A STRENGTH CRITERION FOR THE FLEXURAL BEHAVIOUR OF SPANDRELS IN UN-REINFORCED MASONRY WALLS

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ABSTRACT :

With reference to the in-plane behaviour of masonry walls, it is possible to recognize basically two structural components: *piers* and *spandrels*. Piers are the main vertical resistant elements for both dead and seismic loads. Spandrels are usually classified as secondary elements: although hardly investigated at all in literature (unlike piers), they significantly affect the seismic capacity of the structure. Firstly, a considerable energy dissipation is related to their damage. Moreover, the coupling effectiveness associated with these elements significantly influences the boundary conditions of piers (i.e. fixed-fixed or cantilever) with remarkable repercussions on the global response of the structure. In the case of existing buildings, common practice (explained by the frequent lack of other tensile resistant elements coupled) is to neglect masonry spandrels in the model leading to a large number of historical buildings being assessed as "unsafe" according to current seismic codes. It is clearly evident that such a result is not plausible: the adopted hypothesis is too severe and masonry spandrels supply unknown resources to the structure. Thus reliable predictive models are needed. In the paper a resistance criterion finalized to interpretation of their flexural behaviour is proposed. It has basically been founded on the interlocking phenomena which can be originated at the interface between the end-sections of the spandrel and the contiguous masonry. A set of parametrical non linear analyses has been performed in order to validate the proposal. Finally some applications to complex masonry walls are presented in order to assess the effects on the global response.

KEYWORDS: *masonry, spandrel strength, in-plane response, simplified models, seismic capacity*

1. INTRODUCTION

The large population of existing and historical un-reinforced masonry buildings all over the world points to the need to improve the knowledge of their seismic behaviour (because of their social necessity, economical role or historical value), 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 (that is able to achieve a satisfactory safety level), accurate numerical models to predict the response of the structure are essential. The possibility of simulating the actual conditions of the structure represents a crucial issue: in fact models usually employed for new constructions are not always equally suitable for existing ones.
Focusing the attention on the global seismic verification criteria, with particular reference to the non linear static procedures, the modelling strategy based on the idealization 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 (Eurocode 8, OPCM 3431/05). 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. Each resistant wall is discretized by a set of masonry panels in which the non-linear response is concentrated. Two types of panels are distinguished: "piers", which are the main vertical resistant elements for both dead and seismic loads; "spandrels", which are the secondary horizontal elements, coupling piers in the case of seismic loads. The spandrels significantly affect the boundary conditions of piers (i.e. fixed-fixed or cantilever) with great repercussions on prediction of their load-bearing capacity. Thus pier models, also very accurate but associated with mistakes in the definition of their boundary conditions (due to an unsatisfactory

knowledge of spandrels), can lead to unrealistic prediction of the global response of the wall. Despite this, in the past research programs have almost entirely been focused on piers (FEMA 307 collects some results of international experimental campaigns) and almost nothing on spandrels (Calderoni et al. 2007).
Moreover, the frequent adoption of very simplified models (also suggested by international codes such as FEMA 356), such as "strong spandrel-weak pier" or "weak spandrel-strong pier" types, makes the modelling of spandrels un-requested. The "strong spandrel-weak pier" model assumes that piers crack first, thus averting the failure of spandrels. As a general rule, this assumption is consistent with new buildings in which masonry spandrels are always connected to lintels, tie-beams and slabs made of iron or reinforced concrete. These elements, being stiff and tensile resistant, assure a consistent coupling between piers, making the contribution of masonry negligible. On the contrary, in historical buildings spandrels are intrinsically weak elements (lintels are usually made of wood or masonry, tie-beams are often not present, wooden floors are flexible). Thus according to the "weak spandrel-strong pier" model, the hypothesis of both null strength and null stiffness of spandrels is often adopted and the piers are assumed as uncoupled. 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. Even referring to much more complex models, such as "equivalent frame", spandrels are usually modelled as piers rotated to 90° adopting the same failure criteria. Due to the low values of axial load acting on spandrels, this assumption leads to an unrealistic dominance of the flexural failure. It is worth noting that the boundary conditions of spandrels are very different from those of piers, in particular due to the interlocking with the contiguous masonry regions. This last observation suggests that transposing the experimental results of piers to the spandrels without modifications can be inconsistent. All the aforesaid issues lead to the result that a large number of historical masonry buildings are assessed as "unsafe" according to current seismic codes. It is clearly evident that such a result is not plausible and that, therefore, the adopted hypotheses are too severe: masonry spandrels supply relevant unknown resources to the structure.
Finally, it is worth noting that a design aimed at promoting a "uniform" global mechanism (that is with a first localisation of the damage predominantly on spandrels and with a subsequent collapse of piers only in the final phase) would be advisable for many reasons:

- It is in agreement with the "capacity design" criterion. This design strategy widely adopted for other structural typologies such as r.c. or steel buildings, leads to a better exploitation of the resources of the structure. Moreover, the displacement capacities associated with the uniform global mechanism are greater than those related to the soft storey one: adopting the non linear static procedures, this can lead to remarkable repercussions in terms of global verification (due to the increase of ductility).
- It complies with the concept of "sustainable repair". Damage concentrated in secondary elements promotes the objectives of life safety and post-earthquake use of the building.
- Various experimental campaigns have pointed out that damage to spandrels produces a more significant energy absorption than that to piers (Benedetti et al. 2001).

In this context, the paper proposes a critical review of the simplified models present in literature and codes for the prediction of the strength of masonry spandrels. With particular reference to existing buildings, attention is focused on the base configuration of a spandrel without coupling with other structural elements, such as tie-rods or r.c. beams. Moreover, a resistance criterion finalized to the interpretation of flexural behavior is proposed. A set of parametrical non linear analyses, using the non linear constitutive law proposed by Calderini and Lagomarsino (2008), has been performed in order to validate the proposal. Finally, some applications on complex masonry walls are presented in order to assess the effects on the global response, which derive from varying the hypothesis adopted for the spandrel.

2. OBSERVED SEISMIC FAILURE MODES: CLASSIFICATION AND INTERPRETATION THROUGH AVAILABLE MODELS

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).
However, in the case of spandrel elements, it is worth noting that:

- *Flexural behaviour*. Due to low values of axial load which usually characterize spandrel elements (especially with lack of tie-rods or r.c. beams), *Crushing* (usually associated with a widespread damage pattern, with sub-vertical cracks) represents a very rare instance.
- *Shear behaviour*. Due to the interlocking phenomena which can be originated at the interface between the end-sections of the spandrel and the contiguous masonry, *Sliding failure* (meant as sliding on a vertical bed joint plane) cannot occur.

Finally, the main failure modes of spandrels can be reduced to *Rocking* and *Diagonal Cracking* mechanisms. Moreover, earthquake damage observation in existing buildings shows that *Diagonal Cracking* tends to prevail in spandrels located at mid-storeys, while *Rocking* usually occurs in those on the top floor (Figure 1).
With reference to the safety verification criteria, the common practise is to adopt the same models developed for piers. Thus in the case of *Rocking* the ultimate limit state is obtained by failure at the compressed corners; then the resistance is usually calculated on the basis of the beam theory, neglecting the tensile strength of the material and assuming an appropriate normal stress distribution at the compressed toe (Eq.(2.1) in Table 2.1). In the case of *Diagonal Cracking* it is possible to classify two main types of models: models describing masonry as a composite material (Mann and Müller, 1980), considering separately its constituting components (joints and blocks), and models assuming masonry as an equivalent isotropic material, indistinctly considering the development of a crack along a principal stress direction (Turnšek and Čačovič, 1970).

Figure 1. Some examples of earthquake damage in existing un-reinforced masonry buildings

Most of the codes contain provisions to define 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 made to the verification criteria proposed for piers without significant differences. 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, also in this case, the result of this evaluation is aimed at properly orientating the choice between simplified models as "strong spandrel-weak pier" type or "weak spandrel-strong pier" type. The Italian code (OPCM 3431/05), 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 (N). If the acting axial load is known from the analysis, spandrel behaviour is assumed like that of a pier rotated to 90°. On the contrary, if it is unknown (which is the case of floors assumed as infinitely stiff), the following criteria are adopted: for the flexural behaviour, if the spandrel is coupled to another tensile resistant element, a response as equivalent strut is presupposed (Eq.(2.2) in Table 2.1); for the shear behaviour, only the cohesive contribution is considered. The expressions proposed in the Italian code for the evaluation of the flexural capacity of the spandrel (characterized by height h, width d, and thickness t, respectively) are summarized in Table 2.1. It is worth noting that they are consistent with those usually proposed in literature. Due to moderate values of the axial load which usually characterize spandrel elements, the use of Eq.(2.1) (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. It is actually worth noting that, by adopting Eq. (2.2), the strength associated to *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. Under seismic forces, while in piers flexure produces tensile stresses normal to the bed joints (horizontal) of masonry, in spandrels flexure produces tensile

stresses normal to the head joints (vertical); since masonry is an anisotropic material, the structural response of these elements is different. Moreover, in spandrel elements a further effect of confinement is supplied by floors and a moderate compressive effect may be due to the deformation of the masonry regions contiguous to the spandrel (rigid nodes in the "equivalent frame" idealisation). These latter issues represent the basis of the formulation of the criterion proposed in Paragraph 3.

Table 2.1 Resistance criteria proposed in literature and codes for the flexural behaviour of spandrels

		Notes
$M_u = \frac{Nd}{2}\left(1 - \frac{N}{dt\kappa f_{cu}}\right)$	(2.1)	f_{cu} is the compressive strength of the masonry in the vertical direction; κ =0.85 (assuming a rectangular stress-block distribution)
$M_u = \frac{dH_p}{2}\left[1 - \frac{H_p}{0.85 f_{hu} dt}\right]$	(2.2)	H_p : minimum between the tension resistance of the stretched interposed element inside the spandrel (such as r.c. beam or tie-rod) and $0.4 f_{hu} dt$, where f_{hu} is the compression strength of masonry in the horizontal direction

3. THE PROPOSAL OF A STRENGTH CRITERION FOR THE FLEXURAL BEHAVIOUR

The formulation proposed is 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.
The formulation is based on the following main hypotheses: the distribution of tensile stresses (perpendicular to head mortar joints) and that of shear stresses (which develop on the bed joint) are assumed to be uniform; the mechanical properties of head joints are negligible. Thus, referring to a reference volume at interface end-sections (of which the geometry is fully described in Figure 2.a by means of the following parameters: block height Δ_y, block width Δ_x and mortar joint thickness g), two main failure mechanisms are considered: *a*') tensile failure of the block; *b'*) shear failure of the horizontal mortar joints.
In the case of mechanism *a*' horizontal equilibrium, at the ultimate condition in which the block reaches its tensile strength f_{bt}, leads to:

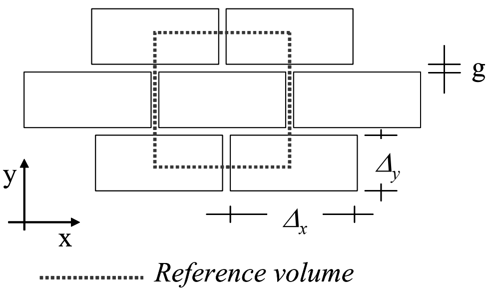
$$f_{tu,a'} = \sigma_x = \frac{f_{bt}}{2} \qquad (3.1)$$

where the thickness of the mortar joint has been neglected, it being sufficiently small if compared with the brick dimensions.
In the case of mechanism *b*', since no stresses can be transferred through head joints, equilibrium is only guaranteed by the shear stresses which develop on horizontal mortar joints. If a Mohr-Coulomb type is assumed as the failure criterion, adding the further hypothesis to neglect the cohesive contribution of mortar joint, the shear stresses result as a function of the σ_y component (normal to bed joint plane, vertical direction y, with reference to Figure 2.a). Thus the "equivalent" tensile strength associated with this mechanism may therefore be expressed in the following form:

$$f_{tu,b'} = \frac{\Delta_x}{2\Delta_y}\mu\sigma_y \qquad (3.2)$$

μ being the friction coefficient. It is worth noting that in the case of mechanism *b'* the interlocking parameter ($\varphi = \Delta_x/2\Delta_y$) and the entity of compressive stresses σ_y acting at the end-sections of the spandrel assume a decisive role. Thus a masonry spandrel characterized by a regular pattern and located at mid-storeys can count on a more significant contribution from this mechanism.
In the end the "equivalent" tensile strength f_{tu} of spandrel is associated with the minimum value obtained between $f_{tu,a'}$ and $f_{tu,b'}$. Usually, apart from masonry characterized by very weak blocks, mechanism *b'* tends to prevail. Moreover, it should be pointed out that tensile failure of the block (*a'*) is a brittle failure, whereas mechanism *b'* may be classified as ductile failure.
Thus assuming a constitutive law elasto-perfectly plastic with limited ductility in both tension and compression (quoted in the following as EPP-RT), the failure domain has been obtained. On the basis of the mechanical

properties which characterize both masonry (that is the compressive strength of the masonry f_{cu} and the limited ductility in compression μ_c) and the examined spandrel element (that is the "equivalent " tensile strength f_{tu} and the limited ductility in tension μ_t, defined on the basis of the prevailing mechanism between *a'* or *b'*), the relationship $M_u = f(N, \eta, \mu_c, \mu_t)$ can be obtained (η being the ratio between f_{tu} and f_{cu}). It should be pointed out that the following simplified assumptions have been assumed: to idealize masonry as a homogenous continuum; the section remains plane. Finally, the relationship which describes the failure domain can be found by solving a system of simple translation and rotation equilibrium equations (for reasons of brevity, the complete set of expressions has been omitted here, but a full description can be found in Cattari 2007).
Figure 2.b illustrates the proposed domain for different values of η. It could be observe that for rather low values of the assigned ductility μ_t the resulting domain differs not significantly from the case of infinite ductility in tension; thus in the case of mechanism *b'* this latter assumption is also reasonable. It is worth highlighting that the strength increase due to the proposed criterion can be truly remarkable in particular for low values of *N* which usually characterize spandrel elements. This beneficial effect is decisive even for very moderate values of η because it confers a strength (even if minimum) also in those cases in which, in the absence of another tensile resistant element coupled to the spandrel, it would be identically equal to zero.



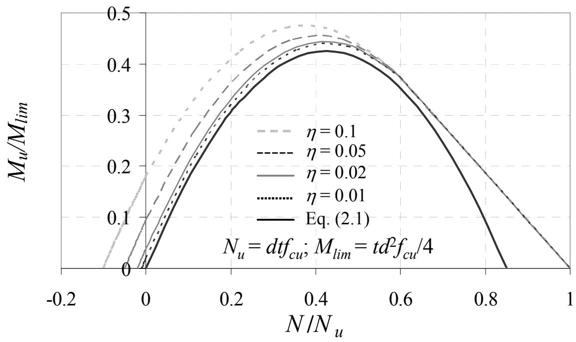


Figure 2. a. Reference volume; b. Proposed domain for different values of η (μ_c = 1.25, μ_t = at infinite)

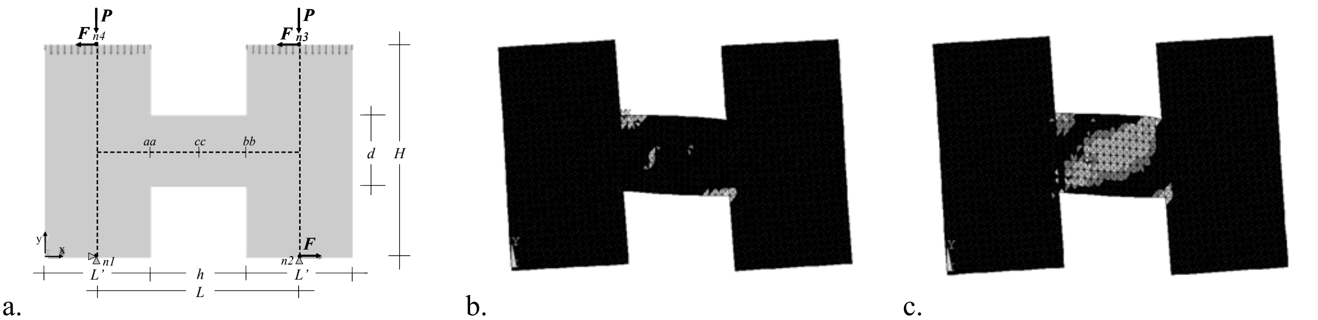
3.1. Validation of the proposed model

In order to validate the proposal, a set of parametrical non linear analyses was performed. The finite element method, together with the non-linear constitutive model for masonry proposed by Calderini and Lagomarsino (2008) was adopted; it considers both friction and cohesive resistant mechanisms of masonry, on the basis of a micromechanical analysis of the composite continuum.
Figure 3.a illustrates the test configuration adopted. Masonry elements were modelled by means of 3-node plane elements (plane stress); the presence of a wood lintel was assumed, modelling it by an elastic beam coupled to the spandrel only for the displacements in *y*-direction in order to simulate potential sliding phenomena. In order to take into account the interlocking phenomena which can be originated at the interface between the end-sections of the spandrel and the contiguous masonry, portions of size *L'* x *H* were modelled. A constant thickness *t* was assumed. Node *n1* was constrained in both *x* and *y* directions, on the contrary node *n2* only in *y* direction. The numerical simulations were carried out by applying: the dead load; a force distribution in *y* direction aimed at producing in the piers an axial load of resultant *P*; a system of horizontal forces *F* applied in nodes *n2*,*n3*,*n4* respectively. Thus the analyses were performed increasing monotonically the horizontal forces *F*. Moreover, starting from zero axial load applied to the spandrel, the influence of this load was also investigated. A standard brick masonry, characterized by a regular pattern and by lime mortar, was considered. The mechanical properties assumed for masonry are: f_{cu} = 6.2 MPa; cohesion of mortar joint *c* = 0.23 MPa; μ = 0.58; f_{bt} = 1.22 MPa; tensile strength of mortar joints 0.04 MPa. On the basis of this test configuration, parametrical non linear analyses were performed, with a different slenderness of spandrels (λ= 1.35, 2), different degrees of interlocking for the masonry (Δ_x/Δ_y =2,4) and different values of axial load applied to the spandrel (*N*=0 ÷100 kN such as to cause a mean stress on spandrel varying between the values of 0÷0.05 of f_{cu}) and to the piers (*P*=37.5 ÷225 kN such as to cause a mean vertical stress on piers varying between the values of 0.01÷0.1 of f_{cu}).
The results obtained may be summarized as follows:

- *Analysis of the evolution of the stress components*. The analysis of σ_y stress component at sections *aa*-*bb*

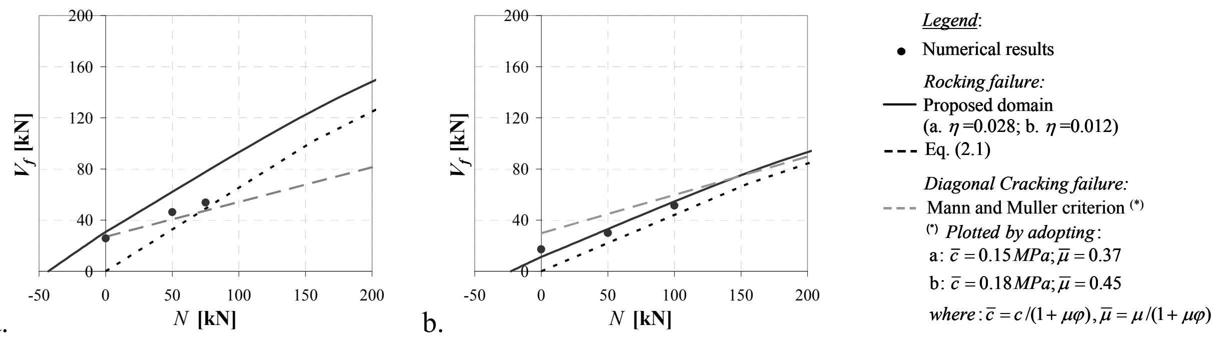
(Figure 3.a) in the first phase (the approximately "elastic" one) of response allowed us to ascertain how it corresponds approximately to 65% of the mean compressive stresses acting on the piers (then this factor was assumed for computing $f_{tu,b'}$ by Eq.(3.2)). Proceeding to the inelastic response, an increase of the σ_x stress component at tense toe of sections *aa-bb* was observed until the maximum value reached by the shear force V_{fu} which develops in these sections; moreover, it was ascertained that the maximum value of σ_x that occurred results greater than 15-20% in terms of percentage compared to the analytical one deduced from Eq.(3.2) (the mechanism *b'* being prevailing as a consequence of the mechanical parameters adopted). Thus, it can be concluded that, even if the σ_y stress distribution varies, a precautionary evaluation of f_{tu} can be obtained on the basis of the axial load acting on piers in static conditions.

- *Analysis of failure mechanisms that occurred*. In the case of the spandrel characterized by λ=1.35 it is possible to distinguish two main phases. After attainment of the maximum tensile value of σ_x at sections *aa-bb* (roughly corresponding also to the attainment of V_{fu}), it is possible here to clearly recognize a sudden fall in all the stress components, due to the activation of a damage mechanism characterized by the opening of the head joints in tense corners (Phase A, Figure 3.b). Then the spandrel gradually starts to behave as an "equivalent strut" with the formation of a diagonal crack, which develops at the centre of the element and then propagates towards the corners (Phase B, Figure 3.c). On the contrary, in the case of a spandrel characterized by λ=2, the failure mechanism may be classified as *Rocking*; thus the crack pattern is like the Phase A described before (focused on the tense corners) without the next activation of *Diagonal Cracking*.
- *Comparison between numerical and analytical strength domains.* The criteria were plotted in Figure 4 on the basis of the mechanical parameters adopted. It is worth clarifying that in the case of *Diagonal Cracking* the criterion formulated by Mann and Müller was adopted; this choice is due to the type of masonry considered here, characterized by a regular texture and by very resistant and stiffer blocks than mortar joints and, thus, by a clearly anisotropic behaviour. In general, a good correlation can be observed from both qualitative (failure mode occurred) and quantitative (predicted value of V_{fu}) points of view (Figure 4). It is worth noting that, in the case of spandrel characterized by λ=1.35, the adoption of Eq. (2.1) could lead to a strong underestimation of the actual resistance; this result would be even worse by adopting Eq.(2.2), since the lack of tensile resistant elements coupled leads to zero value for each value of N applied.



a. b. c.

Figure 3. Test configuration (a) and damage pattern (λ=1.35, Δ_x/Δ_y=4, P=225 kN, N=0) for subsequent steps of the analysis in terms of inelastic strains plotted only in those elements in which failure was attained (b and c): b. opening of head joints (inelastic normal strains along x in Phase A); c. inelastic shear strains (Phase B) (Legend: black colour corresponds to elements not at failure)



a. b.

Figure 4. Comparison between numerical and analytical strength domains: a. λ = 1.35, Δ_x/Δ_y = 2, P=150 kN; b. λ = 2, Δ_x/Δ_y = 4, P = 37.5 kN

4. APPLICATIONS

The failure domain proposed for the flexural behaviour of spandrels was implemented in the 3Muri program which operates in masonry idealization as an equivalent 3D frame.
The examined case is a three-storey masonry structure (Figure 5.a) representative of residential buildings of European and Italian context. The mechanical properties are coherent with the assumption of simple stone for the external walls and brick masonry for the internal ones; the floors are semi-rigid (for further information see Cattari and Lagomarsino 2006).The lack of r.c. beams or tie-rods coupled to the spandrels is assumed. Masonry elements are modelled as non-linear beams characterized by bilinear behavior. Non linear static (*pushover*) analyses were performed with different load pattern assuming for the flexural behaviour of spandrels respectively: case I) the strength criterion expressed by Eq.(2.1); case II) the proposed criterion (by assuming $\Delta_x / \Delta_y = 2$, $\mu = 0.4$, f_{bt} =1.85 MPa).
Figure 5.b shows the results concerning to the analysis in X direction with "uniform" load pattern (that is proportional to mass) in terms of V_{base} (shear base) $-u_{roof}$ (displacement of control node located on top of the building) curve; the analysis was stopped at the step corresponding to 20% decay of the maximum reached base shear. By assuming the proposed criterion for spandrel elements, both a significant increase in the overall resistance and a decrease in the global ductility can be observed. This latter result can be explained by both the different pattern and sequence of damage which occurred in cases I and II.

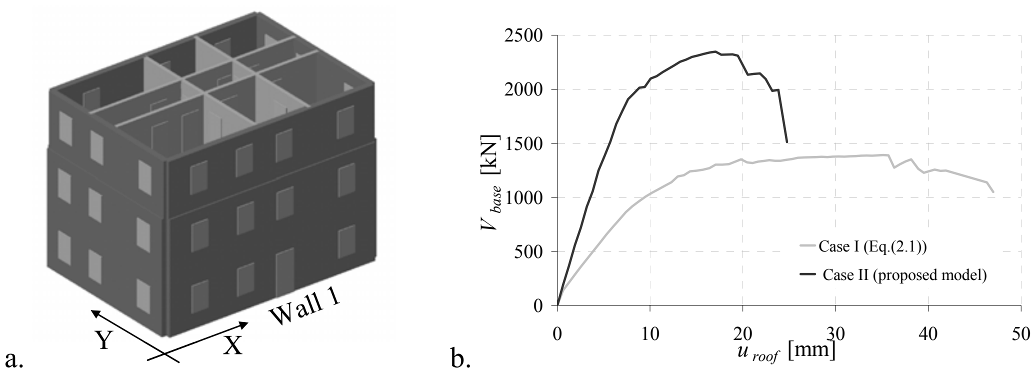


Figure 5. a. 3D view of the examined building; b. V_{base} $-u_{roof}$ curve for X direction-uniform load pattern

In case I, due to the moderate axial load acting on spandrel elements, 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 weak coupling for piers (as testified in Figure 5.b by the global stiffness in the V_{base} $-u_{roof}$ curve which is lower in case I than case II). On the contrary in case II (Figure 6), it can be observed: a first phase (Figure 6.a) 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 (Figure 6.b) in which the damage progressively occurs also in the spandrels located at mid-storeys (by both *Rocking* and *Diagonal Cracking* mechanisms); a final phase (Figure 6.c), in which the damage also spreads to piers located on the ground floor (which is the phase that corresponds to the global collapse of the structure).

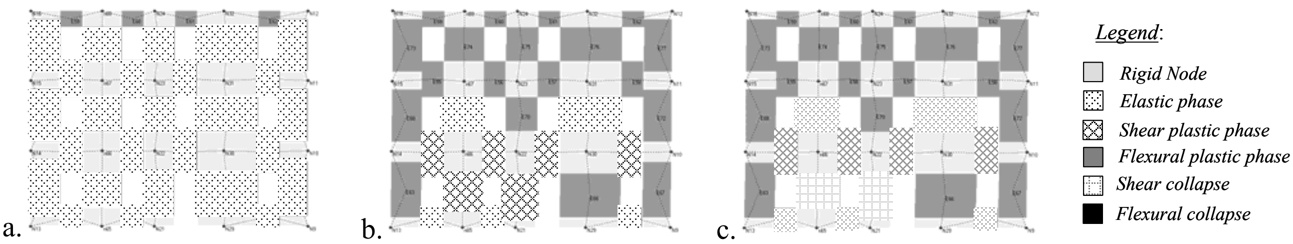


Figure 6. a. Damage sequence of Wall 1 for case II (Note for the legend: *collapse* indicates the overcoming of limit drift values; *plastic phase* indicates the reaching of the maximum value of strength)

5. FINAL REMARKS

In this paper the decisive role played by spandrel elements in the seismic response of un-reinforced masonry buildings has been discussed.
The critical review of the simplified models present in literature and codes for the prediction of their in plane load-bearing capacity has shown how a suitable interpretation of their flexural response represents a crucial point. Particular attention has been paid to the case of existing buildings where further tensile resistant elements, such as r.c. beams or tie-rods, coupled to the spandrels are often absent. It has been highlighted how the common practice of adopting the same models proposed for pier elements can lead to severe underestimations of the strength and to an unrealistic prevalence of the *Rocking* failure mode (not confirmed by earthquake damage observation). In this context the proposal of a resistance criterion finalized to the flexural behavior interpretation has been proposed. It is founded on the assumption that the response as an "equivalent strut" of the 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. The numerical analyses performed confirmed the theoretical basis of the proposal. Some preliminary applications to complex structures, by using masonry idealization as an equivalent 3D frame, highlighted the significant repercussions on the global response of un-reinforced masonry buildings.

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