EUROPEAN PROCEDURES FOR SEISMIC ASSESSMENT OF URM STRUCTURES

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ABSTRACT

The paper reviews two performance-based frameworks that may be used as the basis for the development of a seismic assessment code for load-bearing unreinforced masonry (URM) structures; these are the recently drafted EN 1998-3 (2020) provisions [1] and the Hellenic draft code for seismic assessment of load-bearing URM structures [2]. Important URM characteristics that set them apart from common engineered construction are: the frequent absence of diaphragm action, the excessive distributed mass in the massive load-bearing walls, and the inherent brittleness and negligible tensile strength of URM. For seismic evaluation of URM structures, both guidelines define performance limits for assessment of the individual elements, and acceptance criteria based on deformation demand and supply measures at selected levels of performance. But the required methods of analysis and allowable simplifications and confidence limits differ between the two frameworks thereby influencing the demand and the calculation of member capacities. Mechanistic models that form the background to the basic acceptance criteria associated with reference performance limits are considered, but implementation differs in the two approaches, resulting from the idealizations allowed during modelling. The paper summarizes the two draft provisions with regards to member idealization and performance in an effort to understand and evaluate these differences, using as background the available evidence regarding deformation limits of masonry walls under in plane and out of plane actions.

KEYWORDS: *masonry, performance, URM, code, Eurocode*

INTRODUCTION

The salient points of a practical seismic assessment framework that is currently being developed in Europe in order to address a pressing need in the field of management of the seismic risk of conventional unreinforced masonry are presented. The intent is to also encompass structures that would normally be classified under traditional and/or historical constructions. Buildings considered typically comprise stone or clay or adobe masonry, low-rise construction with timber diaphragms without diaphragm action (i.e., with timber floors insufficiently connected to the

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lateral load resisting system). Well-constructed buildings in this category may be equipped with a timber ring-beam securing partial diaphragmatic action at the building crest which seems to be a very effective means of seismic protection. Other measures may include timber lacing as shear reinforcement in masonry piers [4] or masonry-infilled timber frames [3].

Seismic assessment of URM 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 [5]. From the point of view of computational analysis and in order to capture these aspects of the response it is required that finite element idealizations using shell or solid elements is required; but based on the current state of the art, robust analysis is not possible unless material brittleness and tensile fracture – characteristic features of masonry, both are neglected. So, although frequently stated as an obvious option, nonlinear analysis of unreinforced masonry buildings is not necessarily feasible except under strict conditions. At the same time, partly owing to the great variety of materials available, and the contributing influence of uncertain boundary conditions during testing, the availability of acceptance criteria may still be established only qualitatively, at best. (Such include the shear strength at the various performance limits, stiffness, deformation capacity to in-plane and out-of-plane loading that could be associated to the various design limit states.)

Estimations of Seismic Demand: Seismic assessment of a masonry structure is generally based on estimations of seismic demand 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. Here it is relevant to note that the basic material is brittle in tension and in the absence of reinforcement it becomes a real challenge to obtain convergence of FE analysis beyond the onset of even the minutest degree of tension cracking. In this context, elastic analysis becomes a point of reference in order to determine the disposition of stress and deformation [3]. It is noted here that in order to enable estimation of the out-of-plane response components, spatial modelling with elements that are equipped with translational and rotational degrees of freedom (i.e. beams, or shell-elements rather than plane-stress elements) is essential. 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 [4]. For older URM structures, *Minimum Knowledge Level* is a norm; according with [4] for this knowledge level the only allowed methods as the Lateral Force

Procedure and the Modal Response Spectrum Analysis, both being linear elastic approaches).

ANALYSIS METHODS

Two seismic assessment code proposals have evolved simultaneously with the same objective, namely to provide a guideline for practical seismic assessment of URM structures [1, 2]. A common definition of seismic hazard is adopted in both, originating in EN 1998-1:2004 [6] as defined for all structures. The greatest differences between the two approaches lie in the recommended procedures for modeling and calculation of seismic demands. In the emerging EN1998-1 framework, [1], a minimum doable target for the estimation of demands is considered to be the nonlinear pushover analysis of the URM, the results of which form the backbone of the requirements in the demand side of the design inequality. A recommended value for the behavior (ductility) factor, q, is 1.5, concerning masonry.

In principle, the methods that may be used during analysis in both [1] and [2] are those proposed in the governing Eurocode [3], 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 URM and historical structures are discussed below.

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 height-wise 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 point-wise 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. This is only valid in URM buildings if the load bearing walls in the two main plan directions are nearly orthogonal to each other, whereas,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.

<u>Determination of force and deformation demands:</u> To resolve internal action and deformation demands in order to be used in assessment, the total seismic lateral load is estimated from Eq. (1):

$$V = C_1 C_m S_e(T_1) \cdot m \tag{1}$$

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

$$T_1 = 0.05H^{3/4} \quad (2)$$

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 expression:

$$C_1 = 1 \text{ for } T_1 \geq T_C; \qquad C_1 = [1+(q-1)\cdot(T_C/T_1)]/q \text{ for } T_1 \leq T_C \quad (3)$$

(T_C is the end of the constant acceleration range of the spectrum [3]); $q=V_{el}/V_y$ 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.

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. Modal analysis has several problems when applied to continuous 3-D structures with flexible diaphragms, owing to the fact that one needs to consider a very large number of modes in order to engage a reasonable amount of mass.

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. The assessment approaches in [1] and [2] deviate significantly with regard to this method, particularly for the case of buildings with deformable diaphragms. The two alternatives are summarized below:

Approach 1: Draft EN1998-3, Chapter 9, 2020 [1]. The standard makes explicit reference to a 3-D model of the structure where individual element resistance curves may be expressed in terms of stress and deformation resultants (forces/moments and end rotations, see section 9.3.2 in [1]; [9], [10]). Individual members are therefore linear (prismatic) beam-columns (2-noded) that model masonry piers and spandrels as parts of a 3-D frame. Wherever this is not possible, macroelements may also be used (i.e., 4-noded panels), connected with the scaffold of the analytical frame model using contact (translational) springs. (The properties of the springs are obtained through calibration with specific case studies of failed structures, as there is no obvious way of calculating them from first principles. By removing and neglecting any form of coupling between springs at a certain node, it is possible to circumvent issues of numerical instability and convergence which would otherwise threaten the robustness of a nonlinear model for URM.)

A limited number of application examples also considering out-of-plane effects have been published [5, 6]. For frame type members representing masonry piers and spandrels the code recommends use of cracked stiffness (≈50% of gross value), considering both flexural and shear deformations for in-plane analysis. This type of analysis is intended for the study of in-plane response of the masonry elements provided that the URM structure has rigid diaphragms. If horizontal diaphragms are flexible, then each single wall should be analysed and verified independently, being subjected to its own seismic actions (including those transferred by supported floors) and to those related to connected out-of-plane loaded walls (no details as to how these may be quantified are yet available). Provided the difference in the natural and idealized dimensions of an equivalent frame member representing a pier or spandrel, rigid zones are used to achieve member connectivity with the nodal points. Additional requirements for the use of the equivalent frame approach are that openings are arranged such that the lengths of adjacent piers may be considered approximately equal, from the level of the foundation to the crest of the wall, and that the ratio of height to the length of the pier (in a single floor) exceeds the limit of 2.0.

Rotation demand and rotation capacity are defined in terms of tangential interstorey drift, i.e. the relative storey rotation developed based on the final position of the member chord, after it is corrected to account for possible rotations of the end nodes (which would relieve part of the distress). Displacement-based assessment is recalled only in verification of local out-of-plane mechanisms in nonlinear kinematic analysis, which is an incremental equilibrium limit analysis carried out by considering geometric nonlinearity. Strut-and-tie models may be used to interpret local mechanisms only for those parts of the structure where the disposition and flow of forces is understood with confidence. Demand, to be compared to the capacity, is the roof displacement corresponding to the target displacement of 4.4.4.4 and EN 1998-1: 2004, 4.3.3.4.2.6(1) for the seismic action considered. Target displacement is evaluated after consideration of significant strength degradation in the load-displacement response of the entire structure (idealized as a trilinear curve without hardening so as to define the yield displacement from the point of sharp change of slope). Torsion is neglected in the absence of stiff diaphragms – instead, it is recommended that each wall is examined separately (although not explained in detail as of yet).

Approach 2: The Hellenic Draft Code for Seismic Assessment for URM: A basis of [2] is the underlying recognition of the limitations of easily accessible commercial software towards nonlinear analysis with solid of shell F.E. elements[3] which are used to represent the continuous URM structure. Application of pushover analysis is found problematic as the conversion to an Equivalent Single Degree of Freedom System (required so as to use the so-called N2 approach) is hampered by incomplete understanding of the fundamental mode of vibration in structures with flexible diaphragms and other secondary flexible elements (such as timber traverses and dividing walls); these tend to dominate modal analysis diffusing the estimation of the fundamental mode of vibration. Thus in [2], a key ingredient of the assessment procedure is estimation of 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. Therefore demands are obtained using as a point of reference the results of elastic analysis; in this approach, the structure is subjected to a uniform field of unit acceleration in the direction of the load so as to ensure that all distributed mass is engaged. Thus the structure is loaded in the lateral direction of interest with forces identical to its self-weight. This is the underlying principle of Rayleigh's approximation of the fundamental response shape [8]: here it is obtained from the determined profile of lateral deflections throughout the building, after these are normalized with the peak value wherever that may occur. The shape function thus derived is then used to determine the properties of the ESDOF idealization of the structure for direct implementation in the N2 approach.

The target displacement obtained from the hazard spectrum [3] given the approximated fundamental period (Eq. (2)) corresponds to the "control node", i.e. the point where the normalizing displacement had developed in determining the shape function (i.e., the location where the shape function is = 1 [8].) In buildings lacking diaphragm action the most displaced node usually occurs at the crest of the building, at mid-span of a long wall or over a spandrel as it is shown in Fig. 1c which illustrates the application of the uniform acceleration procedure to establish the fundamental translational mode of the structure shown in Fig. 1a. 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. Inelasticity is accounted for as follows: For the elastic displacement demand obtained from the displacement spectra of the design hazard with the estimated T_1 value (Eq. 2, i.e. $S_d=S_e(T_1)\cdot T_1^2/40$), local displacements, deformations and strains are evaluated for all the members through the shape function (i.e., $\Delta(x,y,z)=\Phi(x,y,z)\cdot S_d(T_1)$); using the estimated deformations, the individual members are evaluated, considering equilibrium, for the occurrence of "yielding" in flexure or shear. The lowest ratio of elastic force demand to yielding force in the

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[3] 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.

individual masonry piers defines the q value for the building. If $q>1$, the coefficient C_1 (Eq. 3) is estimated, as well as the structure inelastic displacement. An advantage of this type of analysis where the structure is modeled as a continuous shell is that out-of-plane and torsional effects are automatically considered, as well as all the sources of flexibility (including the presence of flexible diaphragms) are inevitably reflected in the pattern of the estimated shape function.

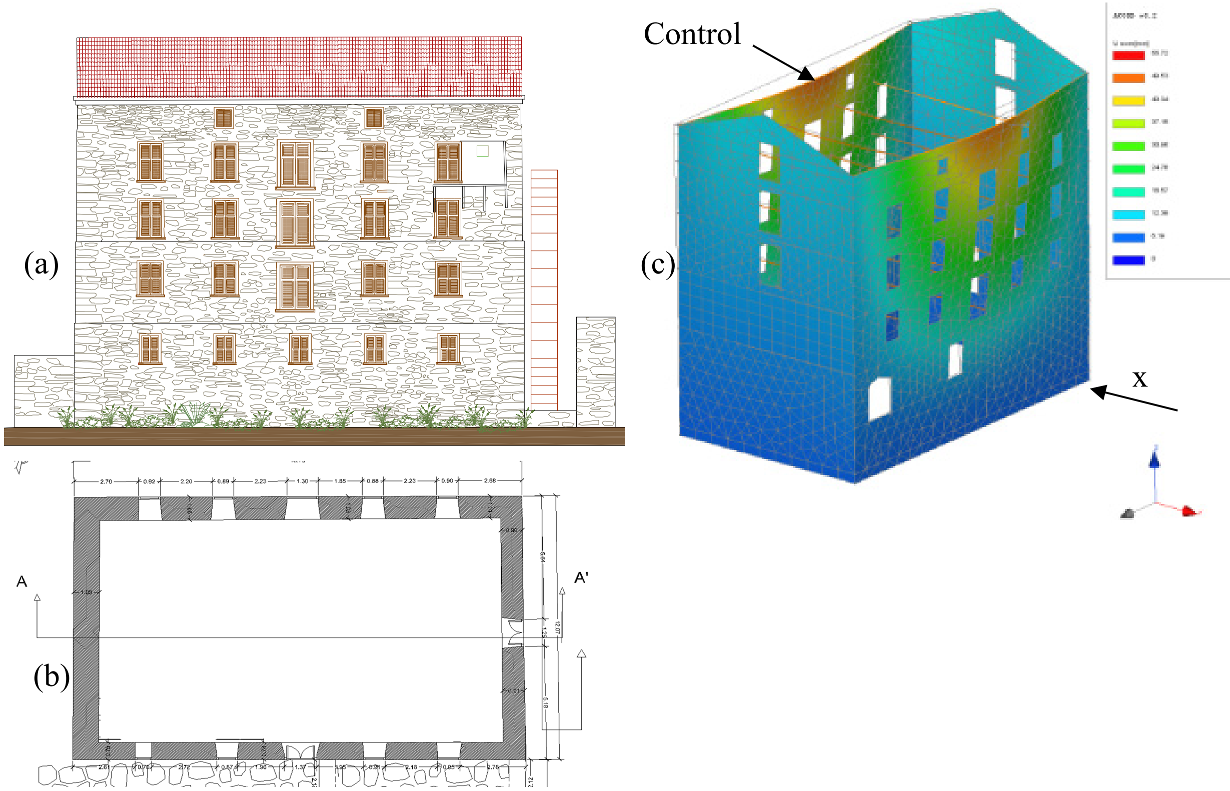

Figure 1: Water mill in Andros –typical application example analysed according with [2]: (a) elevation and (b) plan view (c) Deformation: flexible diaphragms for seismic action // to the x axis

LOAD - DEFORMATION RESISTANCE CURVE OF THE MEMBERS

Apart from the different idealization approach used in the two code models (linear-prismatic vs. surface or solid elements) resistance terms (such as strength and deformation capacity) are obtained using compatible mechanistic models. Therefore, a single approach for the estimation of these terms is given in the following section as an essential accessory of the assessment framework.

The mechanical 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. 9(b)). The kind and direction of the internal action F is selected so that it may characterize the primary stress resultant 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 it would be interstorey drift if the action is flexural, and 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 $\pm\Delta$, 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 in [4] 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 [7] which go as far as even defining nonlinear moment-rotation envelopes for URM piers under reversed cyclic 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%. Several mechanisms may be responsible for this post-cracking resilience, such as friction between wythes, timber lacing in traditional masonry or iron clamps in industrial masonry buildings; also contributing are kinematic constraints that prevent the length change of masonry which precedes its catastrophic collapse; all these of course also depend on the manner of construction.

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.

Notional Elastic Branch up to Phenomenological "Yielding"

All methods recommended for calculating the demand 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. 9(b)). 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 0.15% 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 0.2% 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 exceedance of either of the two strength mechanisms (the least resistance controls the limit of "yielding") as defined below.

Definition of Yield Strength, F_y depending on the mode of failure:

(a) Development of flexural strength of the masonry pier, in the critical cross section. In the absence of reinforcement, the flexural behavior refers to rotation of the pier about the base (see Fig. 3). For any individual pier the length of compression zone a pre-requisite 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.

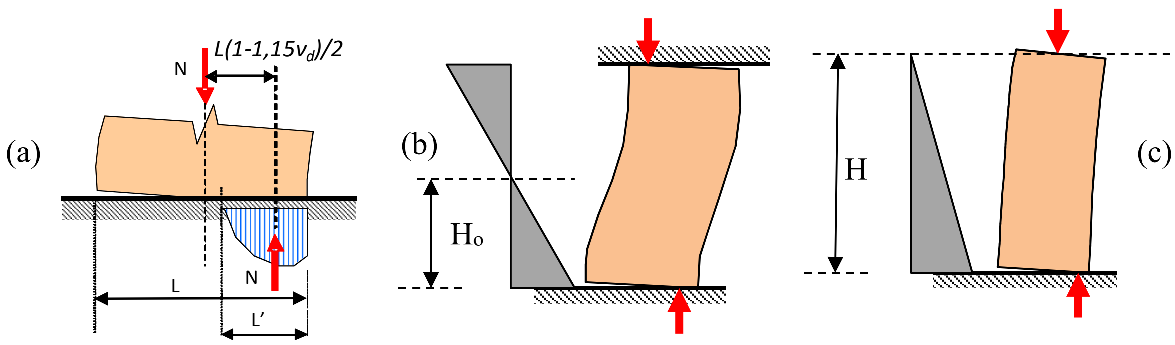

Figure 3: Bending of a pier in its plane of action. a) Definition of Internal Moment and (b) Definition of the Effective Shear span H_o with reference to the moment diagram.

Expressed as a shear force at the onset of flexural yielding, flexural strength is given by Eq. (4) (N is the axial load of the pier, and $v_d = N/(L \cdot t \cdot f_{md})$ is the normalized axial load of the pier (Fig. 3(a)):

$$F_{y,fl} = L \cdot (1 - 1.15 \cdot v_d) \cdot N / 2H_o \qquad (4)$$

(*b*) *Development of Shear Strength of the masonry pier (Fig. 4),* either by exceeding the tensile strength of URM in the principal tensile direction (diagonal cracking in Fig. 4(a), Eqn. (5a)), or as a result of failure by sliding along the horizontal joints of masonry (Fig. 4(b), Eqn. (5b)). The shear strength f_{vd} corresponding to the above two mechanisms of failure ($f_{vd,t}$ and $f_{vd,s}$) are estimated from (sign convention is compression-positive; f_{vo} is the cohesive strength at the mortar-block joint, and $v_d f_d$ is the overbearing compressive stress in the plane of sliding):

$$f_{vd,t} = \sqrt{f_{wdt} \cdot (f_{wdt} + v_d \cdot f_d)} \quad (5.a); \qquad f_{vd,s} = f_{vo} + \mu \cdot (v_d f_d) \quad (5.b)$$

Here μ is the coefficient of friction along the sliding surface (may be taken equal to 0.4). The strength of a URM wall expressed as a shear force associated with either mode b.1) or b.2) is:

$$F_{y,v} = f_{vd} \cdot L' \cdot t \qquad (6)$$

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, $f_{vd} \leq 0{,}065 f_d$, where f_d is the compressive strength of the masonry (f_d could be taken as the strength of the homogenized masonry wall). Timber laces if they exist contribute to the shear strength of piers as was estimated by Eq. (6) through the addition of term V_{tier} (Fig. 5).

$$F_{y,tier} = u_{b,tier} \cdot p_{tier} \cdot L_{b,tier} \tag{7}$$

$u_{b,tier}$ is the specific cohesion strength (MPa) at the interface between the timber lace and the URM, p_{tier} is the contact perimeter 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 45° crack plane.

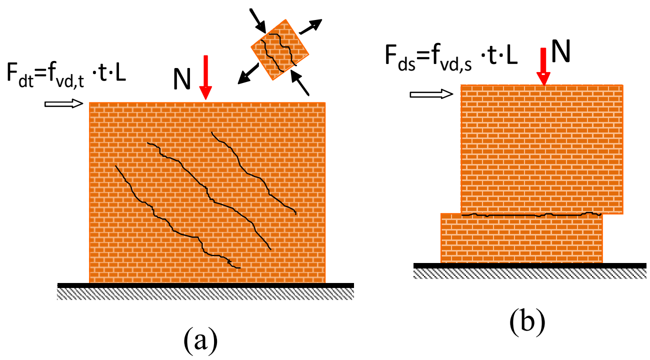

Figure 4: Shear failure: a) diag. cracking, b) sliding

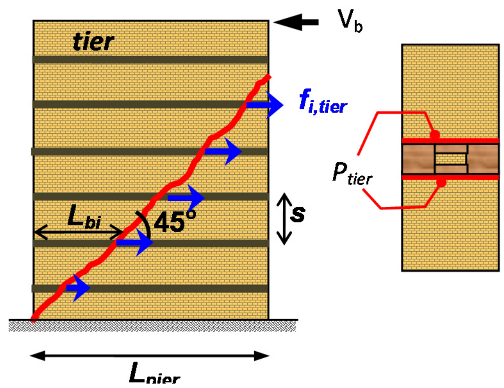

Figure 5: Timber lace contribution

STRENGTH OF WALLS TO OUT-OF-PLANE ACTION

Wall piers loaded normal to their plane of action under a combination of horizontal pressure and 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). Strength calculations are based on the classical approach of superposition of stress blocks resulting from the axial load (v_d is the normalized axial load at the critical section), and the flexural moment (Fig. 6(a)) and the requirement of non-exceedance of the tensile strength of masonry, f_{xk}.

$$\text{Per unit length of wall,} \quad M_{max,\,1} = (f_{xk,1} + v_d f_d) \cdot t^2/6; \quad M_{max,\,2} = f_{xk,2} t^2/6 \tag{8}$$

In Eq. (8) $f_{xk,1}$ is the flexural strength of masonry for bending parallel to the bed joints (i.e., vertical unit-width strips), and $f_{xk,2}$ is the flexural strength of masonry for bending in direction orthogonal to the bed joints. The ultimate flex. strength is supported by the axial load only over the internal lever arm that forms between the centroid of the cross section ($t/2$) where t is the wall thickness, and the centroid of the compression zone (≈0.15t if the compression zone is taken equal to 0.3t, see Fig. 6(b).

$$M_u = N \cdot (0.5 - 0.15t) = 0.35N \cdot t \qquad (9)$$

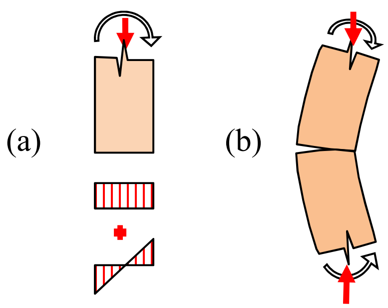

Figure 6: Flexural strength calculation for out of plane bending. (a) Stress superposition at yielding, (b) Ultimate Flexural Strength

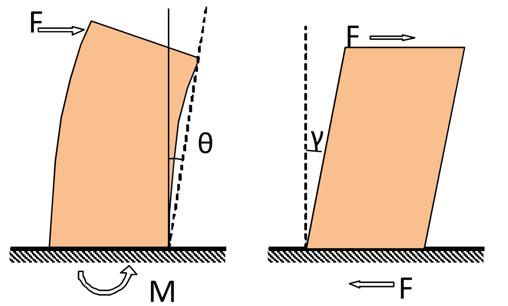

Figure 7: Wall under in plane force

DEFORMATION CAPACITY

The nominal deformation capacity of URM walls, θ_u, is estimated according to the plane of action. For walls loaded in their plane, drift capacity of flexural elements (Fig. 7(a)) is taken equal to $0.8\% \cdot H_0/L$ where L is the pier wall length and H_0 is the distance from the critical cross section

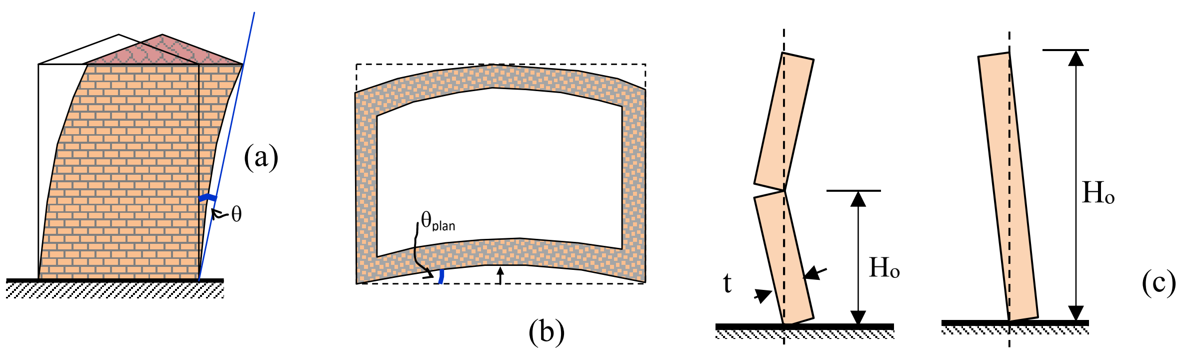

Figure 8: Definition of drifts: (a) In-plane heightwise, (b) Out-of-plane, horizont., (c)

where flexural strength is attained to the point of zero moment (i.e., the shear span, Fig. 3(b)). Ultimate drift capacity of URM walls controlled by shear (Fig. 7(b)) is taken equal to 0.4%. For walls loaded normal to the plane of action, deformation capacity in out of plane bending is obtained considering Fig. 8b and c, where H_o the distance from the point of maximum translation to the pivot of rocking):

$$\theta_{R,u} = t/H_o \quad ; \qquad \theta_u = min\{0.3\% \cdot (H_o/t);\ \theta_{R,u} \cdot (1 - (F_y / F_{Rd}))\} \qquad (10)$$

ACCEPTANCE CRITERIA

Performance (checking of the design inequality) in terms of internal forces and deformations is carried out for individual structural members at performance levels A (Damage Limitation, DL, where the acceptance criteria are expressed in terms of elastic forces / deformations), B and C (Significant Damage, SD, and Collapse Prevention, CP) where the 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. 9).

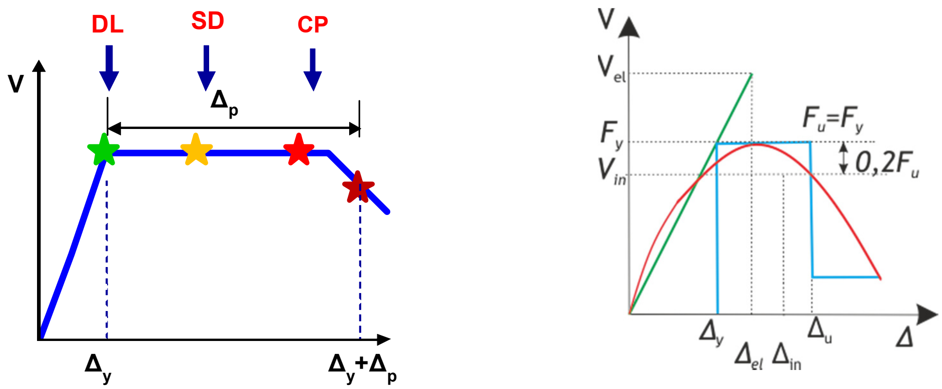

Figure 9: (a) Definition of performance levels; (b) Typical member response curve

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