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TOWARDS THE DEVELOPMENT OF A SEISMIC ASSESSMENT FRAMEWORK FOR URM STRUCTURES

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Abstract

Load-bearing unreinforced masonry (URM) structures present several special characteristics that set them apart from common engineered construction, such as weak diaphragm action, spatial distribution of mass (massive load-bearing walls), and inherent brittleness and negligible tensile strength. For this type of structures, a complete methodology is developed intended to enable seismic protection through assessment and retrofit particularly aimed at historical and heritage structures. The methodology includes the attributes of a performance-based prenormative framework, including systematic procedures for setting assessment response indices and acceptance criteria based on deformation demand and supply measures. The required methods of analysis and allowable simplifications and confidence limits are detailed, along with acceptable interventions that are compatible with the material physical characteristics and retrofit conventions when the URM structure also carries some cultural significance as is frequent the case.

The proposal presented concerns prenormative provisions for management of the seismic risk posed by URM structures, including illustration of methods and handling of uncertainty in all steps of the process. Performance of URM structures in past earthquakes that motivate the need for better understanding certain response aspects, particularly out of plane rocking and ways to limit that through timber lacing and other methods are discussed in order to highlight the vulnerabilities of this class of structures in severe seismic events and quick ways to eliminate critical pathologies against out-of-plane action.

Keywords: unreinforced masonry, out-of-plane action, lateral drift, performance criteria, seismic assessment

1. Introduction

This paper reviews the salient points of a practical seismic assessment framework which was developed in order to address a pressing need in the field of management of the seismic risk of traditional and historical masonry structures representative of the Balkan region ([1], [1], [3]). Here buildings typically comprise stone or clay or adobe masonry, low-rise construction with timber diaphragms that do not provide noticeable diaphragm action. Well-constructed buildings in this category may be equipped with a timber ring-beam known as "γρεπίδα" – (the term comes from the Ancient Greek "Κρηπίς", synonymous with Zophorous or frise, in the context cited in Vi-truvi-ous's De Architectur III, 5, §10. This element secures partial diaphragmatic action at the top of the edifice, which seemed to be a very effective means of seismic protection; a variation may include timber lacing as shear reinforcement in masonry piers [4] or masonry-infilled timber frames in the upper floors [3].

Seismic assessment of this class of structures is hampered by several difficulties that encompass both sides of the design equation, i.e., the estimation of seismic demands as well as the definition of pertinent acceptance criteria. In terms of demand estimation, lack of robust diaphragm action and the ensuing prevalence of out-of-plane bending of masonry walls and piers limit the applicability of simplified idealizations that could take advantage of nonlinear frame-type analysis such as would be possible with special dedicated software such as tremuri [5]. To capture these aspects of the response it is required that finite element idealizations using shell or solid elements is required, and this, with the current state of the art, can only be performed in a robust manner if material brittleness and tensile fracture – characteristic features of masonry, both be neglected. So, although frequently cited as an obvious option, nonlinear analysis of unreinforced masonry buildings is not necessarily possible. On the other hand, partly owing to the great variety of materials available, but also the contributing influence of boundary conditions during testing, the availability of acceptance criteria (e.g. shear strength at the various performance limits, the stiffness, the deformation capacity to in and out-of-plane loading that could be associated to the "damage limitation", repairable" and "collapse prevention" limit states) can still be established only qualitatively, at best.

Seismic assessment of a masonry structure must be based on estimations of seismic demand that would be obtained through analysis following prescribed rules that may be executed in an unambiguous manner by a trained professional. In order to reach a practicable assessment conclusion, the demand estimates should be compared against established acceptance criteria so that performance evaluation may be feasible in a manner that is compatible to modern day earthquake engineering procedures. In this front a Code of assessment that prescribes these procedures for the class of structures envisioned, is an urgent practical need. The present paper attempts to respond to that open need by laying out a framework that encompasses performance-based assessment of older traditional or heritage masonry structures with no diaphragms, as detailed in the forthcoming.

Keeping in mind that it is impossible to obtain convergence of FE analysis when the material is brittle in tension and there is no reinforcement, elastic analysis (e.g. EN 1998-1:2004 [6]) becomes a point of reference in order to determine the disposition of stress and deformation. It is noted here that estimation of the out-of-plane response components spatial modelling with elements that are equipped with translational and rotational degrees of freedom (i.e. shell rather than plane-stress elements) for which it may be difficult or even impossible to obtain convergence beyond the stage of tension cracking. Even if nonlinear solid / shell FE analysis would become possible despite the brittleness of the material in the absence of the stabilizing influence of reinforcement, its widespread use is not advisable since the *knowledge level* for buildings of this class is not compatible with the level of sophistication required in terms of input information for simulation of load-bearing unreinforced masonry buildings. (There is great uncertainty owing to material variability and case-by-case specificity and the geometric constraints and dimensions resulting from ageing and non-industrialized construction methods; EN 1998-3:2005 §3.3, [7]).

2. Modelling

It is possible under conditions to use, apart from Finite Elements, other modelling techniques that may include macroelements, equivalent frame with notional linear elements representing piers and spandrels, and strut and tie approaches. The primary conditions are:

In modelling with F.E. except in such cases where solid elements may be called for, the basic approach should rely on shell element modelling (i.e. translational and rotational d.o.f.), so as to be able to model both in-plane and out-of-plane action. In the general model it is possible to use discretization of some piers using lineal elements if: a) the horizontal cross section of the pier is less than $0.3m^2$, b) the ratio of the longest to the shortest dimension of the piers' cross section is ≤2, and c) the height to length ratio is >2.

Macroelements cannot be included in spatial models – therefore, these elements can only be used when the out-of-plane action is negligible, that is, when the building has rigid diaphragms. Macroelements ought to be interconnected through contact elements/springs.

Modelling a structure as an equivalent frame is possible only if the following restrictions apply simultaneously: a) adequate diaphragm action for the floors and the roof has been secured through proper measures, b) the arrangement of openings is such that the dimensions of adjacent piers may be considered approximately equal (this refers to the horizontal dimension or length of the piers' cross section), from the level of the foundation to the crest of the wall, and c) the ratio of height to the length of the pier (in a single floor) exceeds the limit of 2.0. Use of strut-and-tie modelling is allowed only for such parts of the structure where the disposition and flow of forces is understood with confidence.

3. Analysis methods

In principle, the methods that may be used during analysis are those proposed according with EN 1998-1 namely: (i) Elastic (equivalent) static analysis, (ii) Modal analysis using response spectra for hazard definition (also known as elastic dynamic analysis), (iii) Non-linear static analysis, (iv) Non-linear dynamic analysis (time history). Essential issues that need be addressed during application of the methods to historical structures are discussed below.

3.1. Elastic (equivalent) static analysis

This is a basic point of reference in seismic assessment and rehabilitation. Analysis using equivalent static loads is conducted for calculation of internal forces and element deformations. Two alternative distributions of seismic lateral loads heightwise may be considered: (a) inverted triangular distribution, (b) uniform distribution along the building height and extending over the breadth of the side that is orthogonal to the earthquake (i.e. loads cannot be acting in a plane but they must be applied pointwise on all the nodes of the walls that are normal to the earthquake action). Note here that in the absence of rigid diaphragms, the uniform distribution of the seismic loads is more realistic for structures with a distributed mass as is the case of unreinforced masonry (URM) buildings. This type of analysis may be applied in buildings whose response in each principal direction of the plan may be assumed to occur in the fundamental mode – i.e., it is not influenced significantly by higher mode contributions.

The above requirement is valid if the fundamental period of vibration of the building, T_1, in each principal direction of the plan satisfies the following condition:

$$T_1 \leq \left\{4T_C; 2,0\ \mathrm{sec}\right\} \tag{1}$$

where, T_C is the period at the end of the constant acceleration range in the acceleration design spectrum (see §3.2.2.2 of EN1998-1). These conditions are generally valid in historical URM buildings; while the load bearing walls in the two main plan directions are nearly orthogonal to each other, and in addition the following preconditions hold: 1) piers are continuous along the building height, 2) horizontal systems (floors and roof) are relatively stiff in their plane of action and adequately connected in the perimeter walls so as to deliver the inertia forces to the vertical load bearing system through rigid diaphragm action; and 3) adjacent floors supported on a common URM wall are located in the same height.

3.1.1. Determination of force and deformation demands

To resolve internal action and deformation demands in order to be used in assessment, the total seismic lateral load is estimated from Eq. (2):

$$V = C_1 C_m S_e(T_1) \cdot m \tag{2}$$

Where for URM buildings the fundamental period T_1 may be approximated from the equation:

$$T_1 = 0.05 H^{3/4} \tag{3}$$

H is the building height above ground, and $C_1 = \Delta_{in}/\Delta_{el}$ is the ratio of maximum inelastic displacement of the building divided by the corresponding displacement obtained from elastic analysis. Coefficient C_1 may be estimated using the following expressions:

$$C_1 = 1 \text{ for } T_1 \geq T_C \tag{4}$$

$$C_1 = \frac{1}{q_u} \cdot \left(1 + (q_u - 1) \cdot \frac{T_C}{T_1} \right) \text{ for } T_1 \leq T_C \tag{5}$$

Ratio $q_u = V_{el}/V_y$ is the nominal behavior factor; this is defined by the ratio of the estimated elastic base shear divided by the notional base shear yield strength of the building, see Fig. 1.
C_m is the mass participation factor, taken equal to 1.0 for one-story and two-storey buildings, and equal to 0.8 for buildings with three storeys or more.
$S_e(T_1)$ is the spectral total acceleration that corresponds to the fundamental period, T_1, and
m is the building mass (estimated by dividing the building weight by the acceleration of gravity, g).

If the fundamental translational periods of the structure in the two principal directions of the building are substantially different, then $S_e(T_1)$ is obtained from the design spectrum according with the prevailing period.

3.2. Modal Spectral Analysis (elastic dynamic)

For application of the method the contribution of all significant modes participating in total response are considered. These requirements are considered satisfied if any of the following is demonstrated: a) the sum of the participating masses of the modes considered in the analysis account for more than 75% of the total building mass, b) all modes having a modal mass that exceeds 5% of the total mass are considered in the analysis.

3.3. Non-linear Static Analysis

The seismic demand, as compared to the available capacity, is estimated directly in terms of displacement at the crest of load bearing walls, which corresponds to the target displacement for the seismic hazard scenario established for the given site. In buildings with undeformable (rigid) diaphragms the so-called “control node” (i.e. the node whose displacement is mapped to the target displacement) is usually taken at the centroid of the top slab.

In buildings with deformable diaphragms, a key ingredient of the assessment procedure is the normalized shape of lateral response of the building, which may be the fundamental mode of lateral translation or any modification thereof to account for possible damage localization. This shape is used in order to estimate local deformation demands with the target displacement; here the “control node” is the point where the shape function assumes the value of 1 (i.e. it is the generalized coordinate as per the definition in [9]). With no loss of generality and for mere convenience it is possible to use the most displaced node of the URM as the point whose displacement is used to normalize the assumed response shape, and therefore that point would serve as the control node in assessment. In buildings lacking diaphragm action the most displaced node usually occurs at the crest of the building, at midspan of a long wall or over a spandrel as it is shown in Fig. 2. If gables exist it is advisable to exclude them from the definition of the control node as the local amplification of their cantilevering action may, if used to normalize the lateral displacements for definition of the response shape, introduce significant errors. The target displacement is the elastic displacement demand for an equivalent single degree of freedom system having a period equal to the estimated period T_1 of the building.

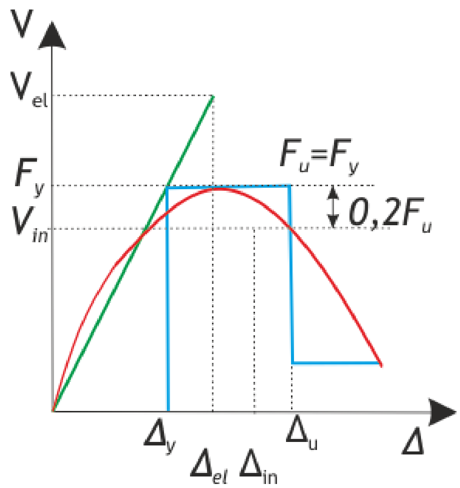


Fig. 1 - Definition of behavior factor, and the resistance curve of a wall element (the blue line)

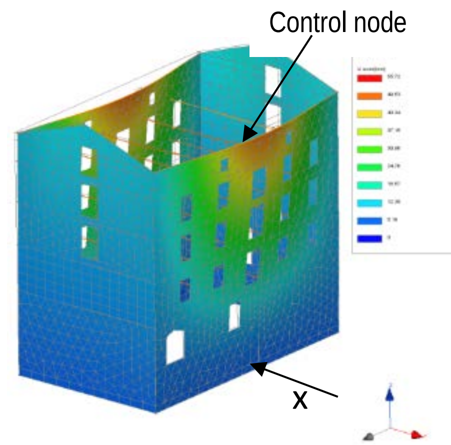


Fig. 2 - Deformation of a building with deformable diaphragms for seismic action parallel the axis x

3.4. Nonlinear dynamic analysis (time-history)

In light of the fact that such an analysis in structures that lack diaphragm action may be excessively complex this analysis type is not recommended but for particularly important monumental structures and only if chosen by the engineer (not compulsory).

4. Load - Deformation Resistance curve of the members

a) The mechanic behavior of a URM pier or a spandrel may be described in the form of a resistance curve where the internal force measure “*F*” is related to the deformation or relative displacement “*Δ*” (red line in Fig. 1). The kind, direction etc of the internal force *F* is selected so that it may characterize the primary component of the action that the excitation is causing in the member. Deformation *Δ* is compatible with the internal force measure so that the product of the two may express the strain energy of the element (or critical region or connection modelled – so drift if the action is flexural, shear distortion if the action is shear).

So long as experimental data are available, it is considered that the mechanical behavior is described by the reduced envelope of *F* in the end of a complete reversed cycle ±*Δ*, up until the loss of the element strength by 20%. The assumed inelasticity in the response curve is consistent with all the relevant standards and codes related to assessment of masonry. For example, Appendix C of EN1998-3 [7] which refers to seismic assessment of masonry prescribes the limit states of masonry (damage limitation and collapse prevention) in terms of drift, whereas the drift capacity is associated with the type of action (in-plane, out-of-plane, shear or flexure dominant). The same approach is also followed in the draft documents by many regulating organizations e.g. Italian Code [8] which go as far as even defining nonlieanr moment-rotation envelopes for URM piers under reversed cycic lateral load. At the same time, concerns have been raised as to the source of non-linearity in what is considered brittle mode of construction. Yet, there is persistent experimental evidence that masonry does not collapse immediately upon cracking. Databases assembled from experiments that are published in the literature clearly support that masonry piers and spandrels can exceed by a significant margin (more than two to three fold) the cracking drift limit which is estimated to be around 0.2%. There are several mechanisms that may be responsible for this post-cracking resilience, suchas friction between wythes, the presence of timber lacing in traditional masonry or of iron clamps in industrial masonry buildings, as well as kinematic constraints that prevent the length change of masonry which precedes its catastrophic collapse; all these of course also depending on the manner of construction.
b) Absent any counterevidence in the experimental data it is assumed that failure of masonry occurs after the exhaustion of its available ductility capacity (particularly relevant for infilled timber frame masonry or timber

laced masonry), or after attainment of notional yielding (for common unreinforced masonry) of the piers and/or spandrels in the structure.

4.1. Notional Elastic Branch up to Phenomenological "Yielding"

a) The simple rules for the calculation of the seismic response with pseudo-elastic methods (e.g. derived inelastic spectra and use of behavior factors, equal displacement rule between elastic and inelastic displacement and their extension, etc.) presume the existence of a bilinear envelope of the force-displacement response curve F-Δ of the building as a whole (e.g. Base shear – target node displacement envelope), with the notional elastic branch reaching yielding. The approximation of the actual F-Δ curve through a multilinear diagram is generally sufficient for practical needs (design or assessment). The first linear branch extends from zero to the effective "yield" point of the member, beyond which the resistance curve F-Δ may be approximated by a horizontal (plateau) branch, (Fig. 1).

The rotation θ_y that corresponds to the limit of "yielding" in URM elements is the mean average angle forming between the chord of the deformed element and the tangent to the deflected shape at the onset of cracking.

(i) This value will be taken equal to 0.0015 with a standard deviation of 35% for in-plane flexure and shear.

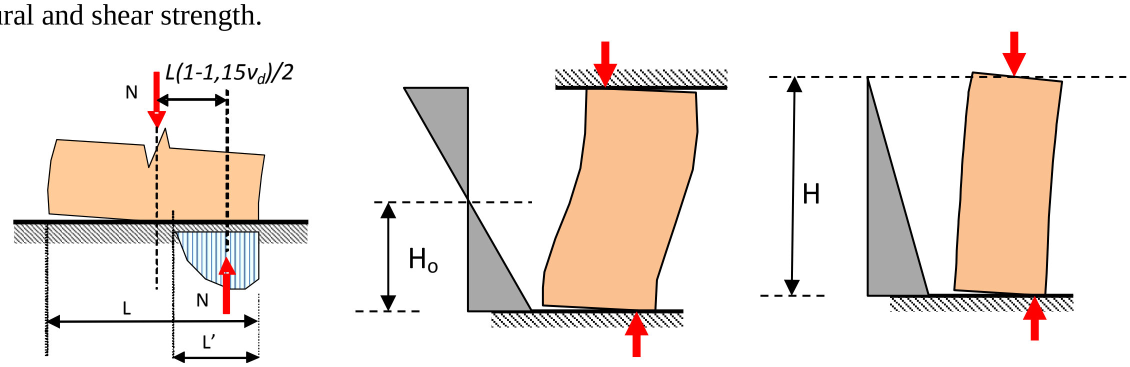
(ii) For out of plane deformation the rotation at "yielding" of the member from its chord, θ_y, is taken equal to 0.0020 with a standard deviation of 35%.

When piers deflect in their plane of action, the rotations that develop are owing to a combination of flexural curvature and shear deformation. The "yielding point" may be associated with the exceedence of either of the two strength mechanisms (the least resistance controls the limit of "yielding"). These strength terms are defined below.

4.2. Definition of Yield Strength, F_y

Depending on the mode of failure the "yield" strength may be approximated as follows:

(α). *Development of flexural strength of the masonry pier, in the critical cross section.* In the absence of reinforcement, the flexural behaviour refers to the rotation of the walls about the cross section at the base (see Fig. 3). To calculate the flexural yield strength of any individual pier, the length of compression zone a prerequisite step is to determine whether the pier is located in the *active* or the *inactive* regions of the building plan. These are the parts of the building plan where normal compression or normal tension develops, respectively, as a result of the combination of the overbearing gravity loads and the overturning moments generated for the entire building by the seismic action. Wall piers located in inactive regions are assumed to possess no flexural and shear strength.



(a) (b)

Fig. 3 - Bending of a pier in its plane of action. a) Definition of Internal Moment and (b) Definition of the Effective Shear span H_o with reference to the moment diagram

Expressed as a shear force at the onset of flexural yielding, flexural strength is given by Eq. (6):

$$F_{y,fl} = \frac{L\cdot(1-1{,}15\cdot\nu_d)}{2H_o}\cdot N \tag{6}$$

Where N is the axial load of the pier, and $\nu_d = N/(L\cdot t\cdot f_{md})$ is the normalized axial load of the pier (Fig. 3(a)).

(***b***). *Development of Shear Strength of the masonry pier which may be owing to (Fig. 4),* either

(b.1) Exceedence of URM's tensile strength due to shear failure (diagonal cracking in the direction of principal compressive stresses, orthogonal to the direction of principal tension, see Fig. 4(a)). Shear strength f_{vd} is determined from the mean tensile strength of masonry and the overbearing axial load from the relationship:

$$\sigma_1 = -f_{wtd} = \frac{\nu_d f_d}{2} - \sqrt{f_{vd,t}^2 + \left(\frac{\nu_d f_d}{2}\right)^2} \tag{7}$$

where:

$$f_{vd,t} = \left[\left(-f_{wtd} - \frac{\nu_d f_d}{2}\right)^2 - \left(\frac{\nu_d f_d}{2}\right)^2\right]^2 = \sqrt{f_{wtd}\cdot\left(f_{wtd} + \nu_d\cdot f_d\right)} \tag{8}$$

The minus sign refers to tension and the plus sign to compression,

$f_{vd,t}$: is the shear strength of masonry associated with diagonal tension cracking

f_{wtd}: is the design strength of masonry to direct tension.

or,

(b.2) Failure by sliding along the horizontal joints of the masonry (Fig. 4(b)). The shear strength against sliding is estimated from the cohesion and friction that develops under the presence of the overbearing compressive loading, as:

$$f_{vd,s} = f_{vo} + \mu\cdot(\nu_d f_d) \tag{9}$$

where, term $f_{vd,s}$ is the shear strength of masonry which is associated with sliding along the frictional contact surface

f_{vo}: is the cohesive strength that develops at the contact interface between the mortar joint and the masonry block

μ: is the coefficient of friction along the sliding surface; for lack of detailed information, it may be taken equal to 0,4.

$\nu_d f_d$: is the overbearing compressive stress in the plane of sliding.

Again, expressed as a shear force associated with either mode b.1) or b.2) the shear strength of a URM wall is,

$$F_{y,v} = f_{vd}\cdot L'\cdot t \tag{10}$$

Here L' is the length of the compression zone of the pier wall cross section. The limiting value $f_{vd}=min\{f_{vd,t}\ f_{vd,s}\}$ is the failure shear strength of masonry (MPa), and it cannot exceed the shear strength of the individual masonry blocks, i.e, $f_{vd}\leq 0{,}065 f_m$, where f_m is the compressive strength of the masonry (f_m could be taken as the strength of the homogenized masonry wall as per EN1998:2005, or instead it would be restricted to take on values of the block unit, f_b as per EN1996:2005. Both options are kept open here, as there is clearly a lack of experimental data to favor one over the other).

The contribution of timber laces if they exist is taken into consideration in calculating the shear strength of piers as follows: Each timber lace that is intersected by an idealized crack that is inclined by 45^o in the plane of masonry contributes to masonry's shear strength, which was estimated by Eq. (9) through the addition of term V_{tier} (Fig. 5).

$$F_{y,tier} = u_{b,tier}\cdot p_{tier}\cdot L_{b,tier} \tag{11}$$

Where, $u_{b,tier}$ is the specific cohesion strength (MPa) that develops at the interface between the timber lace and the URM,
p_{tier}: is the contact perimeter between the timber lace and the URM
and
$L_{b,tier}$: is the minimum contact length between the timber lace and the URM pier counted to the left or to the right from the intersecting crack plane.

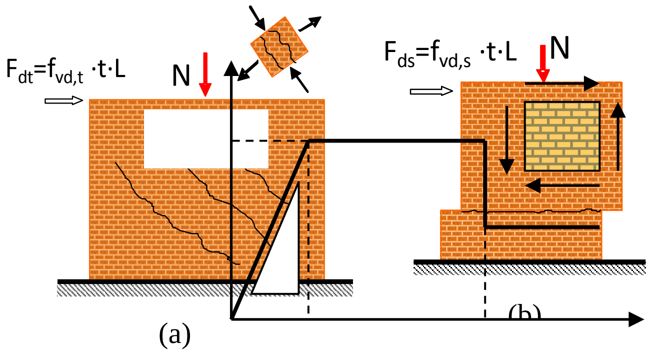


Fig. 4 - Shear failure, a) diagonal cracking, b) sliding

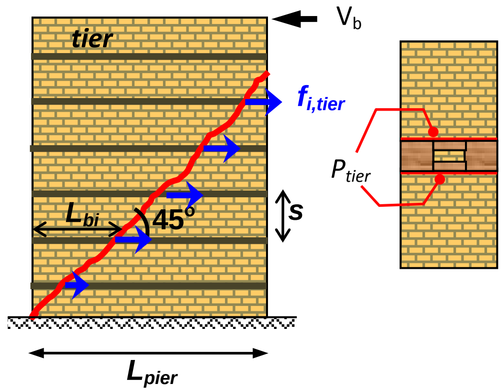


Fig. 5 - Contribution of laces to shear strength

5. Strength of walls to out-of-plane action

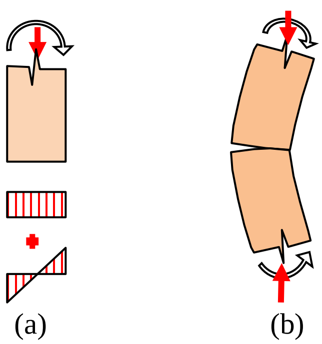
Wall piers loaded normal to their plane of action under a combination of horizontal pressure and axial forces due to overbearing loads, are checked to out of plane bending. In this case, the flexural strength per unit length of the pier; 1.0m wide strips are considered, extending both in the horizontal and the vertical direction (i.e. parallel and orthogonal to the direction of the masonry beds). The flexural strength of the 1.0m wide strip is calculated from Eqs. 12 and 13 for bending about the horizontal and the vertical axes, respectively. These expressions are used with reference to the concept of active and inactive building plan regions (i.e., piers located in inactive regions are completely neglected and are considered to not be contributing to the building's strength). A critical parameter required in the plan of each level studied is the normalized axial load ratio ν_d resulting from analysis of the building for the overturning and load bearing action.

Strength calculations are based on the classical approach of superposition of stress blocks resulting from the axial load and the flexural moment Fig. 6(a) and the requirement of non-exceedence of the tensile strength of masonry, f_{xk}. Thus, per unit length and height of the wall, the flexural strengths are:

$$M_{\max,1} = \left(f_{xk,1} + \nu_d f_d\right)\cdot t^2/6 \quad \text{and} \quad M_{\max,2} = f_{xk,2} t^2/6 \tag{12}$$

where, $f_{xk,1}$ is the characteristic flexural strength of masonry for bending parallel to the bed joints (vertical unit-width strips), and

$f_{xk,2}$ is the characteristic flexural strength of masonry for bending in direction orthogonal to the bed joints (horizontal unit-width strips). Parameter ν_d is the normalized axial load at the critical section.



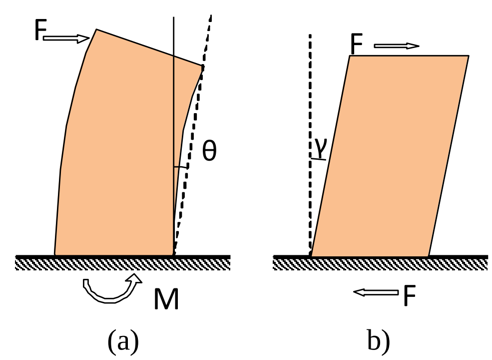


Fig. 6 - Flexural strength calculation for out of plane bending. (a) Stress superposition at yielding, (b) Flexural Strength calculation near the ultimate

Fig. 7 - Wall under in plane force

Near the ultimate, the flexural strength is reduced due to the crack opening as shown in Fig. 6(b) (see also [10] and [12]). The residual flexural strength is owing to the axial load, acting over the internal lever arm that is created between the centroid of the cross section (*t/2*) and the centroid of the compression zone (≈0.15*t* if the compression zone is taken equal to 0.3*t*):

$$M_u = N \cdot (0.5 - 0.15)t = 0.35N \cdot t \tag{13}$$

6. Deformation capacity

The nominal deformation capacity of URM walls, δ_u, is estimated according to the plane of their action.

6.1. Walls loaded parallel to their plane (in-plane-action)

(i-1). The deformation capacity of a URM wall controlled by flexure (Fig. 7(a)) may be expressed in terms of relative drift ratio and is taken equal to $0{,}008 \cdot H_0/L$ for primary seismic walls and $0{,}012 \cdot H_0/L$ for secondary walls (not contributing to the seismic resistance of the building by more than 15%, where L is the pier wall length and H_0 is the distance from the critical cross section where flexural strength is attained to the point of zero moment (i.e., the shear span) (see Fig. 3(b)).

Relative drift ratio is the rotation from its initial position, of the chord that connects any two points in a vertical or horizontal line in the wall plane (see Fig. 8(a) and (b), respectively). If the displacement capacity is measured experimentally, then the relative drift capacity is estimated after dividing the relative displacement capacity with the distance H_0.

(i-2). The deformation capacity of a URM wall controlled by shear (Fig. 7), may be expressed in terms of relative drift ratio and may be taken equal to 0,004 for primary seismic walls and 0,006 for secondary ones.

6.2. Walls loaded normal to their plane of action

Based on the experimental results of Griffith et al. [11] the walls' deformation capacity in out of plane bending depends to a large extent on the antagonism between a rocking motion and out of plane flexural curvature which generates second order membrane forces that provide resilience to the structure against collapse – to predict the membrane actions one would have to consider the actual dimensions and stiffness of the supports of the out of plane bending elements (large deformation geometry). It is recommended to estimate the drift at wall failure as the minimum of values:

$$\theta_{u,1} = 0.003 \cdot \frac{H_0}{t} \quad \text{and,} \qquad \theta_{u,2} = \theta_{R,u} \cdot \left(1 - \frac{F_y}{F_{Rd}}\right) \tag{14}$$

Note that the rocking rotation of the wall about its axis, required to cause instability and overthrow may be calculated from Eq. (15) with reference to Fig. 9:

$$\theta_{R,u} = t / H_0 \tag{15}$$

where t is the wall thickness and H_o is the distance of maximum translation to the axis about which rocking occurs.

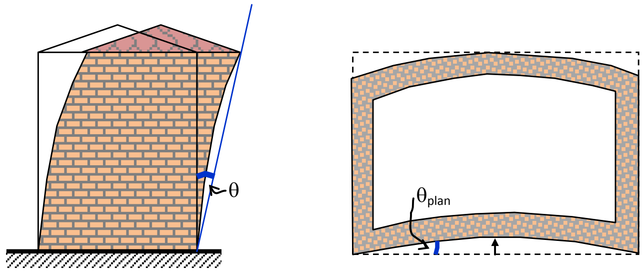


Fig. 8 - Definition of drifts

7. Evaluation criteria

The acceptance criteria are determined with reference to a selected performance level.

Checking against the performance criteria (design inequality) in terms of internal forces and deformations is carried out for individual structural member provided that the member has been previously characterized as "primary" or "secondary".

For performance level A (Damage Limitation, DL) acceptance criteria are expressed in terms of elastic forces / deformations. For levels B and C (Significant Damage, SD, and Collapse Prevention, CP), performance checks for brittle members / and or failure modes are done in terms of forces, whereas checks for nominally ductile members the checks may be expressed preferably in terms of deformation (Fig. 10).

7.1. Acceptance criteria for performance limit A: Damage Limitation (DL)

For level A the general safety inequality is checked for primary and secondary components using:

- S_d : internal force demands obtained from the elastic analysis using a knowledge factor γ_{Sd} according with § 4.5.1 of EN 1998-3 (2005).
- R_d : resistance design values calculated using the material safety coefficients γ_m and representative values for the materials as defined in section § 4.5.3 of EN 1998-3 (2005). If linear elastic analysis is conducted the criteria are expressed in terms of base-shear demand (S_d) and supply (R_d) in the direction of seismic action.

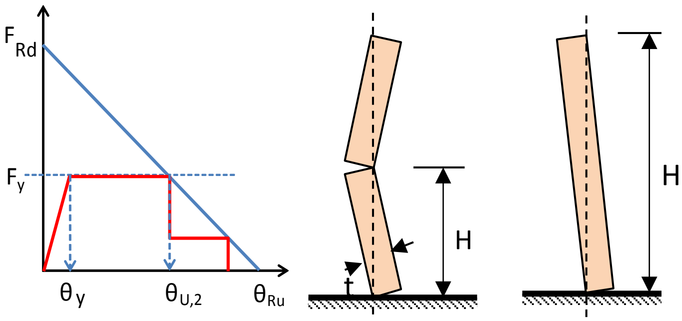


Fig. 9 - Definition of limiting rotation $\theta_{R,u}$

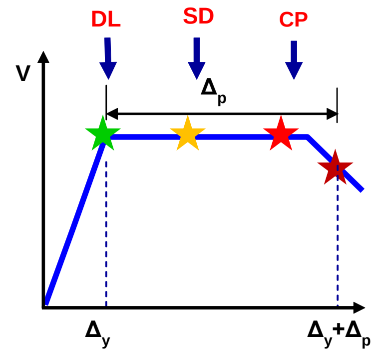


Fig. 10- Definition of performance levels

7.2. Acceptance criteria for performance level B: Significant Damage (SD)

Deformation capacity of a flexure-controlled pier is expressed in terms of relative drift ratio or relative displacement and shall be taken equal to 3/4 of the respective nominal values of δ_u, and θ_u, that were determined in the preceding Paragraph 6.i-1 and Eqs. 14-15 for out-of-plane flexural action. Similarly, the deformation capacity of a shear-controlled pier is taken equal to 3/4 of the respective values determined in Paragraph 6.i-2.

7.3. Acceptance criteria for performance level C: Collapse Prevention (CP)

For level C all the elements of the load bearing system may develop significant inelastic deformations – however, the primary elements ought to provide a significant margin of safety from that level up to the limit of their available deformation capacity. At this performance limit it is not allowed to exceed the available deformation capacity of the primary and secondary vertical components, whereas limited excesses are generally permitted in what concerns horizontal secondary components.

The deformation capacity of a flexure-controlled pier may be expressed in terms of relative drift ratio or re-lative displacement and shall be taken equal to 4/3 of the respective nominal values of δ_u, and θ_u, that were determined in the preceding Section 6.i-1 and Eqs. 14-15 for out-of-plane flexural action. Similarly, the deformation capacity of a shear-controlled pier is taken equal to 4/3 of the respective values determined in Section 6.i-2.

8. Retrofit Measures – Conclusive Remarks

If performance criteria are exceeded or if it is desired to improve the performance limit state (reduce the level of damage) then the following measures are possible: (a) To reduce the magnitude of seismic displacement at the control node by reducing the period of the structure (moving to the left side of the response spectrum) and/or (b) To improve the distribution of deformation demands by aiming for a more uniform distribution, eliminating the tendency for localization. Of the two options the former is the least effective as URM structures are already relatively stiff by air default since the walls are thick and the overbearing weights increase the stability of the system. Actually the relative spectral displacement is seldom very large in the range of the spectrum where the fundamental translational mode of vibration lies. Damage in URM structures occurs because the demands are localized. It is therefore much more effective to aim for improved distribution: based on previous studies this may be achieved by targeted modifications in the shape of lateral translation. Here tendencies for localization that are particularly responsible for the observed damage are, (a) the out-of-plane flexural response of walls orthogonal to the earthquake action, and opening at the corners between orthogonal walls, (b) shear failure of captive piers and spandrels, (c) uplift due to rocking of walls due to reduced / small overbearing pressure that renders that part of wall either ineffective or susceptible to tension failure.

Response aspects (a) and (c) are both owing to the lack of diaphragm action in URM construction which cannot support any form of kinematic constraint against relative displacement between points on the same floor of the structure. In retrofitting the structure every effort should be directed towards improving the diaphragm action at the crest and floor levels. A common retrofit practice which is not excessively invasive, is to install a perimeter tie-beam at the crest level exactly underneath the roof trusses. In common two-storey URM buildings this measure improves the response substantially as compared to the state of the structure without tying. The same is achieved by the stiffening of the floor diaphragm (e.g. by the addition of a bottom flange in timber floors to convert them to cell sections) and through better connection of the horizontal diaphragm to the perimeter walls using steel rods and plates to anchor them by bearing action from the outside.

Shear failure (case (b) above) can be eliminated through local interventions such as (i) deep repointing, (ii) installation of reinforcing elements in the beds (e.g. bars) and (iii) in the form of externally bonded stips in directions parallel to the shear force. Another option is to create confining elements on the perimeter of openings to reduce stress concentrations that lead to localized cracking of these shear and rocking critical elements. However, this may even be eliminated as a concern by reduction of the demands as described above, i.e. through enhancement of the diaphragm action.

The retrofitting effectiveness resulting from the introduction of enhancement of diaphragm action is shown in Fig. 11 for the building of Fig. 2.

The above present the salient points of a performance based assessment framework which may easily be evolved in normative format due to its versatile performance-based assessment procedures designed to support practical evaluation of the seismic hazard and possible rehabilitation of traditional and historical masonry buildings and monuments. Several important issues associated with background analysis of the URM structure, which is needed in order to assess seismic demands and capacities associated with reference performance limits, are resolved in a general manner that complies to established assessment codes used in current earthquake engineering for the mitigation and management of the seismic risk of existing structures, while recognizing and accounting for the limitations posed by the brittleness of the material and the structural system of older URM buildings.

(a) (b)

Fig. 11- Displacements of the building in Fig. 2, after addition of a) one and b) two horizontal diaphragms (Fig. 2 plots the building response with no diaphragm constraints)

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