

## Seismic assessment of mixed masonry-reinforced concrete buildings by non-linear static analyses

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**Abstract.** Since the beginning of the twentieth century, the progressive and rapid spread of reinforced concrete (RC) has led to the adoption of mixed masonry-RC solutions, such as the confined masonry. However, together with structures conceived with a definite role for earthquake behaviour, the spreading of RC technology has caused the birth of mixed solutions inspired more by functional aspects than by structural ones, such as: internal masonry walls replaced by RC frames, RC walls inserted to build staircases or raising made from RC frames. Usually, since these interventions rise from a spontaneous build-up, any capacity design or ductility concepts are neglected being designed only to bear vertical loads: thus, the vulnerability assessment of this class becomes crucial. To investigate the non-linear seismic response of these structures, suitable models and effective numerical tools are needed. Among the various modelling approaches proposed in the literature and codes, the authors focus their attention on the equivalent frame model. After a brief description of the adopted model and its numerical validation, the authors aim to point out some specific peculiarities of the seismic response of mixed masonry-RC structures and their repercussions on safety verification procedures (referring in particular way to the non-linear static ones). In particular, the results of non-linear static analyses performed parametrically to various configurations representative of different interventions are discussed.

**Keywords:** mixed masonry-RC buildings; displacement based assessment; non-linear static analysis; equivalent frame model

### 1. Introduction

Past and recent earthquakes have highlighted the seismic vulnerability of masonry buildings promoting, over the centuries, the development of solutions alternative to the unreinforced one aimed at improving its properties of ductility and dissipation other than strength. One of the main driving ideas of these strategies is that of coupling other structural elements to masonry, thus leading to “mixed structures”. Actually, these solutions have followed the progressive constructive theory and technique evolution. For example, the “Pombalina gaiola” or the “Casa baraccata” systems (originally developed in Portugal and Italy, respectively), are based on the idea of reinforcing masonry through timber-framed walls; then, since the beginning of the twentieth century, the pro-

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gressive and rapid spread of reinforced concrete (RC) has led to the adoption of mixed masonry - RC solutions, such as the confined masonry. For example, in Italy, after the 1908 Messina earthquake, some regulations (Royal Italian Decree 1909) advised the adoption of masonry confined by RC frames as anti-seismic structures: the reconstruction of part of some cities, such as Messina, testifies the application of this suggestion. Nevertheless, in Italy, this explicit proposal has been neglected by successive codes, including the current one: thus its use has remained restricted to a very limited area. On the contrary, in other countries such as the South American ones (e.g. Argentina, Chile, Mexico), confined masonry finds nowadays wide application.

Despite this, together with structures conceived with a specific role for earthquake behaviour, the spreading of RC technology has caused the birth of mixed solutions inspired more by functional aspects than by structural ones. In most cases these configurations start from existing masonry buildings in which, for example: (1) RC frames have replaced internal masonry walls; (2) RC walls have been inserted to build staircases and (3) raising has been made with RC structures. The variety of all these cases poses difficulties not only for their typological classification but also for their structural scheme standardization. Moreover, since these interventions usually rise from a spontaneous building, any capacity design or ductility concepts are neglected and they are designed only to bear vertical loads. In addition it seems important to stress how, in many cases, these interventions concern not only residential buildings but also schools or hospitals.

All the previous issues make the vulnerability assessment of this class worthwhile. Nevertheless, despite its spreading in building practice, very few references to experimental campaigns (Tomažević *et al.* 1990, Jurukovski *et al.* 1992) and numerical studies (Augenti and Parisi 2009) specifically oriented to this class of mixed masonry - RC buildings may be found in the literature. Also national and international codes (e.g. Eurocode 8 2005 - Part 1 and 3 and Italian Code for Structural Design 2008 - §7.4 addressed to the seismic design of masonry structures) provide only brief recommendations not only about their structural idealization, but also on the seismic-safety criteria to be adopted. In addition, although there is a well-established background on non-linear analysis of masonry and RC (also in the case of infilled frames) structures, the set of numerical tools able to study their interaction effects is quite limited. In this regard, existing buildings pose a further fundamental issue: whereas new buildings may be designed and built to be consistent with the hypotheses which the adopted model is based on, in the case of existing ones the possibility of simulating the actual conditions of the structure represents a fundamental issue.

Among the different modelling strategies proposed in the literature, both national and international codes are oriented to masonry idealization as an equivalent 3D frame. This approach starts from the main idea (supported by the earthquake damage survey) that, referring to the in-plane response of 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. Thus, by assembling 2D walls and considering only their in-plane contribution, this approach allows one to analyse complex 3D models by performing non linear analyses with a reasonable computational effort. According to this equivalent frame idealization of masonry walls, the insertion of RC structural elements appears quite natural. Fig. 1 shows the idealization of a mixed masonry-RC structure according to this approach. Within this context, in this paper the attention is focused on the modelling solutions adopted in the Tremuri Program which has been originally developed at the University of Genoa, starting from 2002 (Galasco *et al.* 2004, Lagomarsino *et al.* 2012),

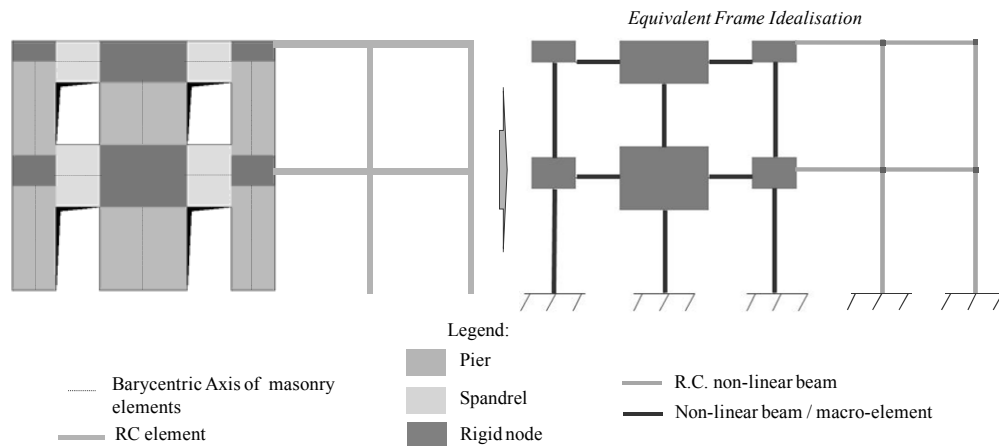


Fig. 1 Example of a mixed RC-masonry structure with the corresponding equivalent frame idealisation

and subsequently implemented in the software 3Muri (3Muri, release 4.0.5). In particular RC non-linear elements (aimed at modelling RC columns, beams and walls) have been implemented in the program (Cattari 2007).

After a brief description of the adopted model and its numerical validation, this paper aims to point out some specific peculiarities of the seismic response of mixed masonry-RC structures and their repercussions on the safety verification procedures. The results of non-linear static analyses, performed parametrically to various configurations representative of different interventions, are discussed. Particular attention is paid to the use of non-linear static procedures as a tool of verification, for which the achievement of performance-based earthquake engineering concepts has led to an increasing utilization in the last decade; in particular, this issue is faced referring to the probabilistic assessment of the ultimate limit state due to the application of the input demand of L'Aquila.

## 2. Classification of mixed masonry-rc structures

According to the previous considerations, a first classification applicable to different expected seismic responses may distinguish: (a) buildings in which masonry and RC elements are adherent leading to a cooperation between these two materials and (b) buildings in which masonry and RC do not directly interact. Within class *a*, it appears relevant to distinguish two main cases: (a1) the presence of RC ring beams at the level of floors and (a2) confined masonry. Within class *b*, several configurations particularly recurring in design practice may be identified, such as: (b1) buildings characterized by perimeter masonry walls and internal isolated RC columns or frames; (b2) masonry buildings with insertion of RC walls (typically corresponding to staircase or lift units) and (b3) masonry buildings which combine whole levels or plan area characterized by RC elements (e.g. due to plan enlargements or raising). Of course, starting from these categories, hybrid solutions are possible.

In the case of class *a*, a significant local interaction between RC and masonry elements is expected; it affects and modifies both the failure damage modes which typically occur in masonry

and the strength criteria to be adopted. However, in the *a1* case, the presence of RC ring beams mainly affects the response of masonry spandrels: so the interaction effects are quite local and may be properly taken into account by modifying only the strength criteria of these elements keeping the other hypotheses usually adopted for masonry piers unchanged. In case of confined masonry (*a2* case), the interaction effects are more significant and imply strong behaviour modifications also in masonry piers: in this case, the evaluation of internal forces is usually performed by idealizing the combined RC-masonry system as a frame with diagonal bracing aimed to simulate the masonry panel as an equivalent strut. This latter sub-class, if properly designed, is conceived to improve the seismic response of the structure, for which a reduction in vulnerability - other factors being equal - is expected.

On the contrary, in the case of class *b*, the interaction between these two structural types is a direct consequence of their differences in terms of both strength and stiffness. In particular, difference in stiffness suggests how, due to the progressing of non-linear behaviour, these structural elements should intervene in subsequent phases of the global response; moreover, it stresses the need for a proper design not only of connections but also of floors in order to guarantee their role of transferring actions among the various structural elements until building collapse. Moreover, since in most of these cases the insertion of RC elements is not conceived with a well defined role with respect to the seismic action, an increasing of building vulnerability may occur. For example, in extreme cases, case *b1* may result in brutal demolition of internal masonry walls, leading to severe weakening of the structural system if not correctly designed. In the case *b2*, in many cases repercussions on ductility and regularity are neglected causing devastating alterations to the global response (due to the consequent torsional effects). All these issues stress how the vulnerability evaluation of the class *b*, which in this paper the attention is focused on, is particularly interesting.

### 3. The equivalent frame model

As previously introduced, the adopted model (for further details see also Galasco *et al.* 2004, Lagomarsino and Cattari 2009) works according to the equivalent frame approach. It appears versatile to model some specific peculiarities of existing buildings, taking into account the effective stiffness of floors (particularly interesting in the case of wooden floors or vaults) and the limited strength of spandrel elements (very crucial in cases in which any tensile resistant elements, such as RC ring beams or tie-rods, are coupled to them). In particular, these features allow one to overcome some drastic hypotheses frequently adopted in the past (such as the case of the POR Method originally proposed in Tomaževič 1978) or by simplified models proposed in some codes (such as the “weak spandrel-strong pier” or “strong spandrel-weak pier” models proposed in FEMA 356 and 306).

Starting from the scheme of box system, the load bearing structure is strictly related to the in-plane capacity of walls and to the connection and load transfer effect due to floors. According to the equivalent frame idealization (Fig. 1), each wall is discretized by a set of masonry panels (*piers* and *spandrels*), in which the non-linear response is concentrated, connected by a rigid area (*nodes*). Rigid end offsets are used to transfer static and kinematic variables between panel ends and nodes. In the case of mixed masonry-RC structures, elements are connected only in nodes. This means that the case of confined masonry is not explicitly considered (since local interactions along elements are not modelled) and the interaction effects derive only from differences on element stiffness (thus not requiring modified failure criteria for masonry panels). Floor elements are modelled

as orthotropic membrane finite elements (with 3 or 4 nodes), in particular: normal stiffness provides a link between piers of a wall, influencing the axial force on spandrels; shear stiffness influences the horizontal force transferred among the walls, both in linear and non-linear phases. Since the model focuses only to the global building response (which is assumed to be governed only by the in-plane behaviour of walls), the local flexural behaviour of floors and the out-of-plane walls' response are not explicitly computed. Despite this assumption, it has to be highlighted that "local" out-of-plane mechanisms may be verified separately through suitable analytical methods and, in most cases, in the existing buildings, they can be inhibited through specific interventions (such as tie rod insertions). In order to assemble three-dimensional models, 3D nodes (characterized by 5 degrees of freedom) are introduced to connect walls in corners and intersections: since membrane behaviour is adopted for walls and floors, the rotational d.o.f. around vertical Z axis is neglected.

Once the masonry (or mixed masonry-RC) wall is idealized into an assemblage of structural elements, the reliable prediction of its overall behaviour mainly depends on the proper interpretation of the single element response. In the following, a non-linear beam idealization is assumed for all structural elements: thus the response is directly faced in terms of stiffness, strength and ultimate displacement capacity by assuming a proper shear-drift (or chord rotation, in the case of RC elements) relationship. Despite some unavoidable approximations of the actual behaviour (e.g. related to the mechanical description of damage and dissipation mechanisms), this simplified formulation implies the following main advantages: to perform non-linear static analyses with a reasonable computational effort, suitable also in engineering practice; to be based on a few mechanical parameters which may be quite simply defined and related to results of standard tests. Finally, it has to be stressed how this formulation agrees with the recommendations of both national and international codes (such as the Eurocode 8 and the Italian Code for Structural Design 2008, called for brevity in the following as NTC 2008).

In the case of masonry panels (modelled as 2D elements), a bi-linear relation with cut-off in strength (without hardening) and stiffness decay in the non-linear phase (for non-monotonic action) is assumed. The initial elastic branch is directly determined by the shear and flexural stiffnesses (computed on the basis of the geometric and mechanical properties of panels), then a secant stiffness unloading is assumed in the non-linear range. The redistribution of the internal forces is made according to the element equilibrium; a non-linear correction procedure of the elastic prediction is carried out based on comparison with the limit strength values as defined hereafter. The panel collapse, according to some recommendations proposed in codes (e.g. NTC 2008 and Eurocode 8), is checked by assuming a maximum drift value which varies as a function of the prevailing failure mode that occurred in the panel: in the case of existing buildings, it is assumed equal to 0.4% and 0.6% in the case of a prevailing shear and flexural responses, respectively. Once collapse is reached, the element contribution to the overall strength is only related to its capacity to bear vertical loads. The ultimate strength is computed according to some simplified criteria which are consistent with the most common ones proposed in the literature and codes for the prediction of the masonry panels strength as a function of different failure modes which may occur (such as Rocking, Crushing, Bed Joint Sliding and Diagonal Cracking). As known, these criteria are based on the approximate evaluation of the local/mean stress state produced by the applied forces on predefined points/sections of the panel, then assessing its admissibility with reference to the limit strength domain of the constituent material, usually idealized through simple models and a few mechanical parameters. Recently in Calderini *et al.* (2009) a critical review of the use and the

Table 1 Strength criteria for masonry panels implemented in Tremuri program

	Failure mechanism	Ultimate strength	Notes
Piers	Rocking/Crushing	$M_u = \frac{Nl}{0.425f_m} \left(1 - \frac{N}{lt}\right)$	$f_m$ masonry compressive strength of masonry, $l$ length of section, $t$ thickness
	Bed joint sliding	$V_{u,bjs} = l'tc + \mu N$	Mohr- Coulomb criterion with: $l'$ length of compressed section; $\mu$ and $c$ friction coefficient and cohesion of mortar joint, respectively. A limit value is imposed to $V_{u,bjs}$ to take into account the failure modes of blocks
	Diagonal cracking	$V_{u,dc} = lt \frac{1.5\tau_o}{b} \sqrt{1 + \frac{N}{1.5\tau_o lt}}$	$\tau_o$ masonry shear strength, $b$ reduction factor as function of slenderness (Turnsek and Cacovič 1970)
	Diagonal cracking	$V_{u,s} = lt\tilde{c} + \hat{\mu}N$	Mohr- Coulomb- type criterion with: $\hat{\mu}$ and $\tilde{c}$ equivalent cohesion and friction parameters, related to the interlocking due to mortar head and bed joints (such as proposed in Mann and Müller 1980)
Spandrel*	Rocking/Crushing	$M_u = \frac{dH_p}{2} \left[1 - \frac{H_p}{0.85f_{hu}dt}\right]$	$H_p$ : minimum value between the tensile strength of elements coupled to the spandrel (such as RC beam or tie-rod) and $0.4f_{hu}dt$ , where $f_{hu}$ is the compression strength of masonry in the horizontal direction
	Shear strength	$V_u = hf_{vo}$	$h$ height of spandrel transversal section

\*NTC 2008, differently from Eurocode 8, makes a distinction in the strength criteria to be adopted for spandrels as a function of the acting axial load: if known from the analysis, the same criteria as piers are assumed; if unknown, a response as equivalent strut is assumed. In Tremuri program, since the axial force computed for spandrels usually represents an underestimation of the actual one, the maximum value provided by these two cases is assumed as reference.

choice of these criteria, as a function of the masonry type examined, has been discussed. Table 1 summarizes the criteria implemented in Tremuri program for masonry piers and spandrels; as a function of the current value of the axial force ( $N$ ) acting on the element, the minimum value - as predicted by the criteria adopted to model the flexural and shear responses - is assumed as reference.

Finally, non-linear RC elements (modelled as 2D or 3D elements in the case of beams or columns and walls, respectively) are idealized by assuming elasto-perfectly plastic hinges concentrated at the ends of the element. The initial elastic branch, similarly to masonry elements, is directly determined by the stiffness contributions in terms of shear and flexural behaviour by neglecting the contribution offered by reinforcements; the reduction of stiffness due to cracking phenomena may be taken into account by simplified reduction factors which are kept constant during the analysis. Failure mechanisms are assumed as follows: (1) shear and compressive/tensile failures, as brittle failures and (2) combined axial-bending moment, modelled by plastic hinges at the end of element, as ductile failure.

Shear strength is computed according to the criteria proposed in NTC 2008 and Eurocode 8 in the case of low-medium ductility classes, as a function of different element types (beam, column and RC-wall). Both cases of transverse shear reinforcements present or not are considered; if pre-

sent, the shear strength criteria adopted are basically founded on the truss model. In particular, in case of beams and columns, according to the truss model, the shear strength is computed as the minimum value provided by the shear-tensile ( $V_{Rd,td}$ ) or shear-compression ( $V_{Rd,cd}$ ) failure modes, as provided by the following expressions

$$\begin{aligned} V_{Rd,td} &= 0,9d \frac{A_{sw}}{s} f_{yd} (\cot \alpha + \cot \theta) \sin \alpha \\ V_{Rd,cd} &= 0,9db_w \alpha_c f'_{cd} (\cot \alpha + \cot \theta) / (1 + \cot^2 \theta) \end{aligned} \quad (1)$$

where:  $\theta$  defines the angle of the inclined concrete compression strut ( $\cot \theta$  varies between 1 and 2.5);  $\alpha$  and  $A_{sw}$  are the angle and the cross-sectional area of the shear reinforcement, respectively;  $b_w$  is the web thickness;  $\alpha_c$  is a coefficient (greater or equal to 1) aimed to take into account effects of significant axial load acting on the element;  $d$  is the effective depth of the section;  $f'_{cd}$  is the reduced concrete compressive strength (obtained by assuming a reduction factor equal to 0.5).

In the case of combined axial ( $N$ )-bending moment ( $M$ ), the interaction  $M$ - $N$  domain is computed on the common hypotheses of: plane-sections; perfect bond between concrete and steel bars; rectangular stress block distribution. In case of columns, only the case of symmetrical reinforcements is considered; in case of RC walls, the domain is computed taking into account the contribution of longitudinal bars in their actual position. In order to determine the formation of a plastic hinge, the comparison between the elastic prediction and the limit values obtained from the  $M$ - $N$  interaction domain, is carried out. The case of RC walls and columns is more complex since these elements can be affected by a biaxial bending-compression behaviour. In this latter case, the  $M_x$  -  $M_y$  -  $N$  domain is traced by computing, on the basis of the axial force acting on the element, the resistant bending moments separately in each plane ( $M_{x,Rd}$  and  $M_{y,Rd}$ , respectively) and, then, by assuming a proper interaction domain (linear or with more accurate formulations, such as elliptic). It is necessary to point out that the plastic hinge, once activated, involves both X and Y planes at the same time. The collapse of the section, in the case of ductile mechanisms, is determined correlating the chord rotation (computed referring to the shear span  $L_V$ ) with the ultimate value ( $\theta_u$ ), calculated by the following expression (as proposed in the Instruction Document 2009 to NTC 2008 at §C8A.6.1)

$$\theta_u = \frac{\delta_1 \delta_3}{\delta_2 1.5} 0,016 \cdot (0,3^v) \left[ \frac{\max(0,01; \omega')}{\max(0,01; \omega)} f_c \right]^{0,225} \left( \frac{L_V}{h} \right)^{0,35} 25^{\left( \alpha_1 \rho_{sx} \frac{f_{yw}}{f_c} \right)} (1,25^{100 \rho_d}) \quad (2)$$

where:  $v$  is the axial load ratio;  $\omega$  and  $\omega'$  are the mechanical ratio of the tension and compression longitudinal reinforcements, respectively;  $f_c$  and  $f_{yw}$  are the uniaxial concrete strength and the yield strength of transverse steel, respectively;  $h$  is the height section;  $\rho_{sx}$  is the ratio of transverse steel parallel to the direction of loading ( $x$ );  $\rho_d$  is the steel ratio of diagonal reinforcement (if present);  $\alpha_1$  and  $\delta_1$  are factors aimed to taken into account the effectiveness of confinement and constructive details (like as anchorage, slip and type of bars), respectively;  $\delta_2$  is equal to 1.6 in case of RC walls and 1 otherwise;  $\delta_3$  varies as a function of the examined limit state (equal to 1 or 0.75 in the case of near collapse or life safety limit states, respectively). This expression is founded on an empirical approach based on experimental data (similarly to those discussed in FIB 2003 or proposed in Eurocode 8- Part 3 and by Panagiotakos and Fardis 2001). Once collapse is reached, for both ductile and brittle failures, the element contribution to the overall strength is only related to

its capacity to bear vertical loads. In the proposed formulation, instability phenomena and of the second order are not considered.

The next paragraph provides a numerical validation of the adopted model addressed in particular to the assessment of mixed masonry-RC structures; further validations of the adopted model may be founded in Galasco *et al.* 2004.

### 3.1 Numerical validation of the adopted model

As abovementioned, experimental campaigns specifically addressed to the seismic response evaluation of the examined mixed masonry-RC structures are limited and concern shaking table tests on reduced scale models, which, in particular, are referable to classes *b1* and *b3*, as introduced in §2 (Tomažević *et al.* 1990, Jurukovski *et al.* 1992). Despite the relevance of these experimental campaigns, they have not been used to validate the proposed model due to the following main motivations: (1) the lack of some information useful to simulate the actual condition of tests performed and (2) the almost negligible contribution to the overall strength of RC elements in the very few samples concerning the configurations aimed to analyze the intrinsic vulnerability of the classes examined in this paper. Regarding this, actually these samples concerned a configuration characterized by perimeter masonry walls and a single internal RC frame (one column and two beams), with a prevailing contribution of masonry to support seismic loads, as confirmed also by the damage occurred. As a consequence, the simulation of these tests would be more useful to validate only the reliability of masonry elements modelling (already carried out, as illustrated in Galasco *et al.* 2004) rather than the mixed masonry-RC system one. Due to all these issues, a numerical validation of the adopted model was carried out as an alternative, by comparing the results with those obtained by using other software packages.

First of all, in order to verify the reliability of RC elements that were implemented in the Tremuri program, the results of a non-linear static analysis on a RC 2D-frame are discussed. In particular, the proposed model, based on a concentrated plasticity approach (*PC Model*), is compared to that of the SeismoStruct program (Release 3.1.0), which is based on a fiber approach (*F Model*). A RC three-storey, three-bay frame has been modelled (storey to storey height 3 m). In particular, a cross section of  $0.3 \times 0.3 \text{ m}^2$  and  $0.3 \times 0.4 \text{ m}^2$  has been assumed for columns and beams, respectively. Moreover, the following reinforcements have been adopted:  $8\phi 16 \text{ mm}$ , for the longitudinal bars;  $\phi 8 \text{ mm}$  spaced  $0.2 \text{ m}$  in the critical regions, for the stirrups. The mechanical properties adopted are respectively, for concrete,  $20 \text{ MPa}$  as compression strength and, for the reinforcement,  $400 \text{ MPa}$  as yield strength. The design considers only vertical loads. Fig. 2 summarizes the results in terms of pushover curve (that is Base Shear – Node control displacement curve) and damage pattern; a load pattern proportional to the mass-height product has been adopted. Although based on very different approaches, the two models provide a quite good agreement on results with particular reference to the maximum base shear reached and the collapse mechanism activated (soft storey at ground floor). Nevertheless, the following issues have to be pointed out. In the case of the *F model*, differently from the *PC* one, the reaching of the ultimate rotation (and the consequent collapse) is not checked during the analysis but during post-processing: thus, in the *F model*, the curve decay is due to second order effects (actually it occurs close to the point of collapse made in the *PC model*). Moreover, the progressive deterioration of stiffness in the *F Model*, caused by cracking phenomena of the section, is neglected in the *PC Model*, where a reduced value of the initial stiffness has been adopted. Despite these differences, the accuracy of the *PC model* seems



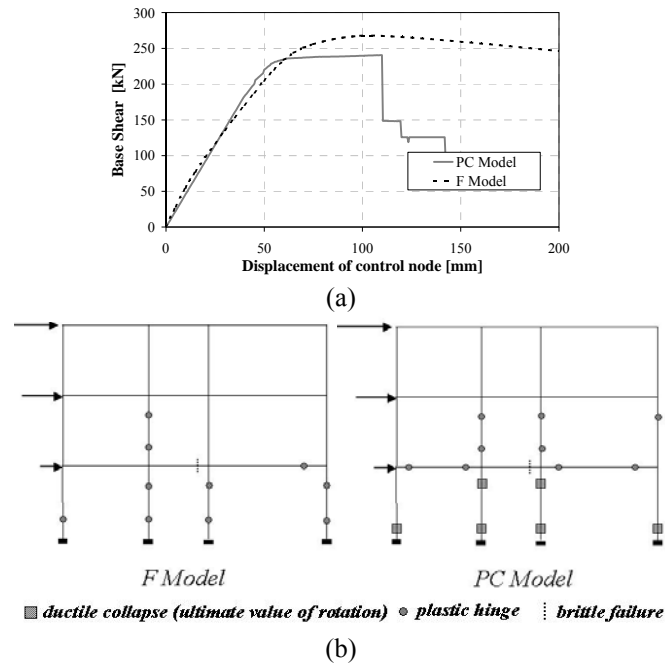


Fig. 2 Comparison between results of a pushover analysis (with a load pattern proportional to the mass-height product) on a 2D- RC frame from Tremuri and Seismostruct programs: (a) pushover curves and (b) damage patterns

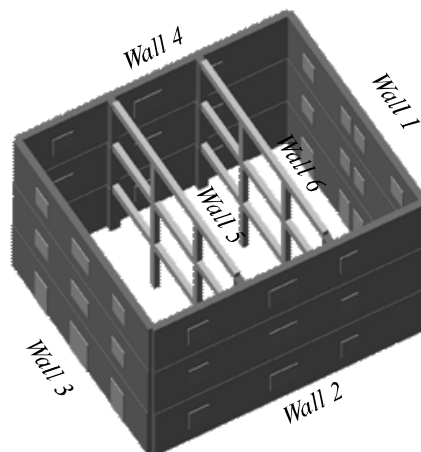


Fig. 3 View of 3D model analyzed by Tremuri and SAM II programs

appropriate if related to the purposes of the Tremuri software, that is the study of mixed masonry-RC buildings and not an accurate evaluation of RC structures.

Secondly, the response of a mixed masonry-RC building, ascribable to class *b1*, was analyzed by comparing the results obtained by Tremuri program to those carried out by using the SAM II

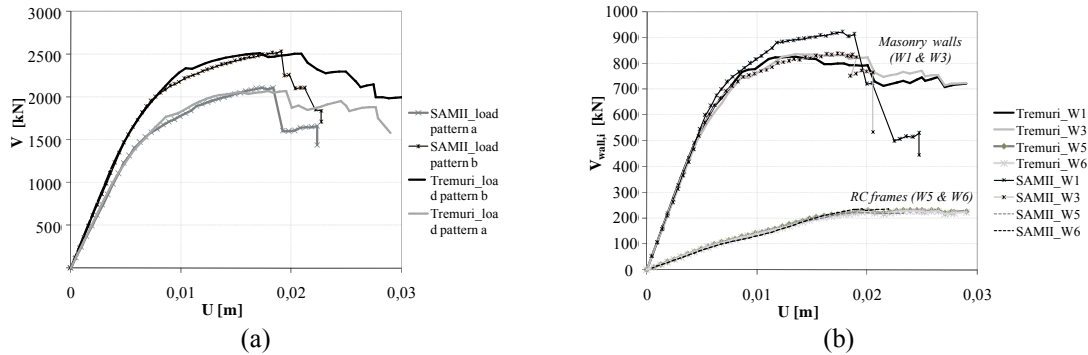


Fig. 4 Comparisons of results provided by Tremuri and SAM II programs: (1) pushover curves (X dir.- load patterns proportional to mass-height product (a) and to masses (b)); (2) shear distribution among RC and masonry walls

program (Magenes and Della Fontana 1998, Magenes 2000), which operates according to the equivalent frame approach. Fig. 3 shows a view of the 3D model analyzed.

In particular, this study was carried out in cooperation with the University of Pavia within the RELUIS DPC- 2005/ 2008 Project funded by the Italian Department of Civil Defence (Lagomarsino and Magenes 2009). Although both these software packages work within the same modelling approach, some important differences may be highlighted, in particular related to: (1) specific implementation solutions adopted to connect 2D walls (in the case of SAM II by introducing rigid beams, in the case of Tremuri by assuming a full coupling among the connected walls and by condensing d.o.f. in 3D nodes); (2) floor modelling (in the case of SAM II assumed as rigid, in the case of Tremuri modelled as orthotropic membranes) and (3) solution algorithms.

Once geometrical features and mechanical parameters are shared, non-linear static analyses were performed for each main direction (X and Y) varying the load pattern applied (proportional to masses and to mass-height product, respectively). Since in this case reinforced concrete slabs are assumed (thus quite rigid), the difference of the two models in floor modelling does not affect the results. Paying attention to the analyses performed in the X direction, Fig. 4 shows the comparison of results in terms of pushover curves. In particular, Fig. 4b shows the contribution to the global base shear distinguishing between that carried by masonry walls and that by RC frames.

The results of two models quite well agree in particular until the reaching of the maximum base shear. Actually, differences in softening phase (more sudden in case of the SAM II program) are ascribable to those in solution algorithms (in particular related to the influence of convergence precision parameters) and in computation of drift (the parameter which governs the collapse elements). Moreover, the two models also agree in terms of global failure mode, that is a soft-story mechanism located on the ground floor.

#### 4. Examples of application

In the following, the response of a complex 3D masonry structure (Fig. 5) was analyzed by the Tremuri program. In particular, for each main direction X and Y, non-linear static analyses were

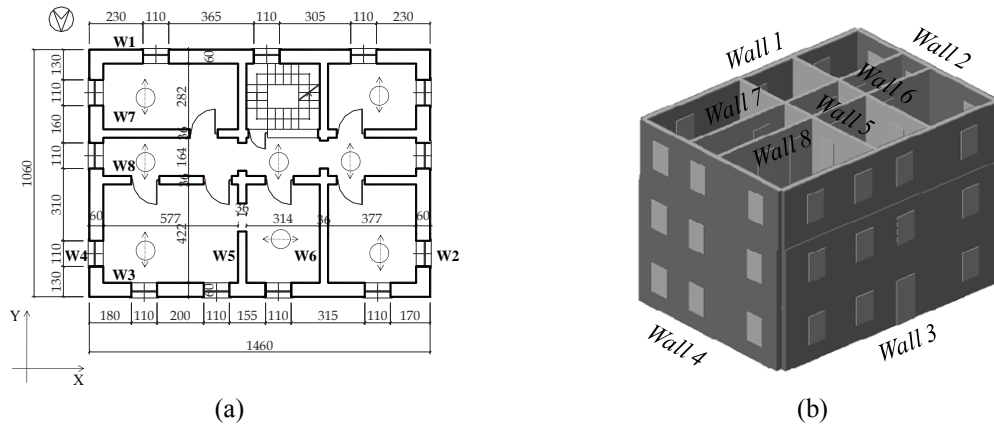


Fig. 5 (a) Ground floor plan (dimensions in cm) and (b) 3D view of the A Model

parametrically performed as a function of: different configurations aimed to simulate various interventions on existing masonry buildings ascribable to classes defined in §2; three load patterns (*uniform* that is proportional to mass, *pseudo-triangular* that is proportional to the mass and height product and *modal* that is proportional to the first modal shape, in each node respectively).

In particular, starting from the basic configuration of a three-storey unreinforced masonry building (A Model), the following interventions were simulated: B Model, presence of RC ring courses in all masonry walls (*a1* class); C Model, demolition of the internal masonry walls (with the exception of the walls delimiting the staircase) and their replacement with RC frames (*b1* class); D Model, starting from C Model, replacement of masonry delimiting the staircase with RC walls (*b2* class); E Model, raising of the A Model by mean of RC frames (*b3* class).

It is important to note that, in all these configurations, RC elements were designed to simulate existing pre-code buildings; thus only vertical loads were considered for the design neglecting any rules of capacity design (e.g. resistance hierarchy or ductility concepts). Moreover, in the case of the B Model, in presence of openings, the effective length of RC ring courses was assumed as a mean value between the opening length and the distance between two adjacent horizontally aligned nodes: actually, much more accurate evaluations (e.g. by performing non-linear FEM analyses) could be carried out to evaluate the actual restraint offered to RC ring courses by the contiguous masonry portions. Table 2 summarizes the main mechanical and geometrical parameters and loads that were adopted. In particular, the values adopted for masonry should be representative of a simple stone type for the internal walls and brick masonry for the external ones. Moreover, on the top level an additional load (equal to  $2.37 \text{ kN/m}^2$ ) was added in order to consider the mass contribution of a wooden roof.

In the next sections, the results are discussed following the two different approaches. Firstly, results are compared in terms of pushover curves in order to *qualitatively* analyze the various configurations that produce different global strength, stiffness and ductility. Secondly, issues related to the seismic safety verification are discussed paying particular attention to the use of non-linear static procedures and the performance-based assessment concepts. In particular, the probabilistic assessment of the ultimate limit state due to the application of the input demand defined for L'Aquila in the NTC 2008 is computed to *quantitatively* compare the repercussion of their differences in terms of seismic performance.

Table 2 Mechanical and geometrical parameters and loads assumed in models

Mechanical Parameters	Masonry elements [MPa] (in brackets the values of internal walls):	Young Modulus $E = 750 (900)^*$
		Shear Modulus $G = 250 (300)^*$
		Shear strength $\tau_0 = 0.07 (0.08)^{**}$
		Compressive strength $f_m = 1.95 (2.3)$
		Specific weight $w = 18 \text{ [kN/m}^3\text{]}$
		Concrete strength class : Rck 25
Geometrical parameters	Masonry walls thickness [m] (in brackets the values of internal walls)	Steel strength class: S450
		Ground and first floor = 0.6 (0.36)
		Second floor = 0.4 (0.24)
	RC elements geometry [cm <sup>2</sup> and m] (in bracket: the total longitudinal reinforcement area $A_l$ ; the area $A_w$ and spacing $s$ stirrup)	B Model 0.2x0.2 ( $A_l = 5$ ; $A_w = 1$ , $s = 0.2$ )
		C Model beams 0.45 x0.6 ( $A_l = 12$ ; $A_w = 1$ , $s = 0.15$ ) columns: 0.45 x0.45- Ground and first floor, 0.35 x0.35 second floor ( $A_l = 12$ ; $A_w = 1$ , $s = 0.15$ )
		D Model Walls: thickness 0.2 m; vertical reinf. $\phi 16/20$ ; horiz.reinf. $\phi 12/20$ with additional inclined (45°) bars at the base
Loads	Gravity loads $G$ of intermediate floors (in brackets the value of variable load $Q$ )	$G = 3.35 (2) \text{ [kN/m}^2\text{]}$
	Gravity loads of staircase (in brackets the value of variable load $Q$ )	$G = 3.78 (4) \text{ [kN/m}^2\text{]}$

\* Actual stiffness parameters assumed in cracked state (by assuming them as equal to 1/2 of elastic ones)

\*\* For masonry panels, shear response has been modelled by adopting the criterion aimed to interpret the Diagonal Cracking failure mode in case of that proposed by Turnsek and Cacovič (1970).

### 3.1 Comparison of results in terms of pushover curves

Fig. 6 shows the comparison between the analyses performed with *pseudo-triangular* load pattern in terms of  $V/W$  (that is total base shear  $V$  normalized to the total building weight  $W$ ) versus  $U_{roof}$  (average displacements of 3<sup>rd</sup> level) curve: this represents a kind of normalized pushover curve aimed to standardize the different configurations examined. The *pseudo-triangular* load pattern has been adopted as reference to discuss in a *qualitative* way the different behaviours occurred. Despite some unavoidable peculiarities of each load pattern, most of the following remarks have, in fact, a “general” meaning: nevertheless, where particularly relevant, specific comments are also included on the other patterns. Pushover analyses were stopped at the step corresponding to 20% decay of the maximum base shear reached: analogously to criteria proposed in some codes (Eurocode 8, NTC 2008), this condition was assumed as reference to define the ultimate displacement capacity of building. In particular, it is assumed as representative of the ultimate limit state corresponding to the life safety condition.

Firstly, the response of the A Model is analyzed. The analyses performed in the Y direction are more punitive than those along the X axis: actually, due to the floor orientation (see Fig. 5), the strength of piers oriented according to the Y axis is more penalized because of the lower initial axial force. For this configuration, a “uniform” global mechanism occurs characterized by a first lo-

calization of the damage predominantly on spandrels followed only in the final phase by the collapse of piers. Actually, due to the moderate axial load acting on spandrel elements and the absence of other tensile resistant elements coupled to them, their resulting strength is very low: since the initial steps of the analysis a *Rocking* mechanism occurs in almost all spandrels which thus supply a very weak coupling for piers.

In the case of the B Model, as expected, the presence of RC ring courses coupled to spandrels (for which the activation of an equivalent strut response is assumed) significantly affects the global response. However, in addition to a significant base shear increase, a strong reduction in the global ductility occurs: in fact, as shown in Fig. 7a, RC ring courses produce a kind of confinement of the structure increasing the demand in terms of both inter-storey drift and soft storey mechanism. In this regard, it is worth noting that a design aimed at promoting a “uniform” global mechanism, like that of the A Model, would be advisable for many reasons: it agrees with the “capacity design” criterion; it complies with the concept of “sustainable repair”; finally, experimental campaigns have pointed out that damage to spandrels produces a more significant energy absorption than that to piers (Benedetti and Magenes 2001). Moreover, due to the more effective coupling among piers associated to the B rather than the A configuration, boundary conditions of piers move to the fixed-fixed conditions: due to this, differently from the A Model, a predominance of shear failure occurs (a fact that contributes to explain the global ductility reduction, since a lower value of drift limit is associated to shear modes). Moreover, the B Model allows further considerations about the consistence between design rules and hypotheses related to models adopted in the past (e.g. the POR Method or the simplified “strong spandrel-weak pier”). In particular, the aforementioned models assume that floors are rigid and piers crack first, thus averting the failure of spandrels, which are usually assumed as infinitely stiff portions ensuring a complete coupling between them: indeed, the actual modelling of spandrels is useless. Usually in the past, in design practice, the consistence of these hypotheses was a-priori assumed by ensuring it, in the case of existing buildings, making in many cases interventions such as replacement of flexible wooden floors by RC slabs or insertions of RC ring courses coupled to spandrels. In some cases, as known, the earthquake damage surveys showed disastrous outcomes associated with their inadequate execution. Fig. 7b shows the comparison between A and B Models and the “shear-type” limit condition (B’ Model) assured in the B Model by restraining rotations of all nodes. Actually, this comparison shows how the B’ Model provides an upper bound which operates on the unsafe side (with a strength overestimation of 15-20% compared to the B Model): this is remarkable in particular in case of seismic verification procedures which are based only on a strength approach.

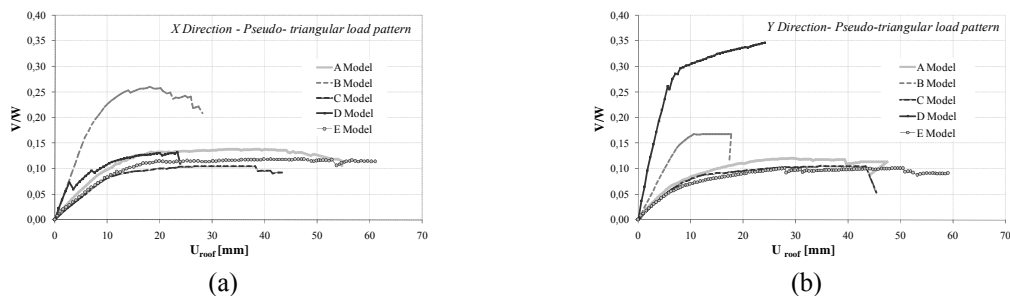


Fig. 6 Pushover analyses with pseudo- triangular load pattern, results on: (a) X direction and (b) Y direction

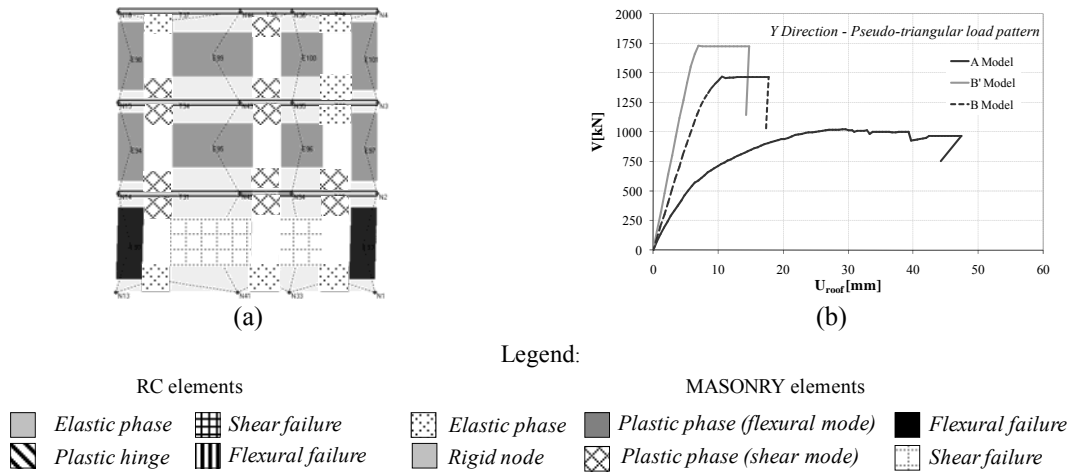


Fig. 7 B Model pushover analysis in Y direction with pseudo-triangular load pattern: (a) damage pattern at collapse and (b) comparisons between B model and the limit configuration assumed by POR method

In the following, the responses of C and D models are analyzed.

The C Model highlights, as expected, a reduction in global strength (Fig. 6). The analysis of progressing non-linear response shows a first phase in which RC elements are elastic and the masonry ones progressively reach their maximum resistance depending on their actual axial force. Finally, in this case, the further increase of resistance, due to RC elements, which are progressively involved in the non-linear phase, is really moderate because of the assumptions concerning their design (in many cases columns collapse precedes that of beams).

In the case of D Model, a considerable increase of the initial stiffness occurs due to the presence of RC walls (Fig. 6). Since the two RC walls of a staircase without openings are oriented along the Y axis, the beneficial increase of base shear is more pronounced in this direction. In this case, it is possible to recognize essentially two main phases in the seismic response (Fig. 8). Firstly, RC walls govern the response until they reach their ultimate flexural strength (*Phase I*); then, during *Phase II*, the masonry walls also contribute to the further increase of base shear. Finally, the global collapse (*Phase III*) is caused by the occurrence of a brittle shear mechanism at the base sections of RC walls: as a consequence, the global ductility is smaller than that of the A Model. This latter outcome is even more pronounced in the case of modal load pattern (as discussed also in §4.2). Moreover, in the case of the X direction, it is possible to note some sudden decay in base shear with it subsequent recovery (see Fig. 6a). Also in this case, these phenomena are associated with the differences in stiffness between RC and masonry elements and to their loading-up in subsequent phases. This fact suggests some remarks on the criteria to be adopted to define the ultimate displacement capacity of the structure: in fact in the case of mixed masonry-RC structures the possibility of having strength recovery even after a base shear decay greater than 20% appears plausible.

Finally, in the case of the E Model, the most relevant issue is related to the strong influence of the load pattern applied on the global response. As shown in Fig. 9, the approximation of the modal pattern by that proportional to the mass-height product (as suggested in some cases by codes) could lead to results which are on the unsafe side: this is due to the strong irregularity in

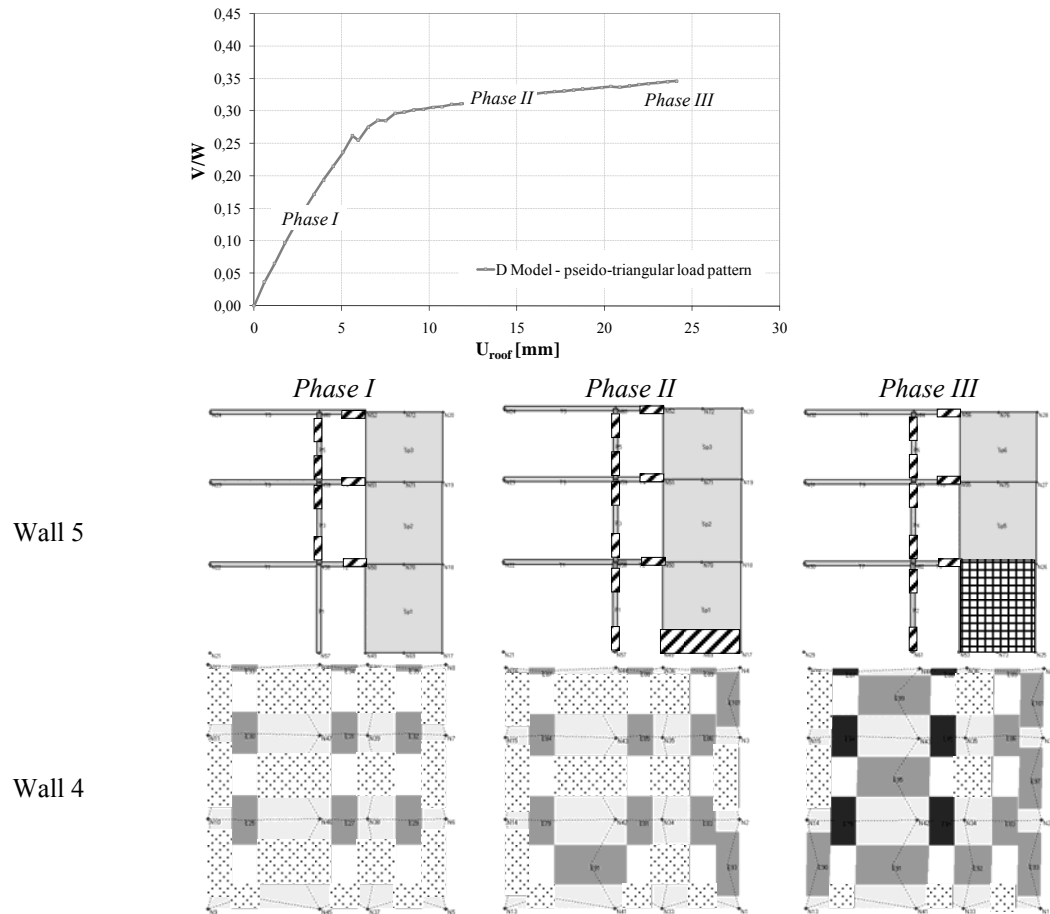


Fig. 8 D Model pushover analysis in Y direction with pseudo-triangular load pattern: pushover curve and damage pattern sequence which progressively occur in RC and masonry elements (for the damage legend see Fig. 7)

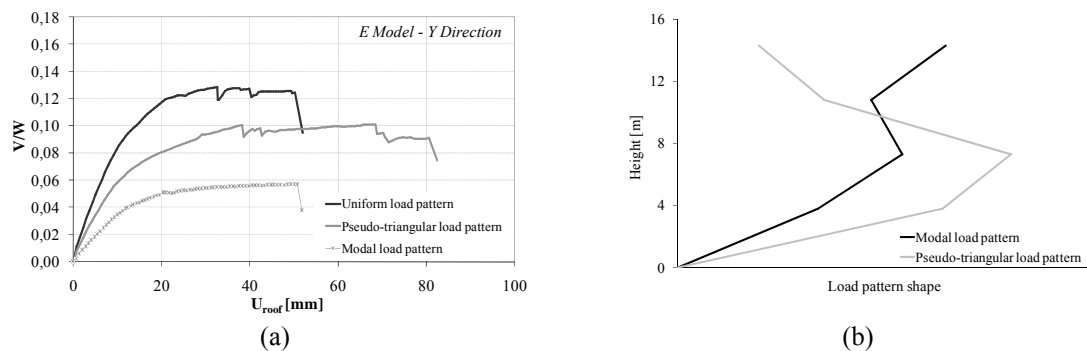


Fig. 9 E Model - pushover analysis in Y direction: (a) load pattern effect on the global response and (b) comparison between the pseudo-triangular load pattern and the modal one (patterns are normalized such as the seismic total force is equal in two cases)

elevation associated to raising by RC frames.

With reference to the load pattern to be adopted, it might also be useful to investigate the use of adaptive or multi-modal pushover analysis. According to the adaptive approach, at each step of the analysis, the load pattern is updated as a function of the progressing of the non-linear response that occurred in the structure; in the last decade, many different proposals have been proposed in the literature (e.g. by Antoniou and Pinho 2004, Chopra *et al.* 2004 and Gupta and Kunnath 2000). However, it is worth noting that, whereas for other structural types (such as RC or steel buildings) the application of this procedure has been quite widely investigated, in the case of masonry structures very few references may be found in the literature (Galasco *et al.* 2006).

### 3.2 Use of non-linear static procedures as seismic safety verification

In the following, the comparison of results discussed in the previous section is revised by applying the concepts of non-linear static procedures as seismic safety verification. The range in pushover curves obtained for models from A to E is quite wide in terms of strength, stiffness and ductility: since all three of these aspects play a fundamental role in non-linear static procedures, some remarks previously introduced could be overturned.

#### 3.2.1 General issues on the use of non-linear static procedures in case of mixed RC-masonry buildings

In the last decade, the achievement of performance-based engineering concepts has led to an increasing utilization of non-linear static procedures in evaluation of the seismic performance of buildings. Actually in the case of mixed structures, such as masonry-RC buildings, for which response is strongly affected by these elements' interaction in the non-linear phase, these procedures seem the standard ones to be adopted.

As known, the focus of these procedures is the identification of the "performance point" (or target displacement), computed from the intersection between the capacity curve and the elastic spectrum properly reduced; the capacity curve is obtained by properly converting the pushover curve representative of the non-linear response of the original multi degree of freedom (MDOF) into that of an equivalent SDOF system. Among the different approaches proposed in the literature, the criteria adopted in both Eurocode 8 and NTC 2008 are assumed as reference; they basically refer to the N2 Method originally proposed in Fajfar (2000) and, as known, based on the use of inelastic spectra.

In particular, in order to clarify some notation and results discussed in §4.2.2, Table 3 summarizes the criteria adopted to: (1) convert the original pushover curve in an equivalent SDOF and (2) evaluate the performance point (called as  $d_{\max}^*$ ). Regarding point a), according to Eurocode 8 and NTC 2008, an elasto-perfectly plastic law has been assumed. Despite this general recommendation, some arbitrariness in the rules to be adopted to define this bilinear is allowed: Table 3 clarifies those specifically assumed in the paper. In particular, regarding the condition aimed to define the ultimate displacement capacity ( $d_u$ ), before regarding the step which corresponds to 20% maximum base shear decay ( $V_{\max}$ ) as decisive to determine  $d_u$ , it is proposed by the authors, in case of mixed masonry-RC structures, to verify whether or not during the next steps of the analysis a recovery in the global strength occurs such as to allow reaching or overcoming of the previous recorded maximum value of  $V_{\max}$ . This criterion is founded on the abovementioned considerations concerning the phases of the global seismic response, with different contributions of RC and masonry elements. Fig. 10 summarizes: (1) the basic idea of the identification of performance point



Table 3 Criteria adopted to define the equivalent SDOF and compute the target displacement

Adopted criteria	
Transformation to an equivalent SDOF	<p><b>Idealized elasto-plastic curve</b></p> <p>i) the ultimate displacement capacity (<math>d_u</math>) is assumed corresponding to 20% decay of the maximum base shear reached (this recommendation is proposed both in Eurocode 8 and NTC 2008);</p> <p>ii) the initial stiffness is defined by imposing the intersection with the point of pushover curve on the initial branch which corresponds to 70% of maximum base shear reached;</p> <p>iii) the yield force (<math>F_y</math>) is determined in such a way that the areas under the actual and the idealized force-displacement curve are equal.</p>
	<p><b>Conversion from MDOF to SDOF system</b></p> <p>The <math>\Gamma</math> factor is applied to both <math>d</math>-displacement and <math>V</math>-shear base values (<math>d^* = d / \Gamma</math>, <math>F^* = V / \Gamma</math>). In particular <math>\Gamma</math> is computed as</p> $\Gamma = \frac{\sum m_i \psi_i}{\sum m_i \psi_i^2} = \frac{m^*}{\sum m_i \psi_i^2}$ <p>where respectively: <math>m_i</math> is the mass in the <math>i</math>-th storey and <math>\psi_i</math> is the component of the assumed displacement shape (like, for example, the first modal shape) <math>\psi</math> (in which the components are normalized in such way that <math>\psi_n = 1</math>, where <math>n</math> is the control node, usually the roof level).</p>
Determination of the target displacement	<p>The target displacement <math>d_{\max}^*</math> is computed as (according to the use of inelastic spectra):</p> $d_{\max}^* = d_{e,\max}^* = \left( \frac{T^*}{2\pi} \right)^2 S_e(T^*) \quad \text{for } T^* > T_c$ $d_{\max}^* = \frac{d_{e,\max}^*}{q^*} \left[ 1 + (q^* - 1) \frac{T_c}{T^*} \right] \geq d_{e,\max}^* \quad \text{for } T^* \leq T_c$ <p>where respectively: <math>T_c</math> is the corner period between the short and medium period range; <math>T^*</math> is the period of the equivalent SDOF; <math>q^*</math> is the ratio between the acceleration in the structure with unlimited elastic behaviour <math>S_e(T^*)</math> and in the structure with limited strength <math>F_y^* / m^*</math>. A limit value of 3 is proposed for <math>q^*</math> in both Eurocode 8 and NTC 2008.</p>

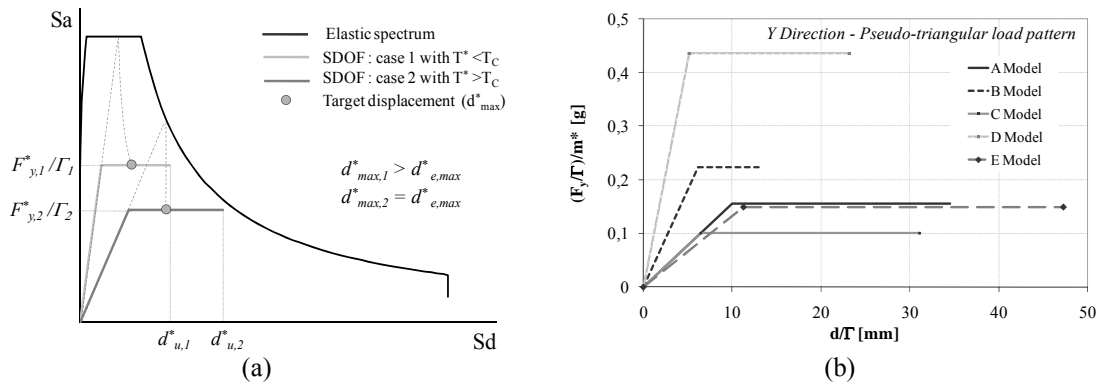


Fig. 10 (a) Identification of performance point by non linear static procedure and (b) Resulting idealized SDOF curves for the examined cases

by the non linear static procedure adopted and (2) the resulting idealized SDOF curves for the examined configurations in the case of Y direction with pseudo-triangular load pattern.

As known, according to this non linear procedure, the safety verification consists of checking that the target displacement ( $d_{\max}^*$ ) is lower than the ultimate displacement capacity ( $d_u^*$ ). Despite this general criterion, a relevant difference has to be noticed between RC and masonry structures in the definition of  $d_u^*$  and, consequently, in the use of non-linear static procedure as a tool for verification (as proposed both in Eurocode 8 and NTC 2008). In fact, in the case of masonry buildings,  $d_u^*$  is defined on heuristic basis through criteria defined at global scale, directly on the capacity curve (e.g. in terms of a conventional percentage of the overall base shear decay): thus, it is evident that in correspondence of  $d_u^*$  (but in general also of  $d_{\max}^*$ ) more than one panel exceeds the drift limit value. Regarding this, it is usually assumed that the reaching of the drift limit value compromises only the lateral load bearing capacity of elements and not the one related to the vertical load. On the contrary, in case of RC buildings, it is requested that in correspondence of  $d_{\max}^*$  no element reached the ultimate condition: actually, all this results in adopting a structural element approach to define  $d_u^*$  on the capacity curve (since it corresponds to the point where the first element collapsed). This criterion seems consistent in case of RC existing buildings, particularly in those where any capacity design criteria are neglected and it is plausible that collapse is reached almost simultaneously in several elements. On the contrary, in case of masonry buildings, it is common to have a marked softening phase with structural elements that progressively reach the ultimate conditions. Even more in the case of mixed masonry-RC structures, the limit conditions may be reached in structural elements for different values of the control node displacement, due to relevant differences in stiffness. Thus, in this case, provided that elements are still able to support vertical loads (that is are far from the collapse condition), it seems more reasonable to adopt an heuristic approach likewise that of masonry buildings. Coherently with this assumption, the structural check of RC elements, in terms of reaching of ultimate conditions, is directly made in each step of the incremental analysis by the Tremuri program: in particular, for the computation of the chord rotation  $\delta_3$  in Eq. (2) it is assumed equal to 0.75, consistent with the life safety limit state.

### 3.2.2 Probabilistic assessment of the ultimate limit state

In order to compare the seismic performances of the different analyzed configurations, once a seismic input is fixed as reference, the probability of exceeding the ultimate limit state was evaluated by using the fragility curve concept. As known, fragility curves are defined by lognormal functions that describe the probability of reaching, or exceeding, a defined damage state, given deterministic (median) estimates of spectral response (for example the spectral displacement); the variability and uncertainties associated with capacity curve properties, damage states, model errors and ground shaking should be properly taken into account.

In particular, the conditional probability  $P[ds | S_d]$  of being in, or exceeding, a particular damage state ( $ds$ ), given the spectral displacement  $S_d$ , is defined by the following expression

$$P[ds | S_d] = \Phi \left[ \frac{1}{\beta_{ds}} \ln \left( \frac{S_d}{\bar{S}_{d,ds}} \right) \right] \quad (3)$$

where:  $\Phi$  is the standard normal cumulative distribution function;  $\beta_{ds}$  is the standard deviation of the natural logarithm of spectral displacement for damage state  $ds$ ;  $\bar{S}_{d,ds}$  is the median value of spectral displacement at which a building reaches the threshold of damage state  $ds$ . For the aims of the proposed application, a single damage state corresponding to the ultimate limit one is assumed as reference, defined on the capacity curve by  $d_u$ ; moreover, the spectral displacement  $S_d$  is

assumed equal to the target displacement corresponding to the performance point  $d_{\max}$ . Thus, the probability of exceeding the ultimate limit state is directly computed as  $P[d_u | d_{\max}]$ . Regarding the definition of  $\beta_{ds}$ , as previously introduced, this parameter summarizes the variability and uncertainties associated with different factors; in particular, it may be expressed as

$$\beta_{ds} = \sqrt{\beta_C^2 + \beta_D^2 + \beta_{ls}^2 + \beta_\epsilon^2} \quad (4)$$

where respectively:  $\beta_C$  represents the variance related to the variability of the capacity curve (i.e., related to that of the mechanical or geometrical parameters which affect the global response);  $\beta_D$  represents the variance related to the variability associated to the seismic demand;  $\beta_{ls}$  describes the variability of the threshold of damage state and  $\beta_\epsilon$  summarizes the model error.

As an example, for the application proposed, the seismic input defined in the NTC 2008 for L'Aquila with probability of exceedance of 10% in 50 years and return period equal to 475 years was assumed as reference. Fig. 11 shows the comparison between this elastic response spectrum (B soil category), and that computed as mean of the actual main shock recordings carried out on the stations located within a 15 km radius from the epicentre during the 2009 L'Aquila earthquake.

It is worth noting that, in addition to the mean value of the elastic spectrum (related to the 50<sup>th</sup> percentile), the Italian hazard maps provided by INGV also propose values associated to the 16<sup>th</sup> and 84<sup>th</sup> percentile (as plotted in Fig. 11). Moreover, in Fig. 11, two dotted lines delimit the range of periods associated with the equivalent SDOF curves of all the configurations examined.

Once the seismic demand is defined, performance points were obtained by applying the non-linear static procedure described in paragraph 4.2.1 for all the performed parametric analyses.

Regarding the value assumed for  $\beta_{ds}$ , the following assumptions were taken:  $\beta_C$  was assumed equal to 0.2;  $\beta_\epsilon$  was assumed equal to 0.25 on a heuristic basis;  $\beta_D$  was computed by applying the Response Surface Technique by evaluating, for each configuration, performance points consequent to the comparison with the 16<sup>th</sup> and 84<sup>th</sup> percentile of demand;  $\beta_{ls}$  was computed by considering a discrete damage state distribution like that proposed in Pagnini *et al.* 2011. As regard  $\beta_C$ , it is worthy stressing that, differently from the case of vulnerability analyses performed on whole classes of buildings with homogeneous behaviour, a not so high variability is expected at scale of the single asset: due to this, the value adopted is quite low (although consistent with reference values proposed in the literature, such as in HAZUS 1999). As regard  $\beta_D$ , the resulting mean value is 0.078; the scatter of values obtained is very low due to the uncertainty on the demand which is weakly affected by the period variation (see Fig. 11). In the paper, the input demand, as actually proposed in NTC 2008 for L'Aquila, has been conventionally assumed; in general, in the case of a more detailed seismic hazard analysis, a wider scatter would be expected. As regard  $\beta_{ls}$ , it is assumed that limit state displacement thresholds ( $\bar{S}_{d,ds}$ ) correspond to the conditional probability of 50% of being in or exceeding the corresponding limit state. Thus, by assuming a uniform probability density function (in an interval around  $\bar{S}_{d,ds}$ ), the resulting value of  $\beta_{ls}$  varies, for each configuration, as a function of the ductility of the capacity curve; it corresponds to a mean value of 0.098. Other uncertainties on the definition of performance levels (e.g. related to the criterion adopted to define  $d_u$  based on a conventional value of the overall base shear decay) are included in  $\beta_\epsilon$ . Since  $\beta_{ls}$  and  $\beta_D$  are quite low, their variation as a function of each specific configuration does not so significantly affect the resulting values of  $\beta_{ds}$ : as a consequence,  $\beta_{ds}$  has been assumed constant and equal to

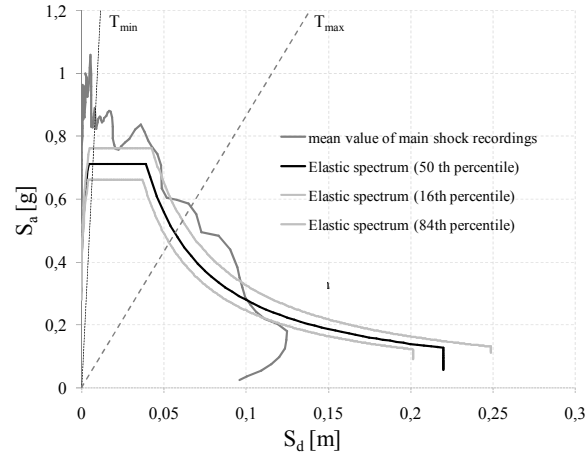


Fig. 11 Comparison between the elastic spectrum defined in the Italian Technical Code for L'Aquila (soil B, 475 years of return period – with 50<sup>th</sup>, 16<sup>th</sup> and 84<sup>th</sup> percentile respectively) and that obtained by the actual main shock recordings

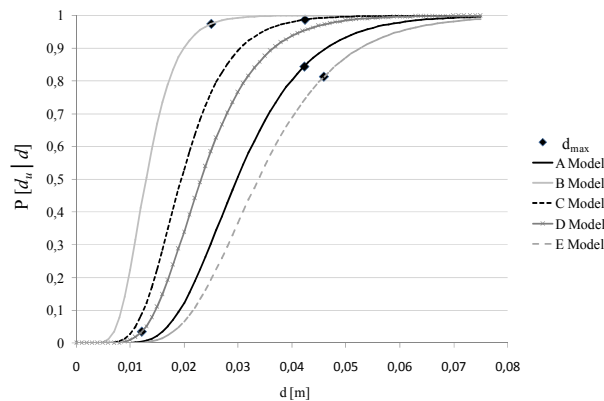


Fig. 12 Resulting fragility curves for different models analyzed (Y dir.- pseudo-triangular load pattern)

0.344. Fig. 12 illustrates the resulting fragility curves in case of the evaluation on pushover curves obtained by applying a pseudo-triangular distribution in Y direction.

Finally, Fig. 13 summarizes the results obtained in terms of probability of exceeding the ultimate limit state; moreover, for each case the corresponding available ductility (computed as the ratio between  $d_u$  and  $d_y$  on the idealized bilinear curve) is illustrated.

It can be stated that in general the adoption of a modal load pattern seems more reliable: as expected, this is true in particular in case of strong irregularities, such as the case of E Model.

Moreover, with respect to the basic configuration of the A Model, it can be observed that:

- *in the case of the B Model.* The rise of total base shear, coupled to a remarkable reduction in ductility (due to the activation of a soft storey mechanism), turns out in an increasing of vulnerability with respect to the A Model. Although this result cannot be generalized, it highlights the importance of considering effects on both strength and ductility, in case of strengthening intervention design. Regarding this, in some cases, it is possible to adopt other traditional interventions, such as the insertions of steel tie-rods.

- *in the case of the C Model.* The higher vulnerability of this class, when design of RC frames is inappropriate, is confirmed.

- *in the case of the D Model.* Focusing the attention on results in the Y direction (in which the effect due to the introduction of RC walls is more remarkable), the significant increase in strength results in a general decrease of vulnerability; the higher probability associated to the modal load pattern – even lower than that of the A Model- may be explained by the strong reduction in global ductility due to a sudden brittle shear failure that occurred at the base of RC walls. In general this phenomenon was noticed in all three applied load patterns (even particularly sudden in the case of modal one). This result suggests how, in order to better exploit all the benefits associated with the insertion of RC walls, technological solutions (such as selective weakening approaches proposed for the seismic retrofit of RC buildings in Bull *et al.* 2006) should be investigated in order to promote ductile failure mechanisms. Finally, in case of insertion of structural elements much stiffer than masonry ones, particular attention has also to be paid to the consequent decreasing of the initial period. In fact, due to the intersection with a more punitive region of the spectrum, if the resulting structural system is unable to afford the required ductility, the beneficial effect on the strength increase risks being useless. Although not so emphasized by the configuration examined, it seems important to stress how particular attention also has to be paid in design to the potential torsion effects associated to the insertions of RC walls.

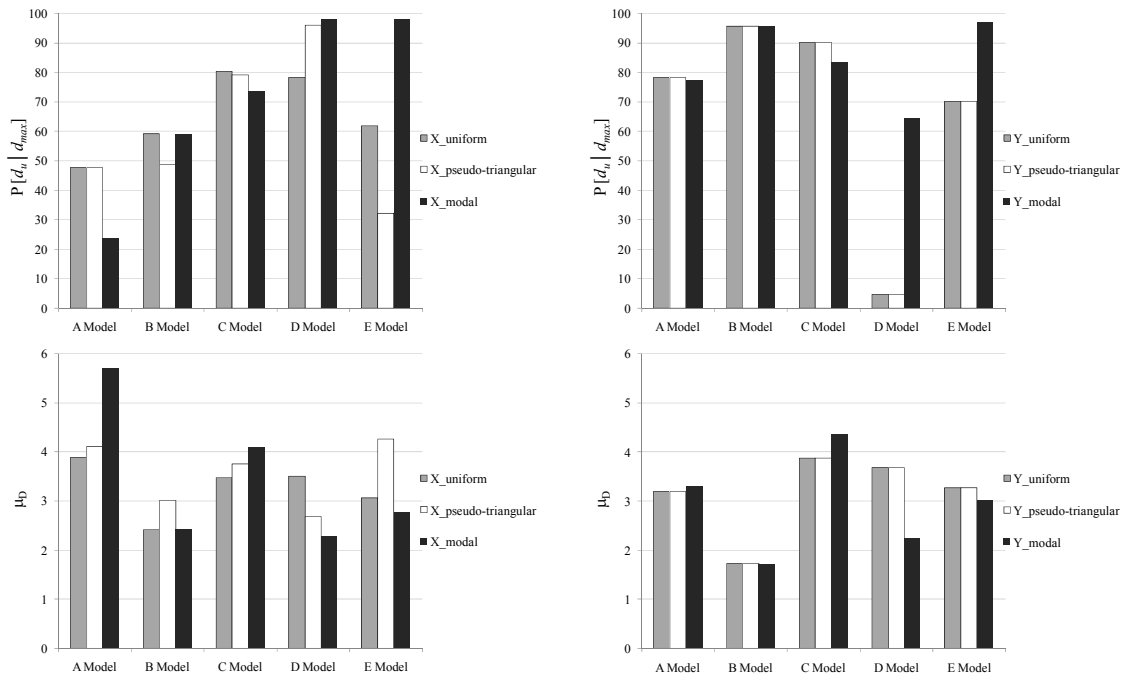


Fig. 13 Probabilities of exceeding the ultimate limit state and available ductility values (without considering any limitation on  $q^*$ )

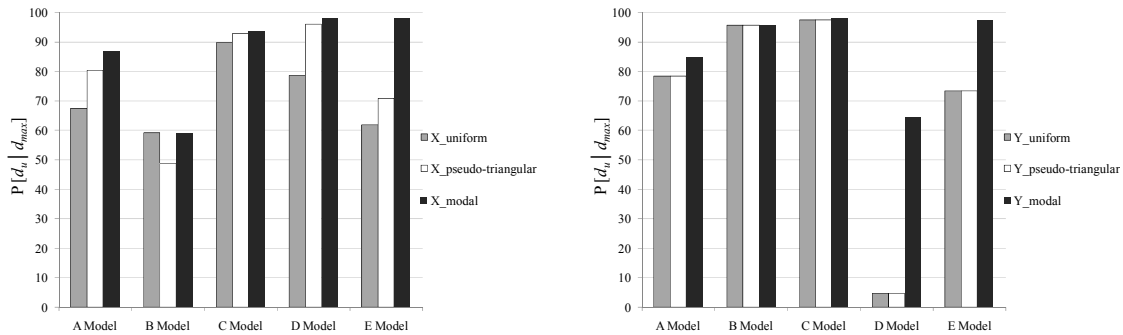


Fig. 14 Probabilities of exceeding the ultimate limit state computed considering the limitation on  $q^*$

- *in the case of the E Model.* This case (together with D Model) stresses the high sensitivity of the seismic response on the load pattern in case of significant irregular stiffness distribution. In particular, an increase of vulnerability than the A Model is highlighted by the modal pattern. However, due to this strong irregularity in elevation, even the adoption of the modal pattern could not be sufficient to provide a reliable evaluation of the actual vulnerability.

Finally, it seems useful remarking that some codes propose a limitation of the full available ductility provided by numerical analyses: Eurocode 8 and NTC 2008 propose a limitation of  $q^*$  value equal to 3. Fig. 14 shows how probability values may change taking into account this recommendation.

#### 4. Conclusions

In this paper the seismic performance of various configurations of mixed masonry-reinforced concrete structures has been analyzed and compared through non-linear static analyses, by adopting the equivalent frame model. The modelling tool employed is particularly effective to assess the effects of structural interventions, which are widely adopted in engineering practice but, in many cases, without an actual quantitative evaluation. The application of non-linear static procedures as a tool of verification has highlighted how, according to performance-based design concepts, it is important to consider the effects of these interventions on the global seismic response in terms of both strength and ductility. In particular, in case of insertion of RC walls, the results obtained suggest the need for further improvements in design of constructive details aimed to promote their ductile failure mechanisms. Moreover, in the global balance of the seismic capacity of a masonry buildings, it seems better to promote “uniform” mechanisms, by allowing damage in spandrels, more than strongly strengthen and stiffen them by producing soft-storey failures. This criterion, other than agree with the “capacity design” usually adopted for other structural types (like as steel and RC structures), complies with the concept of “sustainable repair”.

Finally, some configurations stressed a high sensitivity of the response to the load pattern adopted: in particular, in those cases where irregularities on stiffness distribution in plan or elevation have been introduced (D and E Models). Although in this paper, only the load patterns more commonly adopted in the engineering practice have been analyzed, more accurate approaches - like the adaptive or multimodal ones- are needed.

## Acknowledgments

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