and of boundary conditions different by double-fixed that surely allows enhancing the effective displacement capacity as respect to the one obtained by the tests failing in pure shear.

Table 2. Main statistical parameters of drifts associated with different failure mechanisms for clay brick walls

	ALL			FLEXURAL			SLIDING			SHEAR			HYBRID		
	θ_{Vmax} [%]	$\theta_{20\%drop}$ [%]	θ_{max} [%]	θ_{Vmax} [%]	$\theta_{20\%drop}$ [%]	θ_{max} [%]	θ_{Vmax} [%]	$\theta_{20\%drop}$ [%]	θ_{max} [%]	θ_{Vmax} [%]	$\theta_{20\%drop}$ [%]	θ_{max} [%]	θ_{Vmax} [%]	$\theta_{20\%drop}$ [%]	θ_{max} [%]
median (e^{μ})	0.36	0.65	0.70	-	-	-	-	-	-	0.24	0.43	0.47	0.56	0.95	0.91
β	0.69	0.63	0.57	-	-	-	-	-	-	0.57	0.35	0.32	0.53	0.50	0.42
max	1.76	3.04	3.06	0.48	3.04	3.06	0.86	1.80	2.0	0.48	0.81	0.89	1.76	2.46	1.27
min	0.05	0.20	0.55	0.31	0.55	0.55	0.10	1.38	1.51	0.05	0.20	0.28	0.27	0.45	0.45
mean*	0.46	0.80	0.82	0.39	1.45	1.45	0.48	1.59	1.76	0.28	0.46	0.50	0.64	1.07	0.99
* Arithmetic mean															

5.1 Implications in codified assessment procedures

In codified procedures for seismic assessment, non-probabilistic single values of drift limits are defined; therefore, in order to suggest drift values for codes, the reference to mean/median values from test results appears to be reasonable and sufficiently conservative. Besides the considerations related to the influence of the axial load and shear ratio, some findings can be drawn comparing the drift results discussed above with the drift limits imposed in current codes (i.e., the Italian Instruction of NTC2018 [33] and EC8-3) for existing masonry piers, which are summarized in Table 3 in relation with the different failure modes (shear and flexure) and limit states (Damage Limitation "DL", Severe Damage "SD" and Near Collapse "NC" Limit States). Such limits are also indicated in Figure 9.

Table 3. Drift limits on brick URM piers in the Italian norms and EC8-3

	Damage Limitation (DLLS) [%]	Severe Damage (SDLS) [%]		Near Collapse (NCLS) [%]	
		Flexure	Shear	Flexure	Shear
NTC2018-Instructions NTC2018	0.20	-	-	1.00	0.50
EC8-part 3	-	$0.80 \cdot h_0/l$	0.40	$1.07 \cdot h_0/l$	0.53

If the values of drift at peak force θ_{Vmax} can be assumed as a reference value for the Damage Limitation Limit State, a drift threshold of 0.20 % at DLLS, currently included in NTC 2018, appears to be safe-sided for brick walls also for piers failing in pure shear, as observable in Figure 9. However, no explicit limitation at DLLS for structural masonry buildings is present in the current version of EC8 and surely needs to be included in the new version of the code.
The values of drift $\theta_{20\,\%drop}$ can be related to life preservation/severe damage. As evidenced by Figure 9, the drift limits at SDLS proposed in EC8-3 (0.40 % for shear failures and $0.8h_0/l$ % for flexural modes) seem to be quite adequate for walls failing in shear but overestimate the displacement capacity in the case of flexural mechanism, unless the case of pure rocking modes. The application of the shear limit on specimens with hybrid modes would provide safe-sided results while the application of the flexural one would not; this demonstrates once again the importance of a proper estimation of the failure modes based on strength criteria, to avoid an overestimation of the shear strength which would lead to the use of an unsafe deformation limit.
The drift limits currently recommended in the EC8 and in the current Italian norms for NCLS provide an unsafe value for flexural modes (safe-sided only for pure rocking), and a slightly unconservative threshold for shear failures, as shown in Figure 9 with reference to θ_{max}. Also in this case, the codified shear limits applied on specimens failing with hybrid modes would be sufficiently conservative once the failure mechanism is adequately assessed. It is however important to remark that, on the comparison in terms of maximum displacements, the values of θ_{max} may be affected by a degree of subjectivity and that many of the past tests may have been stopped before the attainment of actual near collapse conditions, underestimating the maximum attainable drift [31].

6 Conclusions

When the out-of-plane failure is prevented by proper measures and the in-plane strength of walls can be exploited, the seismic assessment of masonry buildings can be performed with methods of global analysis, for example resorting to static non-linear analyses on equivalent-frame/macro-element models. Several software packages have been made available to professionals in the last 10-15 years and some of these have been used for a wide study of validation in a benchmark project, providing good results in terms of reliability in the estimation of the global response of the buildings.
However, despite the explicit possibility given by EC8 to use non-linear static procedures, little guidance is given for existing buildings in the current version of EC8 part 3, with the only reference to use strength expressions for new buildings and with gaunt information on deformation capacity of the masonry elements.
In order to provide a rational approach to define the strength criteria and a proper evaluation of the in-plane deformation limits for brick masonry walls, a possible method was thought referring to the results of in-plane cyclic tests on masonry piers conducted in the past.

From a systematic comparison with the experimental data, an effective criterion for the application of the different strength expressions for brick masonry walls has been proposed as a function of the masonry typology, i.e., regular and irregular brick masonry, and expected failure modes, i.e., shear, flexural and sliding.
The displacement capacity derived at different force levels on the experimental envelopes of the in-plane tests has also permitted to provide first results of drift limits for brick walls. Drift limits associated to experimental drift at peak force (θ_{Vmax}), drift at 20 % of strength drop after the peak force ($\theta_{20\ \%drop}$) and at the maximum drift attained ($\theta_{max,f}$ and θ_{max}) have been analysed, assuming respectively as reference values for Damage Limitation (DLLS), Severe Damage (SDLS) and Near Collapse (NCLS) Limit States. With reference to mean values, drifts equal to 0.28, 0.46 and 0.50 % respectively for θ_{Vmax}, $\theta_{20\ \%drop}$ and θ_{max} have been found for experimental shear failures, whereas larger values have been obtained for flexural, sliding and hybrid modes. Moreover, fragility curves on drift capacity derived by the experimental results have been derived for their possible use in probabilistic seismic assessment.
If, on the one hand, the issue about the proper strength criteria to be used in the seismic assessment of masonry buildings made up by brick masonry walls is somehow now clarified and defined, on the other hand many aspects regarding the deformation limits of brick walls are still in discussion. First, the definition of the limit states based on the experimental results needs further investigation, although the presented approach seems rational and comforted by the attained damage levels. Then, if a unique value for codified procedure should be set, it is crucial to understand whether the use of the mean or median value of the tested specimens is appropriate or a lower percentile is necessary. In addition, the idea of defining a relationship of the deformation limits as a function of the axial load ratio and of the boundary conditions/shear ratio needs to be explored, in order to avoid selecting too conservative values for assessment procedures. It would be finally also important to comprehend how and if the bond patterns used for the construction of the brick walls (for example, "stretcher or English bond") may influence and eventually reduce the in-plane deformation capacity.
Although the aforementioned further developments are surely needed, the results of this research has provided useful information for the improvement of the codified approaches, in particular EC8-3, and for professionals when dealing with the seismic assessment of brick masonry buildings in terms of global in-plane response.

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