Seismic Evaluation and Rehabilitation of Masonry Buildings

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Abstract

Considerations on seismic assessment and retrofit of unreinforced masonry buildings are presented with special reference to the recent reasearch activity coordinated by the Italian National Group for the Defense from Earthquakes (GNDT).

Attention is first paid to the study of the response of simple walls. With reference to the fundamental failure mechanisms, recent developments for strength evaluation are presented. The role of shear ratio in the shear strength of URM walls is put in evidence and strength formulas suitable for design and assessment are proposed. The deformational behaviour is also discussed showing how ultimate drift seems to be a meaningful parameter for walls failing in shear. Energy dissipation properties are also discussed, on the basis of experimental results.

The role of experimental testing is briefly discussed, summarizing issues which are meaningful for design and assessment applications.

Structural systems (buildings) are then considered, focusing on simplified methods of structural analysis for seismic assessment. The requirements of simplified methods are discussed, and the progress made in Italy is presented, showing how nonlinear static analysis of multistorey walls have reached a rather satisfactory level of development.

The choice of retrofitting techniques is discussed with regard to the general philosophy of seismic design. Emphasis is put on the increase of ductility as an effective means to enhance seismic performance. The potentials of selected, stable, mechanisms of inelastic behavior of masonry are pointed out. At the same time, attention is drawn to the fact that retrofitting techniques that increase strength may not necessarily have positive effects under severe earthquakes, if the increase in strength triggers brittle failure mechanims.

Needs for future research are pointed out in the course of the paper.

Introduction

Unreinforced masonry (URM) buildings in seismic zones constitute a hazard which has generated significant interest in the development of assessment, analysis, and retrofit methods which are appropriate for these structures. The variety of these buildings, most of which were constructed before the development of rational engineering design procedures, calls for rational approaches to assessment, well supported and validated by experimental research. At the same time, the large number of such buildings in most Italian urban nuclei calls for approaches that are simple enough to be extensively adopted in real engineering practice. The attainment of a rational compromise between reliability and simplicity is one of the most challenging tasks of the present research on seismic assessment. Within this context, the role of experimental research and refined numerical modelling is to give a reference for the calibration of simplified procedures for assessment and retrofit design.

Current established procedures tend to be rather rudimentary, and of the 'walk through' variety, where specific details of the buildings are compared with a check list of possible deficiencies, and where calculations, if carried out at all, are of a simplistic nature, inadequate to determine the probable response. Vulnerability indices generated for statistical elaboration, production of damage scenarios and global risk evaluation at a territorial level, are often being inappropriately used to determine seismic risk of individual buildings. It is obvious that the application of a mean value from a data set with extremely wide scatter will provide little insight beyond indicating the need for more detailed structural calculations. Unfortunately risk analyses are being routinely used to guide retrofit decisions and strategies for specific buildings.

In this paper some recent developments in the field of URM buildings assessment are presented, with particular reference to the research sponsored by CNR-GNDT in Italy. The conceptual evolution of the research can be summarized in the following points:

- identification of basic resisting elements and connections based on available models;
- tests on materials, structural elements and subassemblages;
- development of improved models for structural elements;
- development of refined and simplified numerical models;
- prediction of the global response of whole buildings based on simplified and refined models;
- tests on whole buildings to validate the models;
- parametric analyses using refined models;
- validation of simplified models based on the refined models;
- conceptual development of strengthening techniques;
- parametric analyses of retrofitted buildings;
- full scale tests on retrofitted buildings.

The main results of the research project are summarized in this paper, dealing first with problems related to the behaviour of simple walls, and then focusing the discussion on structural systems. Some indications are also given on the most appropriate use and relative merit of different testing procedures.

It must be stressed that the discussion presented here is based mostly on results obtained on one particular type of masonry, namely fired solid clay brick masonry. Extension to other similar materials may be possible, provided that appropriate experimental data are made available.

Strength of masonry walls

Typical unreinforced masonry buildings are likely to be composed of several loadbearing 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. Once out-of-plane failure is prevented by proper interventions (e.g.reinforced concrete ring beams, steel ties at the floor levels...) the in-plane walls provide the stability necessary to avoid collapse. Usually, these walls are pierced by windows or doors, leaving a series of smaller piers to provide both the gravity and lateral load resisting systems. Mechanisms of lateral load resistance depend primarily on the pier geometry, on its boundary conditions and on the magnitude of vertical loads, and then on the characteristics of the brick, of the mortar and of the brick/mortar interface. Flexural response tends to be dominated by rocking of piers rather than "beam" type behavior, and failure is generally characterized by shear, either in a diagonal tension mode or by sliding on the predefined planes of the mortar joints. The inhomogeneous, composite nature of masonry makes component behavior difficult to predict, and also difficult to replicate in the laboratory.

The principal failure mechanisms of masonry piers subjected to seismic actions can be summarized as follows:

1. rocking failure: as horizontal load or displacement demand increase, bedjoints crack in tension and shear is carried by the compressed masonry; final failure is obtained by overturning of the wall and simultaneuos crushing of the compressed corner.
2. shear cracking: peak resistance is governed by the formation and development of inclined diagonal cracks, which may follow the path of bed- and headjoints or may go through the bricks, depending on the relative strength of mortar joints, brick-mortar interface, and brick units.
3. sliding: due to the formation of tensile horizontal crack in the bedjoints, subjected to reversed seismic action, potential sliding planes can form along the cracked bedjoints; this failure mode is possible for low levels of vertical load and/or low friction coefficients.

The strength associated to these failure modes can be described simply, sacrificing some accuracy to the purpose of emphasizing the relative importance of various parameters on the response of piers. Possible approaches, based on experimental experiences developed so far by several researchers, are here summarized.

Rocking strength

The maximum horizontal shear which can be resisted by a *rocking* pier failing under static in-plane loading may be approximated introducing a proper stress distribution for the masonry in compression and neglecting the tensile strength of bedjoints. With reference to Figure 1, equilibrium leads to the following expression:

$$V_r = \frac{D^2 t}{H_0}\frac{p}{2}\left(1-\frac{p}{\kappa f_u}\right) = \frac{Dt}{\alpha_V}\frac{p}{2}\left(1-\frac{p}{\kappa f_u}\right) \tag{1}$$

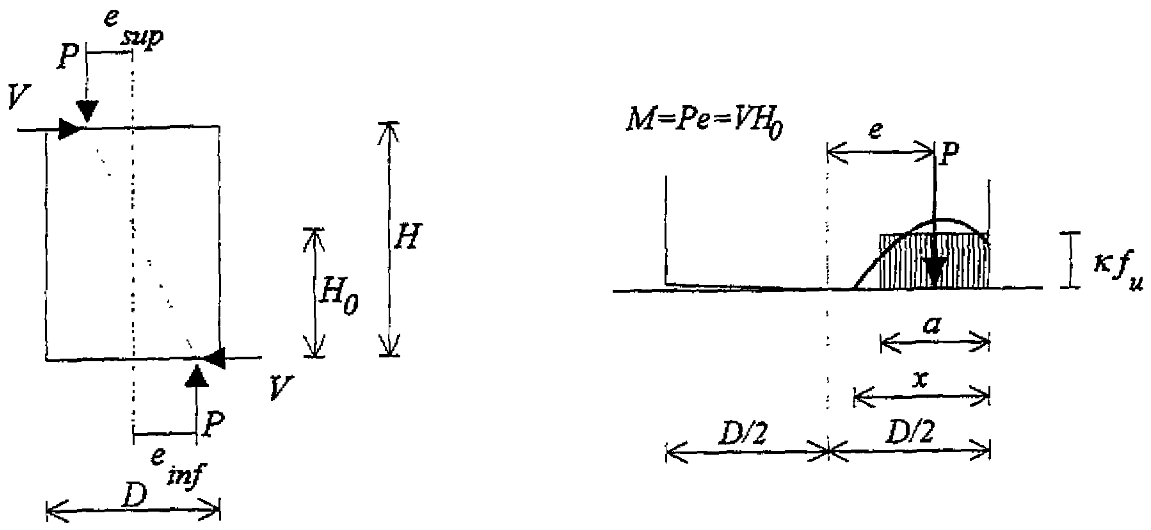


Figure 1. Assumptions for rocking strength evaluation of a wall failing with crushing at the base corner.

where D is the pier width, H_0 is the effective pier height (distance from zero moment), t the pier thickness, $p=P/(Dt)$ is the mean vertical stress on the pier due to the axial load P, f_u is the compressive strength of masonry, κ is a coefficient which takes into account the vertical stress distribution at the compressed toe (a common assumption is an equivalent rectangular stress block with κ=0.85). The effective height H_0 is determined by the boundary conditions of the wall and is related to the shear ratio α_V :

$$\alpha_V = \frac{M}{VD} = \frac{H_0}{D} = \frac{\psi' H}{D} \tag{2}$$

Considering typical test layouts, the parameter ψ' assumes a value of 1 when the pier is fixed on one end and free to rotate on the other, and a value of 0.5 when the pier is fixed at both ends. Equation (1) has a low sensitivity to the parameters κ and f_u , in the range of low mean vertical stresses (i.e. for p/f_u lower than 0.2), while it is strongly affected by parameter α_V (and consequently by ψ').

Shear strength

The ability to predict shear strength with simple design formulas is, as for reinforced members, less satisfactory than for flexural strength. Different formulations were proposed in the past, each of them having its own merits and drawbacks. In a recent work by Magenes and Calvi (19), the problem of shear strength formulation for brick masonry walls was discussed, on the basis of experimental testing and numerical simulations. In that work, attention was drawn to the need for both "cracked section" and "whole section" strength formulations, and to the role played by aspect ratio and/or shear ratio.

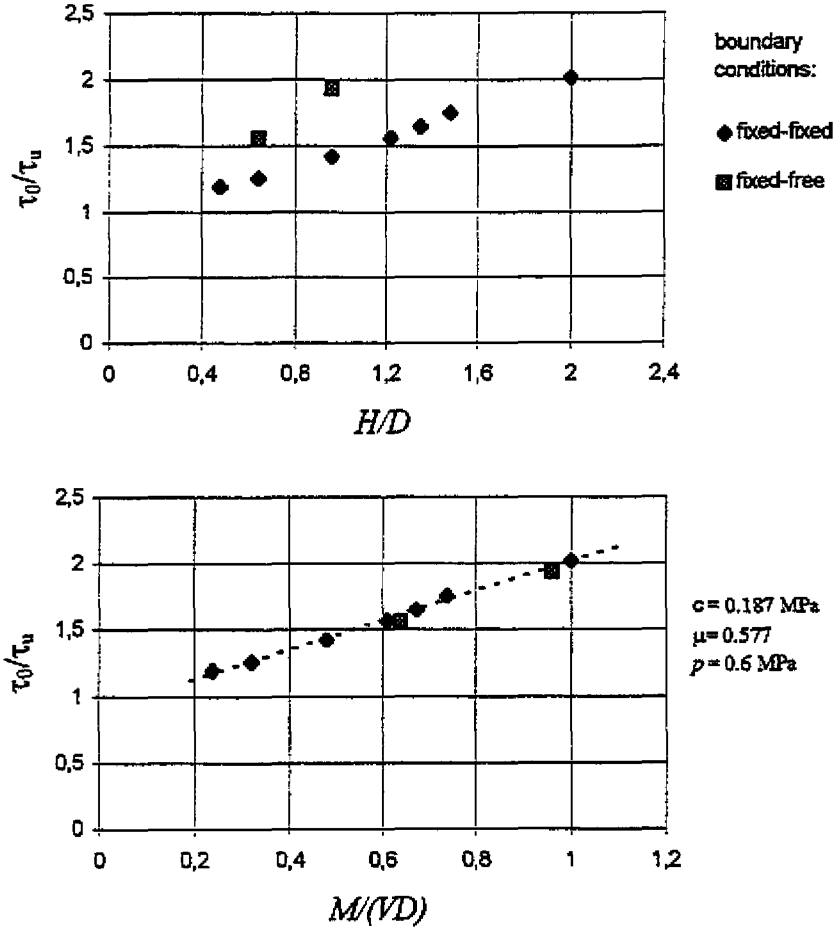


Figure 2. Influence of aspect ratio and shear ratio on shear strength of masonry walls from f.e.m. simulations; $\tau_0 = c + \mu p$, $\tau_u = V_u / Dt$, V_u peak strength from f.e.m. simulation.

Regarding the first issue, shear failure can be associated to unstable crack propagation which can take place in the extreme sections of a wall, already cracked in flexure, or in the centre of the wall, in the shape of the classic diagonal cracks. A proper approach to the evaluation of shear strength should therefore account for both possible mechanisms.

Regarding the issue of the role played by the aspect ratio (*H/D*) or the shear ratio ($\alpha_V = M/(VD) = H_0 / D$), it has long been known from experiments (26, 28) that the mean shear strength of walls increases as the wall gets squatter and viceversa, but since the experimental tests were usually being made with only one type of boundary conditions, aspect ratio and shear ratio varied in the same proportion. As a consequence, some strength formulas introduce a correction factor based on the aspect ratio, while in other formulas the shear ratio is used. Numerical nonlinear f.e.m. simulations, presented in (19), where the change of boundary conditions was easily performed numerically, showed that the dominating factor is the shear ratio α_V. Keeping all other parameter constant, the mean shear strength $\tau_u = V_d / (Dt)$ decreases hyperbolically as the shear ratio increases, or, in a simpler way, the reciprocal of τ_u increases linearly with the shear ratio, as shown in Figure 2. Such influence of the shear ratio is apparently not influenced by the type of damage propagation. A proper simplified strength formulation could thus reflect such influence with a correction factor which is a linear function of α_V. A simple approach for shear failure governed by joint failure is proposed as follows:

$$V_d = Dt\,\tau_u \qquad \text{with } \tau_u = \min(\tau_{cs};\tau_{ws}) \tag{3a}$$

$$\tau_{cs} = \frac{1.5c + \mu p}{1 + 3\dfrac{c\alpha_V}{p}} \text{ relevant to the cracked section} \tag{3b}$$

$$\tau_{ws} = \frac{c + \mu p}{1 + \alpha_V} \text{ relevant to the whole section} \tag{3c}$$

By using the same values for c and μ which were used in the constitutive model for the finite element analyses, the strength criterion given by Eqs. 3 a,b,c would give the results shown in Figure 3. Given the simplicity of the formulation, the results are satisfactory when compared to the results of the finite element simulations, giving errors of less than 10%.

For the application of the proposed strength criterion to real walls two considerations must be made. First, the use of the cohesion and of the friction coefficient of bedjoints, as derived from laboratory or in-situ tests, can often lead to an overestimation of the real strength of a wall. This can be in good part due to the presence of weak headjoints, which are not modelled in the finite element analyses shown here. The role of weak headjoints and a possible correction of the cohesion and friction coefficient were discussed by Mann and Müller (21). The corrected cohesion and friction are expressed as $\bar{c} = \kappa \cdot c$, $\bar{\mu} = \kappa \cdot \mu$, where:

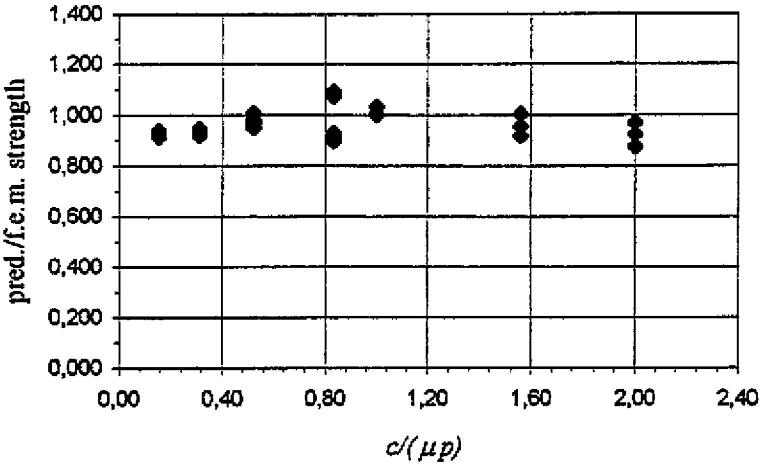
$$\kappa = \frac{1}{1 + \mu \frac{2\Delta_y}{\Delta_x}} \tag{4}$$

and Δ_x, Δ_y, respectively are the length and height of the brick unit.

Furthermore, equations 3a,b,c account only for shear failures governed by joint failures, while experiments show that for high axial loads or high mortar strength the failure may be initiated by shear-tensile cracking of bricks. Again, a possible simple formulation of shear strength based on cracking of bricks, given the tensile strength of bricks f_{bt}, can be found in Mann and Müller (21). Taking into account the influence of the aspect ratio, the shear failure criterion associated to brick unit cracking can be expressed as:

$$V_{d,b} = Dt \cdot \tau_{ub} = Dt \cdot \frac{f_{bt}}{2.3 \cdot (1 + \alpha_V)} \sqrt{1 + \frac{p}{f_{bt}}} \tag{5}$$

The set of strength formulas (3-5), which describe the possible shear failure mechanisms, was checked against experimental results (19) taken from shear tests on URM panels (2,16) The strength predictions appeared to be satisfactory, slightly conservative, and consistent with the



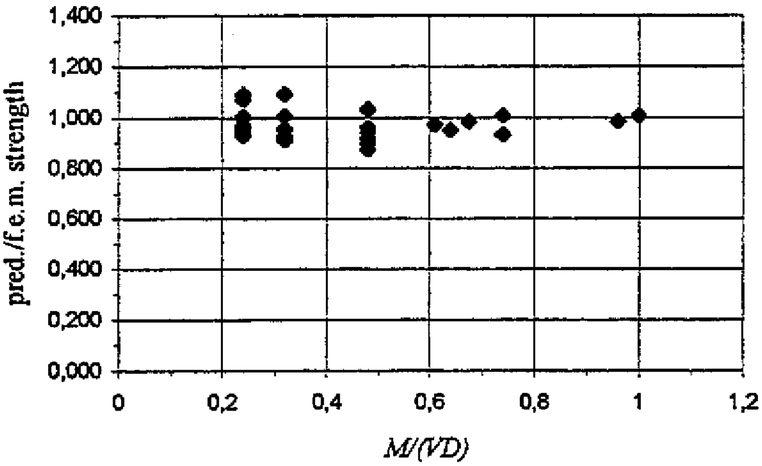


Figure 3. Comparison of the proposed simplified shear strength prediction formulas with the strengths obtained from f.e.m. simulations of shear tests of walls.

failure modes which were experimentally reported. The scatter was comparable with the scatter of the mechanical properties of masonry.

Sliding strength

The strength of a pier undergoing *sliding*, under seismic excitation, along a horizontal joint is often expressed as:

$$V_s = \mu P \tag{6}$$

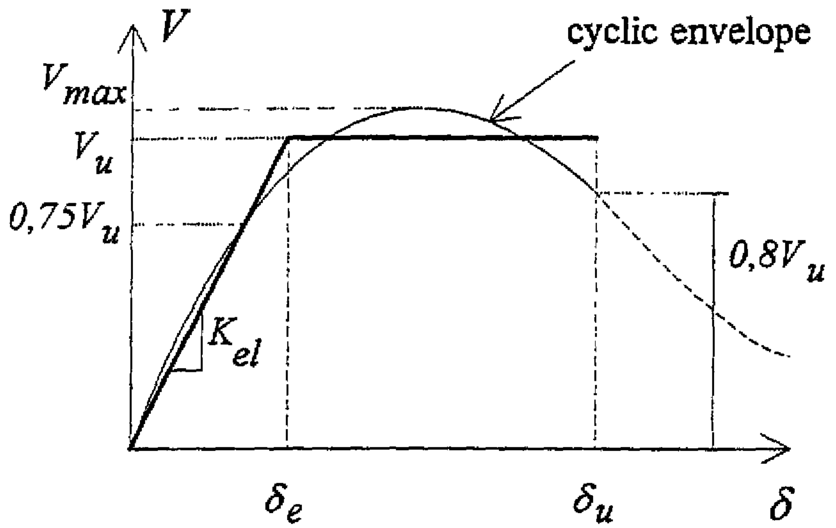


Figure 4. Definition of an equivalent bilinear envelope. $V_u = 0.9\ V_{max}$ for shear failure.

where μ 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 alternate flexure. If the coefficient μ is put equal to the residual friction of a sliding bedjoints, Equation 6 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 bedjoint failing in shear. The use of a nonzero reduced cohesion value in an expression like Equation 3b could be considered more appropriate. However no definitive experimental reference is available at present.

Deformation capacity and stiffness evaluation of URM walls

Response of brick masonry walls is strongly nonlinear also at low level of load, due to the low tensile strength of bed and headjoints. As the damage due to cracking increases, masonry walls show both strength and stiffness degradation. A definition of elastic stiffness of a wall subjected to in-plane shear must therefore be considered conventional. A common approach followed for design and assessment purposes is to idealize the cyclic envelope with a bilinear curve, which allows to synthesize some aspects of the deformational behaviour in a fashion which is familiar to most designers (Figure 4). The envelope curve or skeleton curve is actually dependent on the type of loading history, (e.g. monotonic or cyclic, quasi-static or dynamic, and sequence of cycles) especially when shear failures are involved, and this constitutes a problem for the correlation of different experimental procedures.

Flexural response

In case of a pure flexural response, i. e. of a potential rocking response, very large displacements can be theoretically obtained without significant loss in strength, especially when the mean axial load is low compared to the compressive strength of masonry. In such cases a very moderate hysteretic energy dissipation and an almost nonlinear elastic behaviour is reported, with negligible

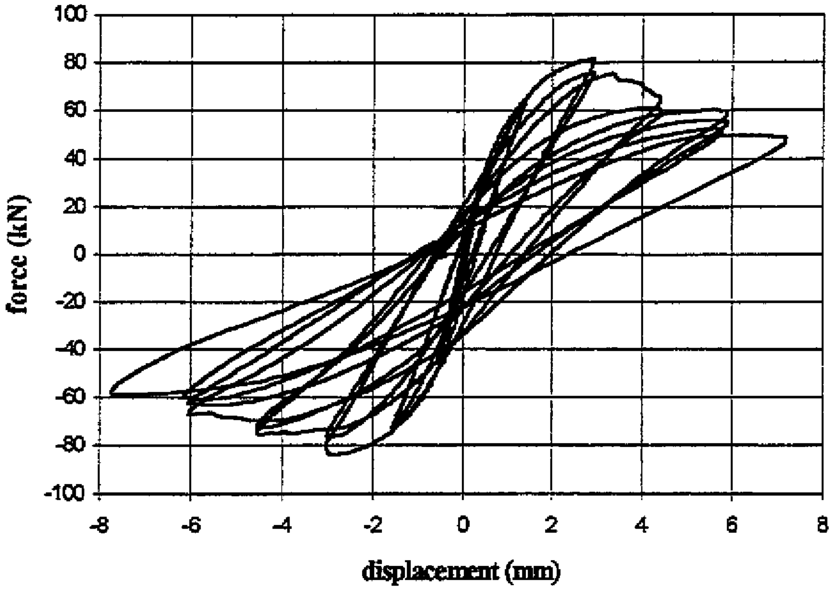


Figure 5. Example of behaviour with diagonal shear cracking (2), H/D = 1.35, p= 0.6 MPa.

strength degradation in subsequent cycles at the same peak displacement, due to moderate crushing at the compressed. If no other failure mechanism occurs, the displacements which can be attained in a rocking response can be limited only by a decrease in strength due to second order effects associated to overturning. These limits can be as high as 10% of the height of the wall and are of no practical use, since other collapse events will always take place before reaching this drift limit. As a consequence neither an ultimate displacement nor a secant stiffness to collapse can be defined for this response mode. Both values will depend on the demand from the seismic input or from the limit imposed by other failure modes.

Diagonal shear cracking response

A typical force-displacement curve in case of a response dominated by shear cracking is shown in Figure 5 (2).

The pre-cracking behaviour is characterized by moderate hysteresis, and by negligible strength and stiffness degradation. Initiation of the first visible diagonal cracks corresponds to a load which in the considered experiments was beyond 90 % of the peak load. Post-peak response is characterized by higher hysteretical dissipation but by rather fast strength and stiffness degradation.

Equivalent bilinear curves were defined for the experimental shear tests reported in (2,16), according to Figure 4, where the ultimate displacement was defined as the displacement corresponding to a strength degradation of 20% below the ultimate strength V_u . Since the number of repetition of the cycles at peak displacement was not the same for all the tests, the reference envelope was evaluated for each wall considering the peaks at the first cycle for a given displacement level (first cycle envelope). The results were examined in terms of 'yield' displacement δ_e, and consequently of initial stiffness $K_{el}= V_u/ \delta_e$, , of ultimate displacement δ_u, , and ductility δ_u / δ_e, . It was found that the evaluation of an equivalent yield point presents a large scatter, and its variation was not apparently correlated in a consistent way with, say, axial load or slenderness, although physical considerations call for an increase in stiffness with increasing mean axial load and a decrease in stiffness with increasing slenderness. The same could be said for the calculated drift (yield displacement divided by wall height) at yield.

On the other hand, the drift at ultimate conditions was found to be extraordinarily uniform, with a mean value of 5.3 x 10^{-3} and a coefficient of variation of 10%. The scatter in the displacement at yield was obviously reflected in the scatter of the measure of ductility (mean value ~ 4.5, c.o.v. 46.3%), ductility being defined as the ratio between the two values discussed above. A possible explanation for this result is that, being the initial elastic stiffness associated to a branch where crack initiation and propagation is dominating on the tensile side, and mortar compaction on the compressive side, high scatter is reported in initial stiffness evaluation. On the other hand, being the ultimate drift associated to friction and interlocking mechanisms in stabilized cracks which increase in size (opening), the scatter is reduced. Ultimate drift appears therefore as a more reliable parameter than ductility. When ultimate conditions are considered, strength and ultimate displacement are the dominating parameters, and the determination of the initial elastic stiffness plays a little role.

Sliding shear response

When sliding on horizontal bedjoints occurs, a very stable mechanism is involved, since very high displacements are possible without the a loss of integrity of the wall. Damage and dissipation are concentrated in a bedjoint, and as long as a vertical load is present high energy dissipation is possible, as shown by cyclic tests on bedjoints (3). Dynamic tests on walls (17) have shown that rocking and sliding typically take place together, when low axial loads are present. An ultimate displacement limit for shear sliding has, as in the case of rocking, no real meaning since it would be so high that the occurrence of other failures would in practice determine the real displacement limit.

Energy dissipation capacity of URM walls

Equivalent Damping

A structural element can dissipate energy in various forms, such as viscosity, friction or hysteresis. While a viscous damping is present even at low level of excitation, and it is often assumed as 5% of the critical damping even if experimental measures usually indicates lower values, friction and hysteresis are usually related to some damage process involving permanent deformations. The amount of dissipated energy is essentially proportional to the area of the hysteretic loops shown when the imposed displacements are large enough to induce a significant non linear response. With a few simplifying hypotheses (essentially assuming constant velocity and resonant response) it is possible to correlate the amount of dissipated energy to an equivalent viscous damping ratio, which multiplies the velocity term in the equation expressing a linear dynamic response. Such equivalent damping ratio, given a single load-displacement cycle, can be expressed as a function of the dissipated energy W_d and the stored elastic energy at peak displacement W_e Shibata and