MODELLING THE SEISMIC RESPONSE OF UNREINFORCED EXISTING MASONRY BUILDINGS: A CRITICAL REVIEW OF SOME MODELS PROPOSED BY CODES

S.Cattari[1] and S.Lagomarsino[2]

[1] Research Assistant, Dept. of Civil, Environmental and Architectural Engineering , University of Genoa, Via Montallegro 1, 16145, Genoa, Italy, serena.cattari@unige.it

[2] Full Professor, Dept. of Civil, Environmental and Architectural Engineering , University of Genoa, Via Montallegro 1, 16145, Genoa, Italy, sergio.lagomarsino@unige.it

ABSTRACT

Distinctive features which characterize the existing unreinforced masonry building (such as the presence of flexible floors and/or weak spandrels) make the possibility of simulating the actual conditions of the structure crucial: indeed, models usually employed for new constructions are not always suitable for existing ones. Different strategies of modelling can be adopted with increasing levels of accuracy and computational effort. In this paper attention will be focused on very Simplified Models, such as “strong spandrel-weak pier” or “weak spandrel-strong pier”, and much more complex models such as “Equivalent Frame Model” proposing a critical review of their use and of the reliability of the assumptions which they are founded on. Particular attention will be paid to the issues related to the assessment of the in-plane strength of URM walls distinguishing between those related to the idealisation of the masonry wall in an equivalent model (e.g. geometry and boundary conditions assumed for each panel) and those related to the proper evaluation of the strength of each structural element, as defined by the previous step, on which the non linear response is concentrated (e.g. resistance criteria to be adopted distinguishing the case of pier from that of spandrel). In particular the fundamental role played by spandrel elements will be discussed.

KEYWORDS: existing masonry buildings, seismic response, simplified model, equivalent frame model

INTRODUCTION

The large population of existing and historical unreinforced masonry buildings all over the world points to the need to improve the knowledge of their seismic behaviour, setting analytical and numerical models for their analysis. Safety evaluations are oriented to assessing whether or not retrofitting interventions are needed. In order to demonstrate that a structural intervention is necessary and effective, accurate numerical models to predict the response of the structure are essential. In case of existing buildings, the possibility of simulating the actual conditions of the structure represents a crucial issue. On the one hand, models usually employed for new constructions are not always equally suitable for existing ones leading to results which are on unsafe side; on the other hand, the systematic adoption of too simplified assumptions may lead to results which are on the safe side, but much too severe underestimation of the actual resistance would not be acceptable due to the invasiveness of the resulting retrofitting interventions.

In particular it is worth referring that, focusing the attention only on the methods of global analysis, the seismic response of un-reinforced masonry (URM) buildings is strictly related to both the in-plane strength of walls and the connection and load transfer effect due to the floors. In particular, with reference to the in plane response of masonry complex masonry walls with openings, it is possible to recognize two main structural components: *piers* and *spandrels*. Piers are the principal vertical resistant elements for both dead and seismic loads; spandrels, which are intended to be those parts of walls between two vertically-aligned openings, are the secondary horizontal elements, coupling piers in the case of seismic loads. It is worth noting that, although "secondary elements", spandrels significantly affect the boundary conditions of piers (i.e. fixed-fixed or cantilever) with great repercussions on the prediction of their load-bearing capacity. Despite this as a function of the modelling strategy adopted to analyze the in-plane response of a masonry wall, the actual modelling of spandrels may even result un-requested.

Among the possible choice of modelling strategies proposed in literature and codes, in this paper attention will be focused on the following approaches: i) very simplified model, such as "strong pier-weak spandrel" or "weak pier-strong spandrel" (Simplified Models); ii) idealisation of the structure through an "equivalent frame" in which each resistant wall is discretized by a set of masonry panels, in which the non-linear response is concentrated, connected by rigid areas (Equivalent Frame Models). In particular a critical review of their use to existing buildings and of the reliability of the assumptions which they are founded on is proposed. Particular attention will be paid to the issues related to the assessment of the in-plane strength of URM walls distinguishing between those related to the idealisation of the masonry wall in an equivalent model (e.g. geometry and boundary conditions assumed for each panel) and those related to the proper evaluation of the strength of each panel, as defined by the previous step, on which the overall response of the wall depends (e.g. resistance criteria to be adopted distinguishing the case of pier from that of spandrel).

ISSUES RELATED TO THE IDEALISATION OF THE MASONRY WALL IN AN EQUIVALENT MODEL

First step of any modelling strategy is the idealisation of the masonry wall in an equivalent model. Thus the first fundamental issue is related to the hypotheses assumed for the main structural components which compose it, such as previously introduced *piers* and *spandrels*.

Regarding this, in fact, as a function of the modelling strategy adopted to analyze the in-plane response of a masonry wall, the actual modelling of spandrels may result un-requested: this is the case of very Simplified Models (like those suggested by international codes such as FEMA 356 [1] and FEMA 306 [2] or some others often adopted in the past such as the POR method [3]). In particular among these latter, the idealisation of a "strong spandrel-weak pier" model (Model *I*) assumes that piers crack first, thus averting the failure of spandrels which are usually assumed as infinitely stiff portions assuring a complete coupling between them. On the contrary in case of "weak spandrel-strong pier" (Model *II*), the hypothesis of both null strength and null stiffness of spandrels is adopted then assuming the piers as uncoupled; however, it is worth noting that in most cases it is licit to assume that the vertical resistant elements are at least coupled by the translational displacement components due to the action carried on by floors. However, it has to be stressed that only preliminary evaluations on the effectiveness of spandrels are requested in order to properly orientate the choice between these two extreme idealisations. Actually, FEMA 356 does not provide explicit guidance on this issue; more detailed indications are traced in FEMA 306, which specifically deals with the evaluation of earthquake damaged buildings. In particular, this latter code addresses the choice between Model *I* and *II* as a function of the damage state of spandrels: if there is no spandrel damage, then Model *I* should be used; if the spandrels are fully cracked, Model *II* has to be adopted; finally, when the spandrels have a reduced capacity, that is they are only partially damaged, (in FEMA 306 specific criteria are proposed to quantify this "reduced capacity", as discussed in the following section), the actual strength of the wall is obtained as the lowest value obtained by adopting Model *I* and Model *II*, in which the expected reduced forces transmitted by spandrels to piers also have to be taken into account, respectively. However, of course, the presence of specific constructive details plays a further crucial role in addressing this choice. For that matter, as a rule, the assumption, which Model *I* is based on, seems consistent with new buildings in which masonry spandrels are always connected to lintels, tie beams and slabs made of iron or reinforced concrete. In fact, these elements, being stiff and tensile resistant, assure a consistent coupling between piers, making the contribution of masonry negligible. On the contrary, in historical and existing buildings spandrels are in many cases intrinsically weak elements. In fact, lintels are usually made of wood or masonry, tie beams are often absent and floors are flexible (e.g. due to the presence of vaults or wooden floors): thus Model *II* is assumed in most cases. Once the choice has been made, according to the assumptions of Simplified Models, since only pier elements are modelled, the definition of both their effective height and boundary conditions plays a crucial role for the reliable assessment of the overall strength of the wall. The method outlined in FEMA 356 addresses these two model types by requiring that piers be designated as either fixed-fixed (coupled – Model *I*) or cantilevered (un-coupled- Model *II*) and that the effective height (H_{eff}) be defined as a function of the height of adjacent openings.

Firstly the issue related to the definition of H_{eff} will be discussed. Since in the case of un-coupled piers (Model *II*) it is assumed that spandrels fail before the pier and thus they cannot in principle be assumed as reference to define the height of piers, analogously to that proposed by FEMA 306 in the case of fully cracked spandrels, in most cases H_{eff} is assumed equal to that of the entire wall (multi-storey pier). However, due to the minimum translational coupling provided at level of each storey by the floors, which reduces the free span of the pier, it seems justifiable relating H_{eff} to the inter-storey height, representing the total height of the wall only an upper limit. Moreover, also in the case of Model *I*, if the definition of H_{eff} appears quite trivial in case of a regular pattern of opening, it turns out more difficult and ambiguous when openings are irregularly arranged. Recently some authors [4] proposed to define H_{eff} as the height over which a compression strut is likely to develop at the steepest possible angle (e.g. assuming that cracks can develop either horizontally or at 45°). Moreover, as highlighted by FEMA 306, also the pattern of pre-existing cracks has to be taken into account to properly define H_{eff}. Secondly, with reference to the boundary condition assumed, it is obvious that the fixed-fixed and the cantilever ones represent only two extreme idealisations, but actually, as a function of the effective stiffness of spandrels, an intermediate scheme should be more appropriate. Regarding this issue, recently Craig et al. [5] proposed to alter H_{eff}, by a factor calibrated on the basis of numerical results, to account for the less than ideal end fixity (e.g. due to the asymmetry of the opening adjacent to the pier). Figure 1 shows a schematic representation of the wall idealisation coherent with the adoption of Simplified Models *I* and *II*.

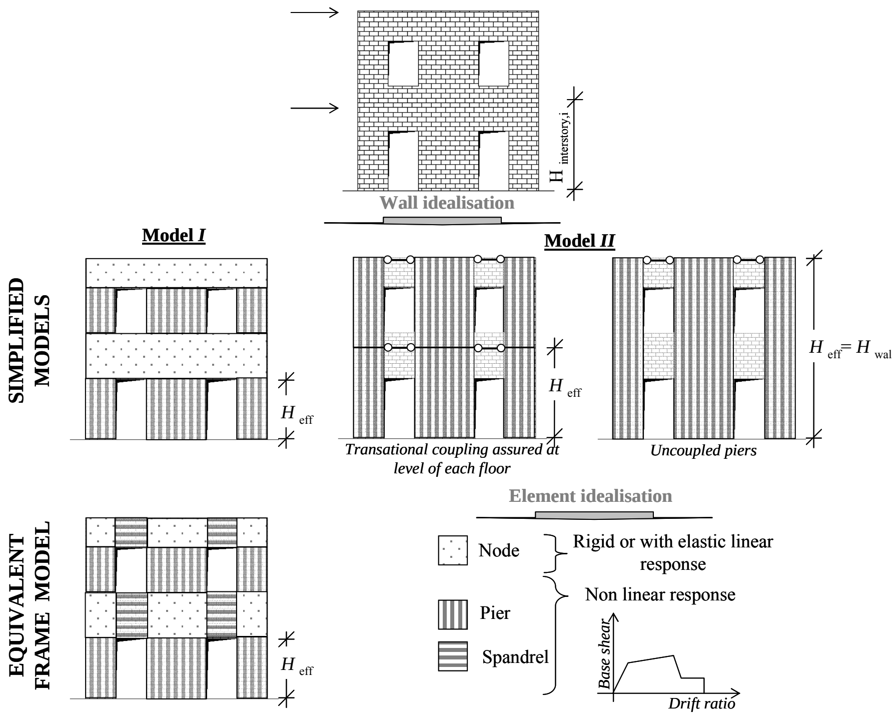

Figure 1: URM wall idealisation according to Simplified and EF Models

Despite the advantage of adopting very simplified and manageable models, since they are based on an aprioristic choice, the following troublesome issues arise. First of all, it is conceivable that both of these limiting cases are inappropriate for certain walls, which may display both types of response in different regions or which can be involved in a different idealisation progressing the non linear response of the structure. Moreover, it is not at all a foregone conclusion that the presence of certain constructive details (e.g. r.c. beams interposed inside the spandrels), not supported by a quantitative evaluation of their effectiveness, is sufficient to assure the achievement of the hypotheses which these simplified models are based on. It may be stated that all the latter issues highlight the need to refer to models that are more detailed. Among the possible alternatives, the modelling strategy based on the idealisation of the structure through an "equivalent frame" seems very suitable for the analysis of standard masonry buildings, as also proposed in recent international and national codes [6,7]. Having the advantage of a reasonable computational effort, complete 3D models of URM structures can be obtained assembling walls, of which only in-plane response is modelled [8, 9]. As previously introduced, each resistant wall is discretized by a set of masonry panels (pier and spandrel), connected by rigid areas (Figure 1). The definition of the portions, in which the non-linear response has to be concentrated, may be carried out following principles similar to those adopted for H_{eff} (pre-existing damage pattern, arrangement of openings). Since both pier and spandrel elements are modelled, by adopting this type of model, the transition through different boundary conditions is directly obtained from the progressive damage of elements. However, it is worth noting that the use of the Equivalent Frame Models is regulated by codes by defining the cases in which masonry spandrels may be taken into account as coupling beams in the structural model; these provisions mainly concern the bonding to the adjoining walls, the connection both to the floor tie beam and to the lintel below. However, when these conditions are satisfied, implicit reference is usually made to the verification criteria proposed for piers without significant differences: due to the low values of axial load usually acting on spandrels, this assumption leads to a dominance of the flexural failure which appears unrealistic if compared with that testified by the earthquake damage observation. Thus the adoption of not reliable resistance criteria risks to lead to much severe underestimation of the actual resistance of the structure in particular in case of existing buildings.

ISSUES RELATED TO THE IDEALISATION OF THE BEHAVIOUR OF THE SINGLE URM PANEL

Once having idealised the masonry wall into an assemblage of structural elements, the reliable prediction of its overall behaviour mainly depends on the proper interpretation of the single panel in terms of stiffness, strength and ultimate displacement capacity by assuming a proper force-drift relationship. A generalized relation like this proposed in FEMA 356 is illustrated in Figure 1; unlike the latter, other codes such as Eurocode 8 [6] or the Italian one [7] assume a relationship bilinear without hardening and residual capacity. Actually the specific characterization of this relationship then depends on the different failure modes which may occur in the panel. Observation of seismic damage to complex masonry walls, as well as laboratory experimental tests, have shown that a masonry panel subjected to in-plane loading may show two typical types of behaviour (that is, flexural and shear), to which different failure modes are associated: *Rocking* and *Crushing* (flexural behaviour); *Sliding Shear Failure* and *Diagonal Cracking* (shear behaviour). As known, the occurrence of different failure modes depends on several parameters: the geometry of the pier; the boundary conditions; the acting axial load (N); the mechanical characteristics of the masonry constituents (mortar, blocks and interfaces); the masonry geometrical characteristics (block aspect ratio, in-plane and cross-section masonry pattern).

In the paper attention will be focused in particular to the prediction of the overall shear strength (V) of masonry panels. The most common simplified models present in the literature and codes are based on the approximate evaluation of the local/mean stress state produced by the applied forces on predefined points/sections of the panel, assessing then its admissibility with reference to the limit strength domain of the constituent material, usually idealised through simple schematizations based on few mechanical parameters (e.g. the compressive strength of the masonry f_{cu}, the diagonal tensile strength of masonry f_t, the tensile strength of block f_{bt}, the parameters characterizing the mortar joints, that is the cohesion c and the friction coefficient μ respectively). The reliability of this approach has recently been assessed by the authors in [10]. Moreover in [10] some incongruence present in the codes related to the use of these criteria has been discussed; these inconsistencies are mainly related to ambiguity on the correspondence between the failure mode interpreted and the resistance criterion adopted or incongruence between the attribution of some coefficient values and the hypotheses which the criterion proposed is based on. Table 1 summarizes the most common simplified models present in the literature and codes, where in particular: k_{1r} is a coefficient taking into account the slenderness (λ) and the boundary conditions of the pier (assumed equal to $\lambda \cdot \Psi'$, being Ψ' the distance from zero moment adimensionalized to the height of panel H); κ is a coefficient taking into account the assumed normal stress distribution at the compression toe (usually κ =0.85); k_{1s} is a coefficient which takes into account the actual compressed part of transversal section A (usually k_{1s}=1 in case of Eq. (3) and k_{1s} <1 in case of Eq.(2) as a function of the constitutive law and the distribution of stresses assumed); k_{1d} is the ratio between the shear stress in the reference point/section and the mean shear stress (usually it is assumed as a function of λ in case of Eq. (3) ÷(5) and equal to 1 in case of Eq. (1)); k_{2d} is the ratio between the shear stress applied on a block and the local shear stress at its centre (usually k_{2d} =2.3); φ is a parameter describing the interlocking of masonry pattern ($\varphi = 2h/b$, h and b being the height and width of blocks.

Table 1: Simplified models aimed to interpret the failure modes of masonry panels

Failure mode [References]	Strength criterion
Rocking/Crushing	$V_{Crushing} = \frac{N}{2k_{1r}}\left(1-\frac{N}{A\kappa f_{cu}}\right)$ (1)
Bed Joint Sliding	$V_{BJS} = \frac{k_{1s}A}{k_{1d}}\left(c+\mu\frac{N}{k_{1s}A}\right)$ (2)
Diagonal Cracking (*through joints*) [11]	$V_{DC,a1} = \frac{k_{1s}A}{k_{1d}}\left(\bar{c}+\bar{\mu}\frac{N}{k_{1s}A}\right), being\ \bar{c} = \frac{c}{1+\mu\varphi}, \bar{\mu} = \frac{\mu}{1+\mu\varphi}$ (3)
Diagonal Cracking (*through blocks*) [11]	$V_{DC,a2} \frac{f_t A}{k_{1d}k_{2d}}\sqrt{1+\frac{N}{Af_t}}$ (4)
Diagonal Cracking [12]	$V_{DC,b} = \frac{f_t A}{k_{1d}}\sqrt{1+\frac{N}{Af_t}}$ (5)

In particular the following significant remarks can be stated. In the case of *Diagonal Cracking* it is possible to recognize two main types of model: models describing masonry as a composite material [11], considering the development of cracks along its constituting components (joints and blocks) separately; models describing masonry as an equivalent isotropic material [12], considering the development of cracks along principal stress directions indistinctly. These two models may provide very different strength previsions describing typologies of masonry which are very different. For these reasons, differently from the use usually proposed in codes which either takes both into account indistinctly assuming then the minimum prediction provided (this the case of FEMA 356) or neglects the possible occurrence of one of them (EC8), a choice between them should be made as a function of the type of masonry examined. The anisotropy of masonry plays a decisive role in addressing such a choice [10]. In fact, coherently with the hypotheses adopted, Turnšek and Čačovič's criterion [12] seems more suitable if masonry behaves as a homogeneous and isotropic material, whereas Mann and Müller's theory [11] seems more appropriate if masonry behaves as an anisotropic material. Moreover it is worth noting that *Sliding Shear Failure* and *Diagonal Cracking* (based on Mann and Müller theory) may be traced back to the same formal expression (as in FEMA 356), thus the meaning attributed to the parameters (cohesive and frictional contributions, $k_{1d,}$ k_{1s}) represents a very crucial distinctive feature. Finally, a further remark is related to the dependence of the criteria aimed at describing flexural or shear behaviour on boundary conditions. It is worth noting that, whereas in the case of *Rocking* and *Crushing* failure modes the parameter k_{1r}, takes into account both the slenderness and the boundary conditions of the panel, in case of *Diagonal Cracking* only the influence of the slenderness is considered by the k_{1d} coefficient. Since passing from fixed-fixed to cantilever boundary conditions the flexural capacity of the panel halves (in fact Ψ' passes from 0.5 to 1) whereas that associated with the shear response remains unchanged, in the case of cantilever condition risks *Rocking* or *Crushing* failure modes prevailing indistinctly. With reference to this issue, the results of parametrical non linear FEM analyses performed recently by the authors on panels subjected to static in-plane loading with different levels of axial loads, slenderness and boundary conditions, showed that, as a function of the latter two factors, both the shear stress distribution on the panel and the point at which the diagonal crack acts significantly vary. Figure 2.a shows the qualitative trend obtained for k_{1d} coefficient as a function of λ and Ψ' (calibrated on the basis only of those cases in which a *Diagonal Cracking* failure mode occurred).

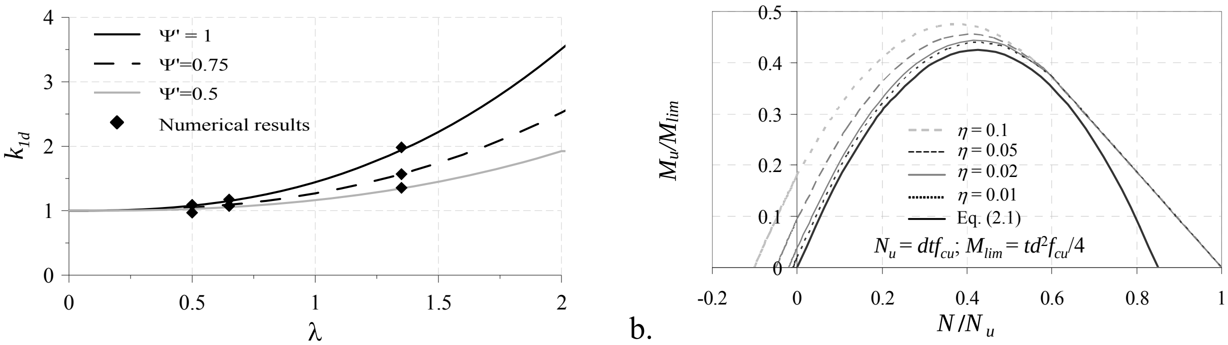

Figure 2: a. Trend of the k_{1d} coefficient as a function of λ and Ψ'; b. Proposed domain for spandrel elements in [13] for different values of η (= f_{tu}/f_{cu})

It is worth noting that all the afore discussed criteria have been formulated and validated by comparison with experimental results with reference to the case of pier elements. Common practice is then to adopt the same failure criteria even in the case of spandrel elements, assuming spandrel behaviour as that of a pier rotated by 90°. However, the boundary conditions of spandrels are very different from those of piers, in particular due to the interlocking with the contiguous masonry regions, thus transposing the experimental results of piers to spandrels without modifications can be inconsistent. Only a few codes propose some specifications for these elements. FEMA 306 proposes an evaluation procedure for the moment capacity of the spandrel which, unlike the pier, is assumed to be derived from the interlocking between the bed joints and collar joint at the interface between the pier and the spandrel; however the result of this evaluation is only aimed at properly orientating the choice between Simplified Models *I* and *II*. The Italian code [7], which has recently been revised, makes a distinction in the resistance criteria of spandrels as a function of the hypothesis assumed for the acting axial force: if it is known from the analysis the same criteria as the piers are assumed; if it is unknown, for the flexural behaviour, if the spandrel is coupled to another tensile resistant element, a response as equivalent strut is presupposed. Due to moderate values of the axial load which usually characterize spandrel elements, the use of Eq.(1) (analogous to that proposed for piers) leads to very precautionary predictions of the strength: as a consequence in many cases *Rocking* tends to prevail over *Diagonal Cracking* much more frequently than that testified by earthquake damage observation in existing buildings or in experimental campaigns. Moreover also presupposing a strut response like the Italian code, the strength associated with *Rocking* mechanism differs from zero value only if a tensile resistant element is coupled to the masonry spandrel. In order to overcome this implausible result, it seems reasonable to assume that masonry spandrels supply further unknown resources with regard to the flexural response. Regarding this issue, recently the authors [13] have proposed an original formulation founded on the assumption that the response as “equivalent strut” of spandrel may also occur by virtue of the interlocking phenomena which can be originated at the interface between its end-sections and the contiguous masonry: as a consequence, it can define an “equivalent” tensile strength f_{tu}, which properly characterizes the spandrel element, not the masonry material. Figure 2.b illustrates the proposed domain for different values of the ratio η between f_{tu} and f_{cu}. It is worth highlighting that the beneficial effect due to the proposed criterion is decisive even for very moderate values of η because it confers a strength (though minimum) even in those cases in which, in the absence of another tensile resistant element coupled to the spandrel, it would be identically equal to zero.

COMPARISON OF THE PREDICTIONS PROVIDED BY THE DISCUSSED MODELLING STRATEGIES

In order to quantify the differences in the overall capacity of a masonry wall resulting from the adoption of the modelling strategies discussed in the previous section, the response of a three-storey URM wall with two lines of vertically aligned openings has been analyzed. In particular the following cases have been compared by adopting respectively: Model *I*; Model *II* (assuming H_{eff} of piers equal to the inter-storey height); EF Model in which the strength criteria equal to those of piers have been assumed for the spandrel (Case *A*); EF Model in which for the flexural behaviour the criterion proposed in [13] has been assumed for the spandrel by assuming η as 0.05 (Case *B*); EF Model in which reinforced concrete beams have been modelled coupled to spandrel elements (Case *C*). Figure 3.a shows the results concerning the pushover analysis with “uniform” load pattern (that is proportional to mass) in terms of V_{base}/M (shear base adimensionalized to the total mass of the structure) $-u/u_{ultimateA}$ (displacement of control node located on top of the wall adimensionalized to the ultimate value obtained in the case of Model *I*) curve. The analysis was stopped at the step corresponding to 20% decay of the maximum base shear reached. Moreover the collapse of each structural element has been defined by assuming the drift values equal to 0.4% and 0.8% for the shear and flexural failure modes respectively. It is worth noting that the comparison of the results in terms of adimensionalized force-displacement curve seems particularly significant according to nonlinear static procedures which in the last decade have been increasingly promoted as tool of verification by the achievement of performance-based earthquake engineering concepts.

The results obtained may be summarized as follows. Models *I* and *II* define the range of the possible pushover curves which can be associated with the structure; actually it needs to be pointed out that in the case of Model *II* the adoption of a multi-storey pier should lead to a lower bound even more punitive which in most cases seems excessively conservative. However, this range appears too wide in terms of strength, stiffness and ductility definition, all three aspects which play a fundamental role by referring to the adoption of non linear static procedures as tools of verification. Moreover, even if the systematic adoption of Model *II* in case of existing buildings of course should lead to results which are on the safe side, a much too severe underestimation of the actual resistance would not be acceptable due to the invasiveness of the resulting retrofitting interventions. The comparison between Model *I* and Case *C* stresses how the presence of certain constructive details is not in general sufficient to assure the satisfaction of some simplified hypotheses: this is the case of the presence of reinforced concrete beams (usually associated with the presence of rigid floors), characterized by a finite stiffness, which do not correspond with the assumption of fixed rotations at the level of each floor. As a consequence, Model *I* provides an upper bound which operates on the unsafe side. Case *A* provides results similar to those of Model *II* highlighting how the advantages associated with the use of much more complex models risks being defeated by the adoption of resistance criteria, in particular for spandrel elements, which are not suitable. By assuming the proposed criterion for spandrel elements (Case *B*), both a significant increase in the overall resistance and a decrease in the global ductility can be observed with respect to case *A*. The latter result can be explained by both the different pattern and sequence of damage which occurred in cases *A* and *B*. In fact in Case *A*, due to the moderate axial load acting on the spandrel elements, since the initial steps of the analysis a *Rocking* mechanism occurs in almost all spandrels which thus supply a weak coupling for piers. On the contrary, in case *B* one can observe: a first phase in which only the spandrels located on the top floor show the activation of a *Rocking* mechanism (in fact, due to the moderate compressive stresses acting on the contiguous masonry portions, they cannot rely much on the interlocking phenomena); an intermediate phase in which the damage progressively occurs also in the spandrels located at mid-storeys; a final phase, in which the damage also spreads to piers located on the ground floor. It is worth noting that a design aimed at promoting a "uniform" global mechanism like that of Case *B* would be advisable for many reasons: it agrees with the "capacity design" criterion; it complies with the concept of "sustainable repair"; experimental campaigns have pointed out that damage to spandrels produces a more significant energy absorption than that to piers [14].

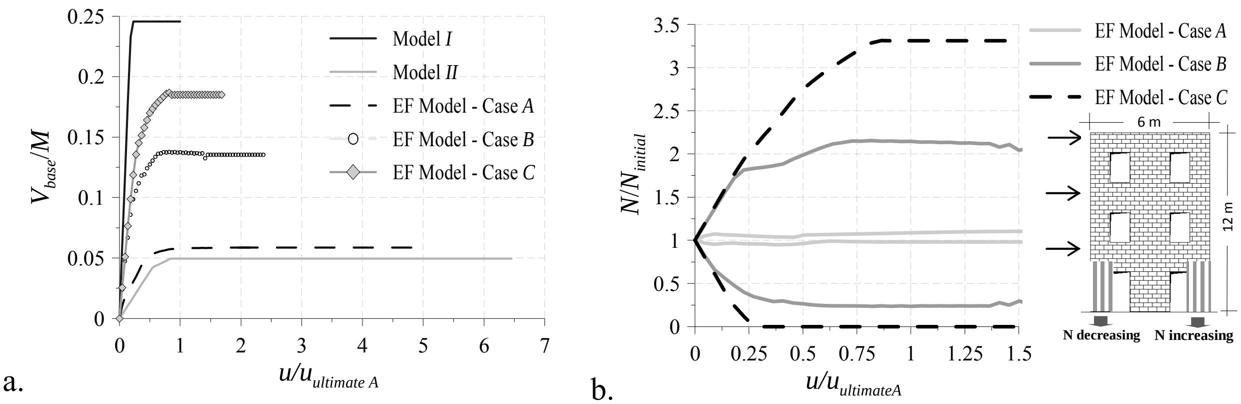

Figure 3: a. Pushover curves resulting by the adoption of the different models discussed; b. Variation of the axial force on piers passing from Case *A* to Case *C*

Further remarks on the use of EF Models with respect to the Simplified ones, is related to the evaluation of the overturning effects. Indeed, while Simplified Models neglect this effect, with adoption of EF Models the prediction of the actual strength of the panel may easily depend on the axial force at the current step of the analysis. Since the in-plane response of a masonry panel significantly depends on the acting axial load, this aspect is particularly relevant. As licit to be expected, more the coupling among piers is significant, more the overturning moment acts to alter the vertical stresses increasing or decreasing their initial value. Figure 3.b illustrates this phenomenon passing from Case *A* to *C*.

FINAL REMARKS

In the paper the evaluation of the in-plane response of URM walls has been investigated by comparing the results associated to the adoption of Simplified and Equivalent Frame Models. The results may be summarized as follows. The range of possible responses delimited by Simplified Models seems too wide in terms of strength, stiffness and ductility definition.

Moreover it has been highlighted how the presence of certain constructive details is not in general sufficient to assure the satisfaction of the simplified hypotheses which they are based on. This is the case in particular of “weak pier-strong spandrel” model which is usually adopted for new buildings: as a consequence the results obtained can be on the unsafe side. On the other hand, even if the systematic adoption, in the case of existing buildings, of model such as “weak spandrel-strong pier”, leads to results which are on the safe side, however too severe underestimation of the actual resistance would not be acceptable due to the invasiveness of the resulting retrofitting interventions. In order to overcome these implausible results, the adoption of Equivalent Frame Models seems particularly attractive and efficient. The role at all secondary played by spandrel elements emerged: in addition to the pressing needed to take into account their actual modelling, the importance of providing suitable strength criteria to interpret their response has been stressed.

REFERENCES

1. Applied Technology Council (ATC).(2000). Pre-standard and commentary for the seismic rehabilitation of buildings, FEMA 356, Washington D.C.
2. Applied Technology Council (ATC).(1998). Evaluation of earthquake damaged concrete and masonry wall buildings. Basic procedures manual, FEMA 306, Washington D.C.
3. Tomazevic M. (1978). “The computer program POR”, Report ZRMK, 1978 (in Sloveno).
4. Moon F.L., Tianyi Y., Leon R.T., Kahn L.F. (2006). “Recommendations for Seismic Evaluation and Retrofit of Low-Rise URM Structures”, *Journal of Structural Engineering*, ASCE , Vol.132, n°5.
5. Craig Ji., Goodno BJ.,Towashiraporn P., Park J. (2002). “Response modification applications for essential facilities”, Mid-America Earthquake Center Project ST-4, Final Report.
6. Eurocode 8. (2005). Design of structures for earthquake resistance. Part 3: Assessment and retrofitting of buildings. ENV 1998-3, CEN: Brussels.
7. OPCM, no.3431, May 3 2005. Official Bullettin no. 107 of May 10 2005 (in Italian).
8. Galasco A., Lagomarsino S., Penna A., Resemini S. (2004). “Non linear analysis of masonry structures”, Proc. of the 13th WCEE, Vancouver, Canada, on CD-ROM.
9. Magenes G., Della Fontana A. (1998). “Simplified Non-linear Seismic Analysis of Masonry Buildings”, Proc. of the British Masonry Society, Vol. 8, pp. 190-195.
10. Calderini C, Cattari S, Lagomarsino S. (2009). “In-plane strength of unreinforced masonry piers”. *Earthquake Engineering and Structural Dynamics*, 38(2), 243-267.
11. Mann W, Müller H. (1980). “Failure of shear-stressed masonry – An enlarged theory, tests and application to shear-walls”. Proc. of the International Symposium on Load-bearing Brickwork, London, 1-13.
12. Turnšek V, Čačovič F. (1970). “Some experimental results on the strength of brick masonry walls”. Proc. of the 2nd International Brick Masonry Conference, Stoke-on-Trent, 149-156.
13. Cattari S., Lagomarsino S.(2008). “A strength criterion for the flexural behaviour of spandrel in un-reinforced masonry walls”, Proc. of the 14th WCEE, Beijing, China, on CD-ROM.
14. Benedetti D., Carydis P., Limongelli M.P. (2001). “Evaluation of the seismic response of masonry buildings based on energy functions”. *Earthquake Engineering and Structural Dynamic*, 30, 1061-1081.