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Deformation capacity of unreinforced masonry walls subjected to in-plane loading: a state-of-the-art review

Amir Hosein Salmanpour[1*], Nebojsa Mojsilovic[1] and Joseph Schwartz[2]

Abstract

A research project on the deformation capacity of unreinforced masonry structures is underway at the Institute of Structural Engineering of ETH Zurich. The development of the basic building blocks for the displacement-based design of unreinforced masonry structures is the objective of the present research project, which should be seen as a first step in an initiative to investigate the limits of the deformation capacity of unreinforced masonry walls. This paper presents a summary review of previous experimental and analytical studies on the deformation capacity of unreinforced masonry walls subjected to in-plane loading. This review is the first phase of the aforementioned research program. A summary of 71 shear tests on unreinforced masonry walls is presented in the form of a database, along with the statistical analysis and discussion of the tests results. Furthermore, three different computational approaches for structural masonry, i.e. micro-modelling, macro-modelling and macro-element discretization, are discussed, and a review of macro-elements for the in-plane response of unreinforced masonry walls is presented. The reviewed models are discussed and a set of conclusions is given. Special attention is devoted to the deformation capacity parameter throughout the paper. Finally, the paper shows the limitations of our current state of knowledge of the deformation capacity of structural masonry.

Keywords: Deformation capacity; Experimental research; Shear test; Analytical research; In-plane response; Macro-element; Structural masonry; URM

Introduction

Masonry structures and materials represent one of the oldest building concepts available. Masonry construction is a traditional, widely used, extremely flexible and economical construction method with considerable potential for future developments. However, possibly due to the substantial empirical knowledge collected over several centuries of utilization of masonry as a structural material, the need for establishing a more modern basis for the design of masonry structures has not been appreciated in the same manner as for concrete structures. As a result, conventional masonry design practice is overly conservative, particularly in regard to the assessment of seismic resistance. Hence, the potential of masonry has not yet been fully exploited and there is a clear need for better utilization. For example, while current codes of practice severely limit the use of unreinforced masonry (URM) in construction, mainly because of the requirement of over-conservative values for the force-reduction factor (q-factor in Europe or R-factor in the US), recent studies show that the performance of structurally-designed low-rise URM buildings should be considered adequate for the category of ordinary buildings even in regions with appreciable seismic hazard. Furthermore, these studies also show that unreinforced masonry is still a very competitive choice for two- or three-story residential buildings (Magenes et. al 2009 and Lourenco et al. 2009).

Based on the positive experience gained in recent years in developing the basis for the displacement-based seismic design of concrete structures, it appears that the most feasible approach to enhance the rationality for the design of masonry structures is to apply the same fundamentals

* Correspondence: salmanpour@ibk.baug.ethz.ch
[1]Institute of Structural Engineering, ETH Zurich, CH-8093, Zurich, Switzerland
Full list of author information is available at the end of the article

in the analysis of masonry structures. A more consistent representation of the material resistance as well as of the (seismic) loads leads to more economical (or at least more reliable) designs in general, and especially for masonry structures where the safety margin, i.e. safety factor at present, is mostly based on experience rather than on quantified engineering modelling. Obviously, the first step towards the development of such an approach for masonry structures is to investigate the limits of the deformation capacity of masonry structures.

Given the above, a research project on the deformation capacity of unreinforced masonry structures has been started at the Institute of Structural Engineering of ETH Zurich. The objective of the present research project is the development of the basic building blocks for the displacement-based design of unreinforced masonry structures, which should be seen as the first step in an initiative to investigate the limits of the deformation capacity of unreinforced masonry walls. The research project includes a thorough survey and assessment of existing experimental and theoretical research in the area of the deformation capacity of structural masonry, an intensive testing campaign as well as developing and introducing new sophisticated mechanical models for structural masonry. This paper presents a summary review of previous experimental and theoretical studies on the deformation capacity of unreinforced masonry walls subjected to in-plane loading. This review is the first phase of the above-mentioned research program. It is worth noting here that even the conventional force-based seismic design approach is strongly connected to the deformation capacity parameter through the force-reduction factor which is used in the estimation of design force of structures in the force-based seismic design approach. Furthermore, the deformation capacity plays a crucial role in the seismic assessment and retrofitting of existing structures, which has become one of the main research topics in structural engineering.

Deformation capacity of structural masonry

In masonry structures subjected to seismic action, if local brittle failure modes, e.g., out-of-plane failure, are prevented by providing proper connections between intersecting walls and also between walls and diaphragms, a rather ductile global behaviour governed by the in-plane response of walls can develop. Hence, the investigation of the deformation capacity of masonry structures should start by studying the in-plane behaviour of masonry walls and their constitutive elements, i.e. piers and spandrels (in the case of walls with openings). Even though spandrels have a significant effect on the in-plane response of URM masonry walls, it seems, however, that the deformation capacity of masonry walls is mainly identified with the deformation capacity of piers. This is because experimental studies on spandrel and pier elements have shown that for most wall geometries the spandrel deformation capacity is larger than the pier deformation capacity; see, e.g., (Beyer and Dazio 2012).

The in-plane behaviour of masonry walls subjected to horizontal and vertical forces has been investigated in various test programs. As indicated by experiments, the in-plane response of masonry walls depends mainly on their failure mechanism. In the case of low vertical load and/or poor quality mortar, seismic loads cause shearing of the wall in two parts and sliding of the upper part on the other part. The mechanism is called sliding shear failure. The in-plane response of masonry walls failing in sliding shear mode is very stable and close to an elastic, perfectly plastic response with high energy dissipation and displacement capacity. In the case of sliding shear failure mode, the displacement capacity is very large. However, for practical applications it should be limited since the shear walls normally interact with other building elements.

The diagonal shear mode (simple shear mode) occurs where the principal tensile stress exceeds the in-plane tensile strength of the masonry. Peak resistance is governed by the formation and development of diagonal cracks. In the case of the diagonal shear mode, the typical response of masonry walls is characterized by rapid strength and stiffness degradation, moderate energy dissipation and limited displacement capacity. Although the diagonal shear mechanism, which often governs the in-plane response of masonry walls subjected to seismic loads, has limited deformation capacity, a classification of such a mechanism as simply brittle would lead to significant underestimation of the seismic capacity of masonry buildings. Hence, a moderate ductility, or better, a non-negligible nonlinear behaviour and deformation capacity has to be recognised for the diagonal shear failure mode (Magenes and Penna 2011).

Rocking-flexural, or simply flexural, failure usually takes place in the case of a high moment/shear ratio, i.e. in slender walls. As the horizontal load increases, bed joints crack in tension and shear is carried by the compressed masonry. The final failure is obtained by crushing of the compressed corner. In general, the in-plane response of masonry walls failing in the rocking-flexural mode is almost nonlinear elastic with very moderate hysteretic energy dissipation and negligible strength degradation. Regarding the displacement capacity, very large displacements can be obtained, especially when the axial load is low compared to the compressive strength of masonry. Actually, if no other failure mechanisms occur, the displacement, which can be attained in a rocking response, can be limited only by second order (P-Δ) effects associated with overturning (Magenes and Calvi 1997).

A substantial amount of experimental and theoretical work has been carried out to investigate the in-plane seismic response of masonry piers. This paper presents a summary review of these studies, with special attention given to the ultimate deformation capacity of masonry piers. It should be noted that very little experimental and theoretical research has been conducted so far on the response of masonry spandrels and only recently they have been subjected to full-scale in-plane testing (Magenes and Penna Magenes and Penna 2011). Clearly, to better understand the deformation capacity of structural masonry, there is a need for a thorough investigation of the in-plane behaviour of masonry spandrels.

Experimental research on the deformation capacity of unreinforced masonry walls

Experimental research database

As the initial phase of the research project, a review of the technical literature on experimental research on the deformation capacity of structural masonry was conducted. The main objective of the literature review was to provide a comprehensive database of available test results for statistical analysis of the deformation capacity of structural masonry as well as for identifying the decisive parameters for our own experimental work.

Table S1 presents a summary of 71 tests that have been reviewed so far; see Additional file 1: Table S1. It gives information regarding material properties of the constituents: unit dimensions and compressive strengths of units, f_b, mortar, f_m and masonry (perpendicular to the bed joints), f_x; specimen geometry: wall length, l_w, height, h_w and thickness, t_w; boundary conditions and applied vertical pre-compression, σ_0. The database is limited to the shear tests conducted on full-scale unreinforced, unconfined masonry shear walls made of clay bricks and bed joints with general purpose mortar, whose joints had a nominal thickness of 10 mm. Different types of head joints including fully mortared (F), unfilled (U), mortar pocket (MP) and tongue and groove (TG) have been considered. All tests were static-cyclic, except for test MI1m, in which the lateral displacement was applied monotonically. Where the composition of the mortar has not been specified, the mortar is a cement-lime mortar. Further, it should be noted that a considerable number of reviewed shear tests on structural masonry have not been considered in the database since they did not provide useful information on the deformation capacity.

Additional file 1: Table S1 also reports the results of the reviewed tests in terms of failure mechanism and parameters of the idealized bilinear envelope; see Figure 1. Regarding the failure mechanism, the data was classified into four categories: shear-dominated (SH), flexure-dominated (F), sliding (SL) and hybrid (H), i.e. combined shear and flexural, failure modes. The classification was based on the shape of the reported hysteretic loops and available photos, sketches and description of the damage propagation and failure modes given in the reviewed reports. Among the analysed 71 tests, 24 tests (34%) were characterized by the shear-dominated failure mode, 22 tests (31%) by the flexure-dominated failure mode and 21 tests (29%) by the hybrid failure mode. Only 4 tests (6%) failed by sliding.

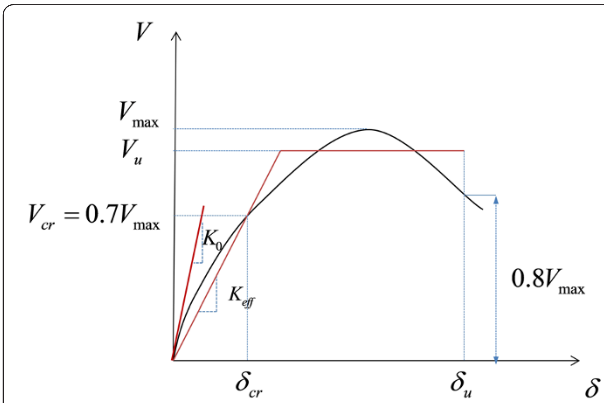

Figure 1 Bilinear idealization of the hysteretic envelope (Tomazevic 1999).

In order to ensure consistency throughout the database, the actual hysteretic behaviour of the walls was approximated by the linear elastic, ideal plastic bilinear envelope illustrated in Figure 1 (Tomazevic 1999). In order to determine the idealized bilinear envelope curve, after constructing the hysteretic envelope, three parameters had to be identified: the effective stiffness (K_{eff}), the ultimate displacement capacity (δ_u) and the ultimate shear strength (V_u). The effective stiffness is calculated from the secant of the cyclic envelope at $0.7V_{max}$, where V_{max} is the maximal lateral load obtained from the test. The ultimate displacement capacity is the displacement corresponding to a strength degradation of 20%. The definition of the ultimate displacement capacity is to some extent subjective. However, the above criterion has been widely used for the definition of the ultimate deformation capacity by a majority of researchers and been adopted by most of the current structural codes. The ultimate shear strength is obtained by equating the areas under the experimental and bilinear envelopes.

Statistical analysis of the experimental data

Figure 2 illustrates the ultimate drift capacity (the ultimate displacement capacity divided by the height of the specimen, δ_u/h_w) for each failure mode except for the sliding failure mode, where the tests were interrupted before reaching the ultimate displacement capacity of the specimens. As mentioned before, the displacement capacity of these walls is very large.

As indicated by Figure 2, the mean values of the ultimate drift capacity for walls failing in flexural, hybrid and shear failure modes were 1.21%, 0.96% and 0.40%, respectively. The deformation capacity of walls that have undergone a

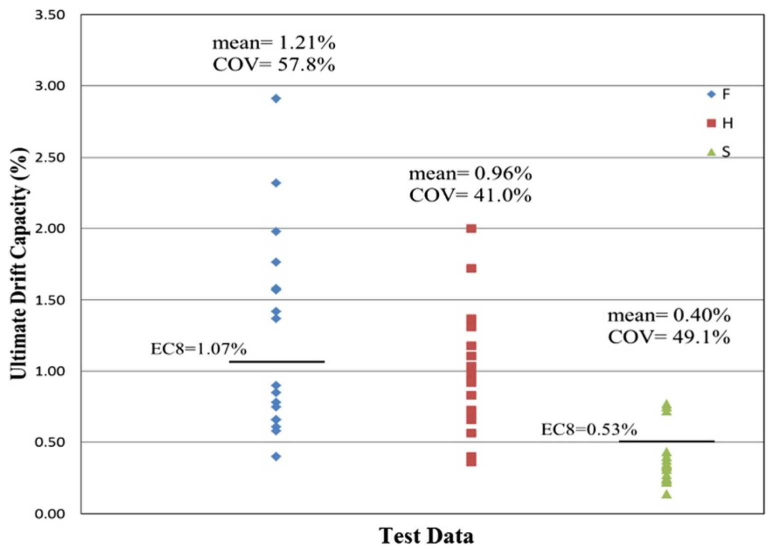

Figure 2 Ultimate drift capacity of unreinforced masonry walls obtained from tests.

shear-dominated failure mode is much lower than that of walls failing by hybrid or flexure-dominated modes. Thus, it was concluded that the global displacement capacity of unreinforced masonry structures is mostly controlled by the displacement capacity of walls failing in shear-dominated modes. In the following, we will discuss the ultimate drift capacity of walls with shear-dominated failure modes in more detail.

Figure 3 illustrates the ultimate drift capacity of walls that failed in the shear-dominated failure mode. The hatched and solid bars correspond to the specimens with cantilever and fixed ends boundary conditions, respectively. As shown, the ultimate drift capacity of walls with shear failure modes ranges between 0.14% and 0.78% with a mean value of 0.40% and a coefficient of variation (COV) of 49.1%. It should be mentioned that tests that did not reach the ultimate limit state have been excluded from the analysis and from Figures 2 and 3 (4 tests in the case of shear-dominated failure). The minimum value of the drift capacity corresponded to wall 16_2. It is important to point out that in this case, such a rather limited ultimate drift capacity of 0.14% is basically due to the

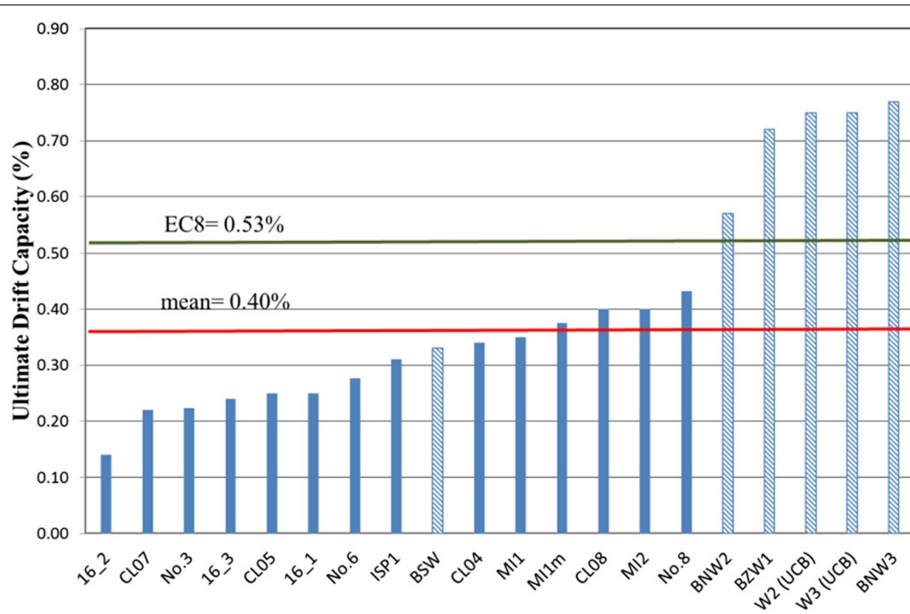

Figure 3 Ultimate drift capacity of walls with shear-dominated failure mode.

definition of the ultimate state as the state corresponding to a strength degradation of 20%. As given in (Frumento et al. 2009) the hysteretic envelope of wall 16_2 exhibits sudden strength degradation after reaching the maximum shear strength, but after that the wall exhibits further deformation capacity before the collapse. A similar behaviour was observed for walls 16_1 and 16_3 from the same reference. Hence, in order to take advantage of the complete deformation capacity of masonry structures, it seems necessary to develop more consistent criteria for the ultimate deformation capacity.

The maximum value of the ultimate drift capacity was reached for specimen BNW3, which was tested under cantilever boundary conditions. For this wall, the early occurrence of shear cracks and the large difference between the cracking load and the maximum shear load were typical (Frumento et al. 2009).

From Figure 3, it can be seen that the specimens with fixed ends boundary conditions exhibited lower drift capacity than those with cantilever boundary conditions. Further inspection of the comparable data revealed that the ultimate drift capacity decreases as the vertical pre-compression increases or as the aspect ratio of the specimen, i.e. h_w/l_w, decreases. The influence of the other factors, e.g., head joint type and size effect (specimens with the same aspect ratio), could not be investigated because of the inhomogeneity of the available experimental data.

It is clear from Figure 2 that the ultimate drift capacity exhibited rather large scatter. The corresponding (large) values of COV for walls with flexure-dominated, hybrid and shear-dominated failure modes were 57.8%, 41.0% and 49.1%, respectively. Due to this scatter, it is not easy to identify a rational value for the ultimate displacement capacity of unreinforced masonry shear walls based only on the available experimental data. However, some guidance for practicing engineers must be provided and such values are given by structural codes. In Europe, the ultimate drift capacity provided by Annex 3 of Eurocode 8 (CEN-EN 1998-3 2005) is 0.53% for unreinforced masonry walls in the shear failure mode and 1.07% for walls failing in the flexural mode for the limit state of Near Collapse (NC). As can be seen from Figures 2 and 3, these values do not always guarantee a safe design.

In general, the deformation capacity of structural masonry is influenced not only by the failure mechanism but also by many other factors, such as constituent materials, geometry, pre-compression level, etc. Hence, despite the inherent randomness associated with masonry as a material, the classification of the URM piers based only on their failure mechanisms seems to be the main reason for the observed scatter in the values of the ultimate drift capacity. Unfortunately, due to inhomogeneous experimental data and a lack of reliable mechanical models (as described in the following sections), we are still not able to properly take into account the influence of all factors affecting the deformation capacity of structural masonry. Furthermore, the assignment of a specific failure mechanism to a test is often a fairly subjective matter. Obviously, to get a clearer picture of the problem, in addition to conducting more tests, we need to develop reliable mechanical models to describe the load-deformation behaviour of structural masonry.

Analytical research on the deformation capacity of unreinforced masonry walls

Computational strategies for structural masonry

A substantial amount of theoretical work has been invested in modelling structural masonry. Simple models are based on linear theory of elasticity and its application to structural masonry. Regarding the serviceability limit state, i.e. when investigating the behaviour of masonry subjected to load levels up to 40-50% of the ultimate load, the applicability of the linear theory of elasticity is beyond dispute. However, when approaching higher load levels nonlinear modelling is generally required. Hereby, both geometrical and material nonlinearities must be taken into account. In general, there are three sources of nonlinearity in the in-plane response of URM piers: geometrical nonlinearity due to the evolutionary partialization of cross-sections as cracking spreads within the panel, material nonlinearity in the elastic range and material nonlinearity in the plastic range (Augenti and Parisi 2009a). Hence, a reliable model must be able to consider all the above-mentioned sources of nonlinearity in a proper way. Since only very few closed-form solutions for nonlinear problems are available, numerical solution methods must be applied. Such solutions are usually obtained by means of Finite Element Method (FEM) procedures. However, the main problem when applying FEM is related to the modelling of the material. Since masonry is composed of two components, i.e. masonry units and mortar, and is highly anisotropic and nonlinear, modelling the physical reality is very demanding.

In general, three different approaches are found in literature for modelling the seismic response of URM structures: micro-modelling, macro-modelling and macro-element discretization. In the micro-modelling strategy, the different components, i.e. the units, mortar, and the unit-mortar interface are distinctly represented. In detailed micro-models, masonry units and mortar joints are represented by continuum elements, whereas the masonry unit-mortar interface is represented by discontinuous elements (Figure 4a). Detailed micro-modelling requires much computational effort. This drawback is partially overcome by the simplified micro-models. In the simplified micro-modelling strategy, masonry units are represented by continuum elements whilst the mortar joints and masonry unit-mortar interface are lumped into discontinuous

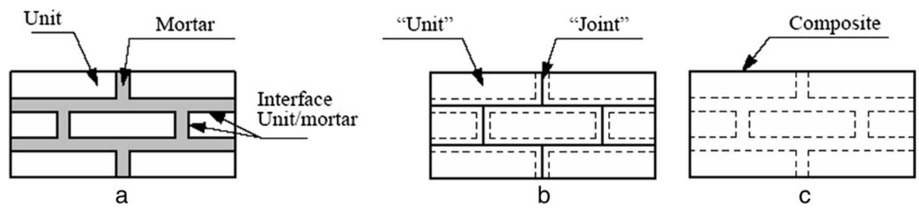

Figure 4 Detailed (a) and simplified (b) micro-modelling; macro-modelling (c) (Lourenco 1996).

elements (Figure 4b). The micro-modelling approaches are suitable for small structural elements with particular interest in strongly heterogeneous states of stress and strain. The primary aim is to closely represent masonry based on knowledge of the properties of each constituent and of the interface (Roca et al. 2010). In the macro-modelling strategy, masonry is treated as a fictitious homogeneous orthotropic continuum with different tensile and compressive strengths as well as different inelastic properties along the material axes (Figure 4c). In particular, FE meshes are simpler since they do not have to accurately describe the internal structure of masonry and the finite elements can have dimensions greater than the single brick units (Roca et al. 2010).

Although significant progress has been made in the field of micro- and macro-modelling strategies, e.g., development of the so-called homogenized modelling, see, e.g., (Lourenco et al. 2007), these approaches are still not suitable for the analysis of whole buildings in everyday engineering practice. This is because a considerable number of material parameters are needed as input for a meaningful analysis using these approaches, and these parameters are usually unavailable. Furthermore, the current micro and macro models have a limited range of validity and also require significant computational resources and high expertise. In addition, due to the great difficulty in the formulation of robust numerical algorithms representing satisfactorily the inelastic behaviour of masonry, micro and macro analyses of masonry structures are often limited to the structural pre-peak regime (Maruccio 2010 and Xu et al. 2012). However, the importance of the post-peak response is evident in order to evaluate the deformation capacity and to assess the structural safety.

As a consequence, several methods based on macro-element discretization have been developed, particularly in Italy. In this approach, each panel in the structure, i.e. piers and spandrels, is modelled by using a single element. Such elements called macro-elements are based on the simplification of both the material behaviour and the stress field within the panel. These elements seem to be the most appropriate for the design and assessment of masonry buildings because of the simplicity of modelling, the straightforward interpretation of the results, particularly in terms of collapse mechanisms, and the accuracy demonstrated in different validations (Lourenco et al. 2009 and Grande et al. 2011). The use of macro-elements for the nonlinear analysis of masonry structures has been introduced in several guidelines, e.g., FEMA 356 (Applied Technology Council ATC 2000) and Eurocode 8 (CEN-EN 1998-3 2005), with particular reference to the use of the pushover analysis method. Given the above, in the following section a review on macro-elements for the seismic analysis of unreinforced masonry piers is presented with emphasis on the ways in which they deal with the deformation capacity. Readers interested in the micro- and macro-modelling approaches can find a recent comprehensive review in (Roca et al. 2010).

Macro-elements for URM piers

One-dimensional macro-elements

The simplest one-dimensional macro-elements are single shear springs which represent experimental resistance envelopes of URM piers with idealized bilinear (linear elastic, perfectly plastic) relationships. In order to determine an idealized bilinear envelope curve, three parameters must be identified: the effective stiffness (K_{eff}), the ultimate shear strength (V_u) and the ultimate displacement capacity (δ_u).

In general, the effective stiffness is a complex parameter and difficult to determine. For practical applications it is usually taken as the elastic stiffness, K_0 (see Figure 1), which is calculated based on the elastic beam theory incorporating shear deformation, or as 50% of the elastic stiffness; see, e.g., (Tomazevic 1999), Eurocode 8 and FEMA 356. It should be noted here that, in general, the determination of the effective stiffness is subject to variation and the data obtained from tests exhibits a large scatter. Comparison with experimental results shows that the effective stiffness varies between 40% and 80% of the elastic stiffness and it may be influenced by the pre-compression level. The influence of the pre-compression level on the effective stiffness is a controversial subject. While some experiments have indicated that the ratio between the effective stiffness and the elastic stiffness (K_{eff}/K_0) strongly depends on the pre-compression level, e.g., (Bosiljkov et al. 2005), no such dependencies were found in other experiments; see, e.g., (Magenes et al. 2008). Regarding the ultimate shear strength, it was found by evaluating the results of several tests that the choice of $V_u = 0.9V_{max}$ is appropriate for the ultimate shear strength (Tomazevic 1999). The maximum shear resistance of URM piers, V_{max}, can be predicted with acceptable accuracy using the formulations provided by the codes of practice. Different recommendations could be found in the literature for the ultimate displacement capacity parameter that are in effect based on a statistical analysis of the results of past experiments. Unfortunately, the proposed values are not always readily applicable, because as discussed before the data obtained from tests exhibits a rather large scatter.

Among the methods using bilinear shear springs, the assessment method proposed by (Tomazevic 1978) which is usually known as the POR method and the FEMA 356 method are well-known and extensively used. The POR method is an equivalent static, simplified nonlinear assessment method which assumes that the failure occurs only in the piers without any damage of spandrels. This method, which is historically the first seismic assessment method for structural masonry, is based on the storey mechanism approach. The procedure consists of a separate inter-storey shear-displacement curve for each storey, in which each masonry pier is typically modelled by a linear elastic, perfectly plastic shear spring of limited ductility. The FEMA 356 method also employs nonlinear shear springs to model the force-displacement response of individual piers. The spandrels of URM walls are only considered to affect the boundary conditions of the piers, i.e. fixed ends or cantilever. The force-displacement relationship for each pier is defined based on the governing failure mode, which is taken as the failure mode of least lateral resistance. In conclusion, the macro-elements that are based on bilinear idealization of the experimental resistance envelope are not reliable, particularly regarding the prediction of the ultimate displacement capacity, due to large scatter in available experimental data.

(Magenes and Della Fontana 1998) and (Magenes 2000) proposed an improvement to the POR method based on the so-called equivalent frame idealization. In the proposed method, termed SAM (Simplified Analysis of Masonry buildings), both the spandrels and the piers are modelled as beam-column elements with shear deformation, while their intersections are modelled by means of rigid offsets at the ends of the pier and spandrel elements. To describe the nonlinear response of piers and spandrels, the SAM method employs several plastic hinges that are located by the user to account for the possible failure modes. Typically these plastic hinges are placed at both ends and at the mid-span of the beam-column elements to capture the flexural and the shear failure modes, respectively. This approach can be easily implemented by the conventional commercial programs, e.g., SAP2000 (Pasticier et al. 2008). However, since the properties of the plastic hinges are mainly based on the available experimental data, the model suffers from the lack of reliability of the results, particularly in terms of the displacement capacity.

Figure 5a shows the macro-elements proposed by (Chen et al. 2008) for URM piers and spandrels. The proposed approach is meant to be used in conjunction with FEMA 356. This approach, which provides rotational, shear, and axial springs in series, was first introduced for the analysis of reinforced concrete (RC) shear walls (Kabeyasawa et al. 1982 and James and Kunnath 1994). The elements developed for RC shear walls were improved for the modelling of URM piers by the addition of two shear springs at the top and bottom of the macro-element to account for bed joint sliding deformation in these regions. For modelling spandrels constructed of running bond masonry, these sliding springs are not needed because the interlocking of

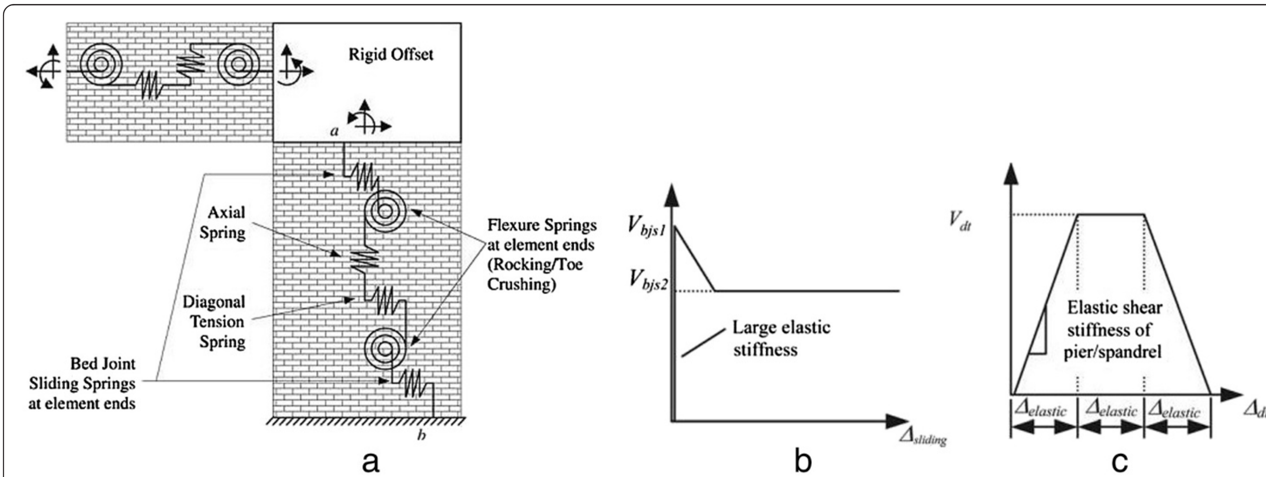

Figure 5 Macro-elements for URM piers and spandrels (a); adopted force-displacement relationships for bed joint sliding (b) and diagonal shear behaviour (c) (Chen et al. 2008).

units prevents sliding along head joints. As shown in Figure 5a, these macro-elements have three degrees of freedom (DOF) at each end and thus can be used within the equivalent frame idealization strategy for the analysis of perforated URM walls.

Regarding the pier element, the axial spring has linear elastic response in compression and no resistance to tensile forces beyond the tensile strength of masonry. The properties of the flexural springs are based on the moment-curvature response of the top and bottom sections of the pier, which are established through the use of a fibre model. The moment-rotation properties of the rotational springs are then obtained by integrating the curvature along the height of the pier. The adopted force-displacement relationships for the shear springs corresponding to the bed joint sliding and diagonal shear behaviour are shown in Figures 5b and c.

Figure 6 provides a comparison between the results of the macro-element simulation and experimentally obtained hysteresis response for the different failure modes. It can be seen that in the case of rocking-flexural and shear sliding failure modes, the simulation results are almost satisfactory, but in the case of diagonal shear failure, the simulation is highly erroneous. The error arises mainly from the adopted force-displacement response for the diagonal shear spring and also from the inability of the element to model the geometrical nonlinearity.

Two-dimensional macro-elements

There are some limitations in the use of one-dimensional macro-elements, namely due to inaccurate simulation of the interaction between piers and spandrels, and due to the weak modelling of the geometrical nonlinearity of the panels (Marques and Lourenco 2011). Two-dimensional macro-elements cannot be applied within the equivalent frame idealization framework and require more computational effort compared to one-dimensional macro-elements. However, they offer a more accurate simulation of the nonlinear response of the masonry piers and spandrels, particularly in the pre-peak regime. This is because unlike one-dimensional elements, these elements are able to simulate the propagation of the tensile cracks along the height of the pier, i.e. geometrical nonlinearity.

The "no-tension multi-fan panel element" was developed by (Braga and Liberatore 1990) based on the idea that the stress field of a masonry panel with free edges

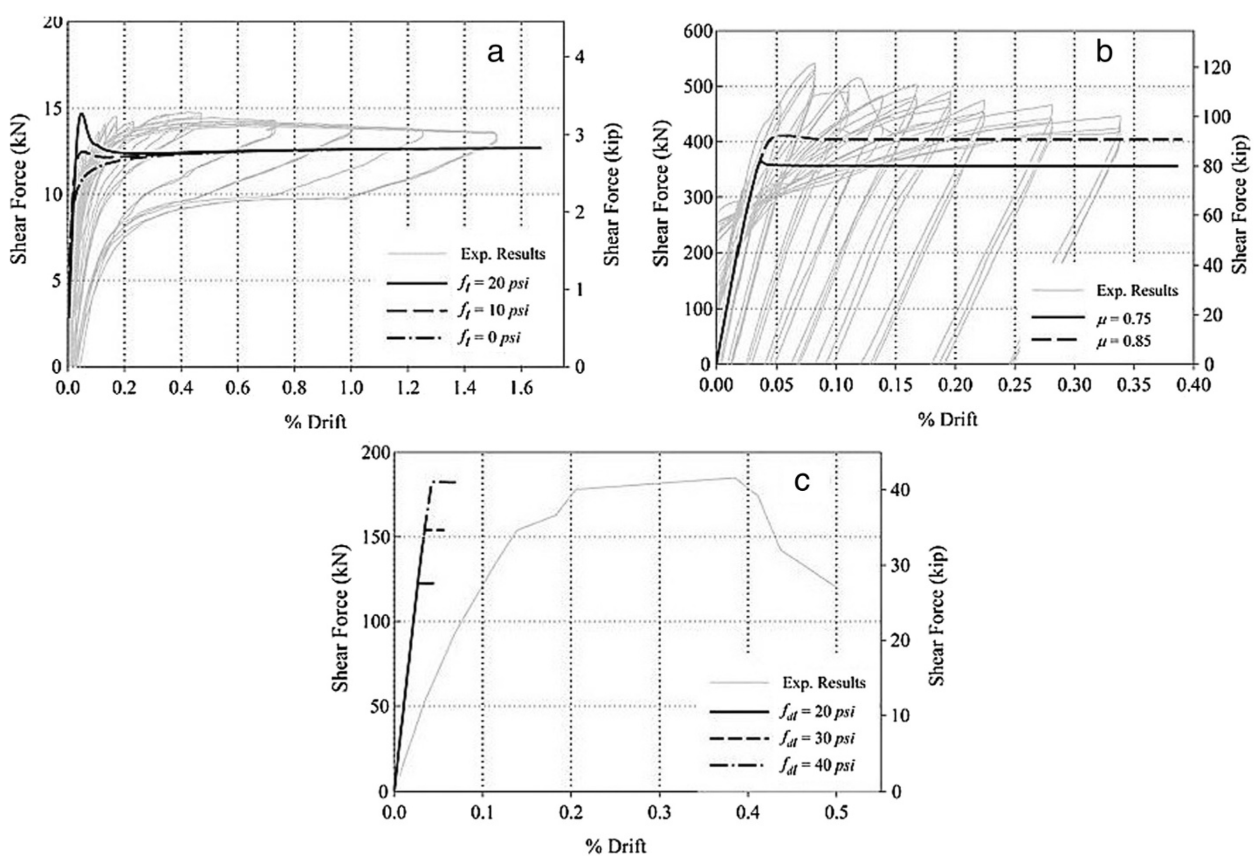

Figure 6 Comparison between the macro-element simulation and experimental results for flexural (a), bed joint sliding (b) and diagonal shear (c) failure modes (Chen et al. 2008).

follows a multi-fan pattern (Figure 7a). In addition, it is assumed that the upper and lower faces of the panel are rigid, and that there is no interaction in the circumferential direction between the infinitesimal fans. The material behaviour is assumed linear elastic in compression and non-reacting in tension. There is very good agreement between simulation results obtained using the multi-fan element and experimental results up to a certain level of lateral displacement, but for larger displacements, the accuracy of simulation decreases rapidly because of the adopted elastic constitutive law (Liberatore et al. 1996).

A modification of the multi-fan element is proposed by (Maruccio 2010). The updated multi-fan element, introduces zero-length springs into the multi-fan element to add failure mechanisms in the constitutive law; see Figure 7b. In fact, the behaviour in the elastic stage is defined by a set of radial stress fields in the panel, while the springs are required to define failure mechanisms and the inelastic response. The properties of these springs are based on past component tests. Therefore, as discussed previously in detail, the simulation results could not be considered reliable in terms of displacement capacity. The multi-fan model seems to have a considerable potential for further development.

(Yi et al. 2005) proposed a macro-element, named “Effective pier model”, to describe the nonlinear behaviour of an individual URM pier subjected to external forces. The model describes the effective area of the pier by eliminating the flexural tensile cracks and the toe crushing regions; see Figure 8a. Regarding toe crushing, the model assumes that the compressive strength of masonry immediately drops to zero after the ultimate compressive strength of the masonry has been exceeded. Although this assumption is questionable, it greatly simplifies the problem and results in a conservative strength estimate (Yi et al. 2005). However, more sophisticated stress-strain relationships that account for the nonlinear stress-strain behaviour of masonry can be employed if needed; e.g., the macro-element model developed by (Augenti and Parisi 2009b). In order to address the problem of bed joint shear sliding, the model employs the Mohr-Coulomb friction model. Regarding diagonal shear, a smeared crack technique is employed. It is assumed that the effective area of the pier remains continuous even after the development of diagonal cracks. To model the potentially rapid and unstable propagation of these cracks, the effective tangent elastic modulus of masonry is assumed to have a negative value. Since no test data is available for the softening behaviour of URM piers after diagonal cracking, the tangent modulus of URM piers with diagonal shear crack is set equal to–*0.1E*, where E is the initial elastic modulus of masonry (Figure 8b).

According to this model, when a URM pier experiences a reduction of cross section due to either tensile or compressive failure, the remaining part of the pier will typically be inclined at some angle. Hence, after cracking, a portion of the lateral force will be resisted through axial deformation, and the lateral force that causes shear and flexural deformation will be $V - P\ tan(\theta)$, where θ is the angle between the central axis of the pier and the vertical line, and V and P are applied lateral and vertical loads, respectively. The proposed model only considers the lateral deformation of the pier caused by flexure and shear:

$$\Delta = \frac{V - P\tan(\theta)}{K} \qquad K = 1/\left[\frac{4\gamma h^3}{EL^3 t} + \frac{h}{GLt}\right]$$

where Δ is the lateral deformation; K is the lateral stiffness of the pier; γ is a coefficient that describes the boundary conditions of the pier (γ is equal to 0.83 for fixed ends and 3.33 for cantilever boundary conditions); E is the elastic modulus of masonry, and G is the shear modulus of masonry, which is taken as *0.4E*. In order to consider the nonlinear compressive stress-strain behaviour of the

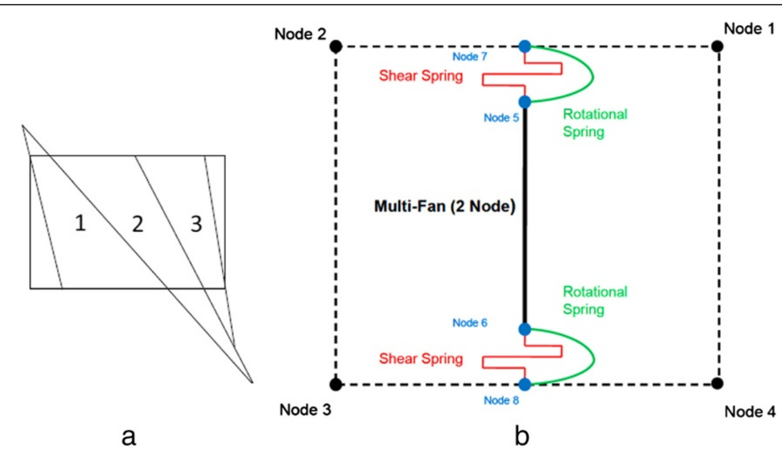

Figure 7 **Multi-fan element (a); modified multi-fan element (b) (Maruccio 2010).**

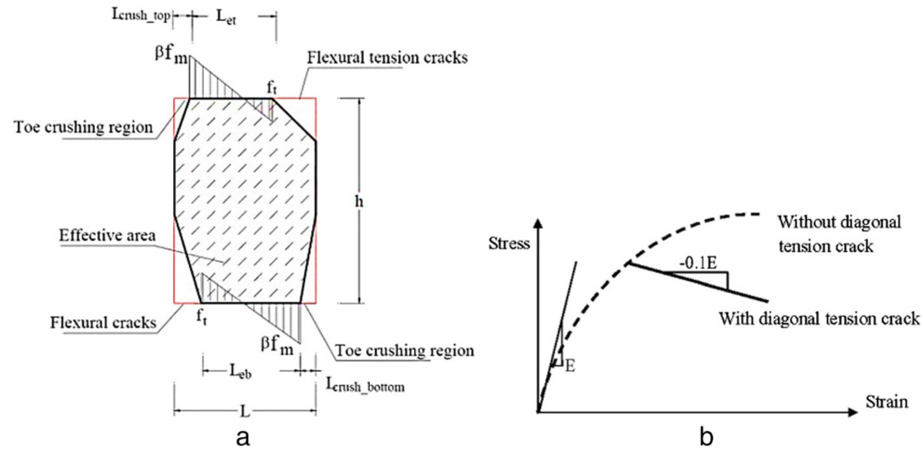

Figure 8 **Effective pier model (a); principal compressive stress-strain relationship of masonry before and after diagonal shear cracks (b) (Yi et al. 2005).**

masonry, the model uses the relationship proposed by (Naraine and Sinha 1989).

Figure 9 provides a comparison between the results of the effective pier model and the experimentally obtained hysteresis response for the different failure modes. It can be seen that the effective pier model is able to partially predict the behaviour of masonry piers failing in sliding shear or rocking-flexural modes, but in the case of diagonal shear, there is no good agreement between the model and the experimental results (except at the beginning of the

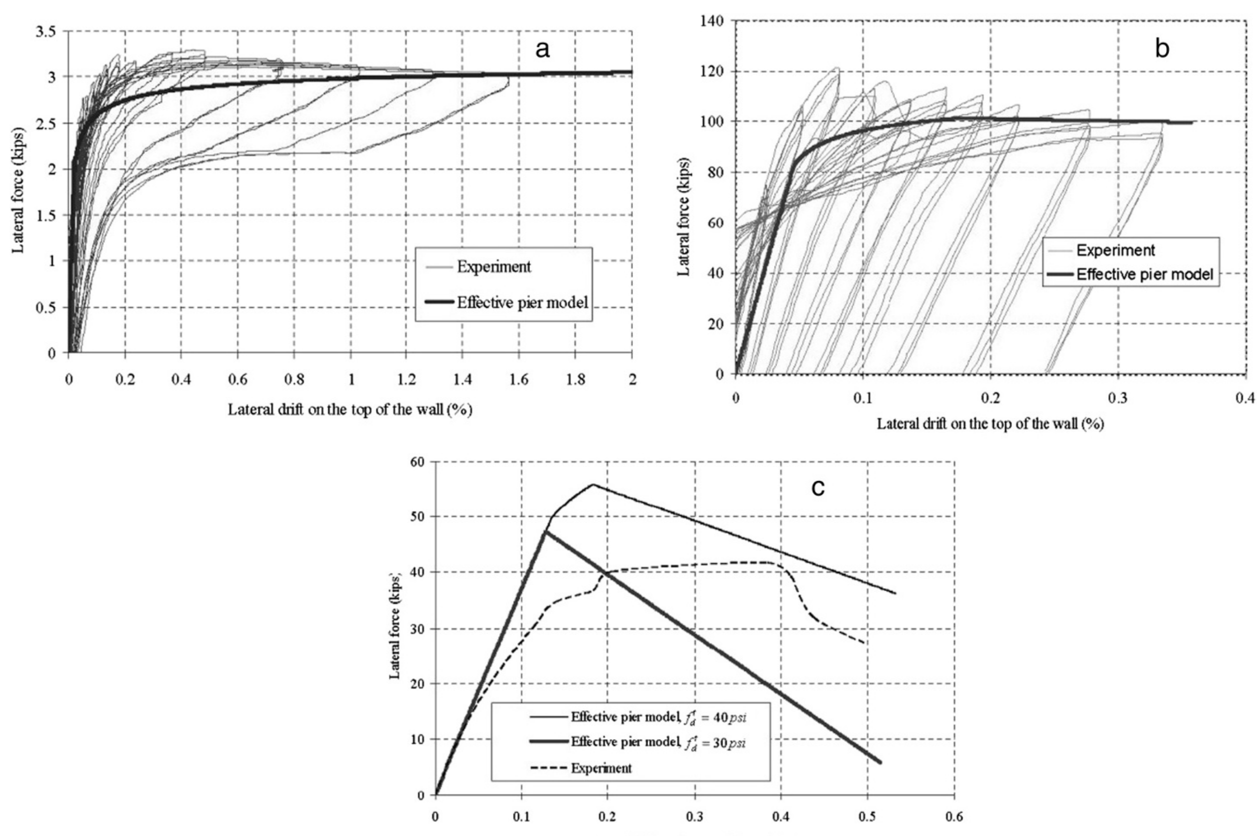

Figure 9 **Macro-element simulation of the flexural (a), bed joint sliding (b) and diagonal shear (c) response using Effective pier model (Yi et al. 2005).**

response), which could be attributed to the assumed stress field and stress-strain relationship after the development of diagonal shear cracks.

In conclusion, although available macro-elements (specifically two-dimensional ones) provide acceptable results in the case of sliding and rocking-flexural failure modes, they are still not able to simulate the response of unreinforced masonry piers with the diagonal shear failure mode closely enough. In general, there are two main obstacles to the simulation of the in-plane response of URM piers, especially in the post-peak regime: complexity of the stress field after development of the shear cracks and more important, lack of experimental data about the softening of masonry (as a material) under biaxial loading. Furthermore, current macro-elements cannot distinguish between the diagonal cracks passing through the units and those running through bed and head joints, while in the former case, the deformation capacity is limited, but in the latter case there is a considerable displacement capacity.

Conclusions

The ultimate deformation capacity is the most important parameter in the seismic design and evaluation of structures. Our current state of knowledge about the deformation capacity of structural masonry is limited. The available experimental data exhibits too much scatter and it is not possible to identify a rational value for the deformation capacity of masonry structures based only on such experimental data. Furthermore, there are no reliable and practical analytical models for the force-deformation relationship of structural masonry: refined finite element models, besides being too complex for everyday engineering practice, suffer from numerical instabilities in the post-peak regime, and available structural macro-elements are still so far from being considered accurate enough regarding the deformation capacity parameter, especially in the case of the diagonal shear failure mode. It should be mentioned here that there are several other macro-elements that have not been reviewed in this paper because they do not introduce further improvement or different concepts regarding the deformation capacity.

Obviously, to get a clearer picture of the problem, in addition to conducting more tests, we need to develop reliable mechanical models to describe the load-deformation behaviour of structural masonry. This task is being approached within the framework of the current research project. The research project includes several cyclic quasi-static shear tests on full-scale, storey-high masonry walls as well as developing and introducing new sophisticated mechanical models for structural masonry. A novel approach will be developed and utilized for the purpose of applying experimental evidence collected from our own tests performed for the development of reliable mechanical models.

Additional file

Additional file 1: Table S1. Summary of the experimental studies on the deformation capacity of unreinforced masonry walls.

Competing interests
The authors declare that they do not have any competing interests.

Authors' contributions
AHS carried out the review and drafted the manuscript. NM and JS supervised the research. All authors read and approved the final manuscript.

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Author details
[1]Institute of Structural Engineering, ETH Zurich, CH-8093, Zurich, Switzerland. [2]Institute of Technology in Architecture, ETH Zurich, CH-8093, Zurich, Switzerland.

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