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SEISMIC VULNERABILITY METHODS FOR MASONRY BUILDINGS IN HISTORICAL CENTRES: VALIDATION AND APPLICATION FOR PREDICTION ANALYSES AND INTERVENTION PROPOSALS

Maria Rosa VALLUZZI[1], Giuliana CARDANI[2], Luigia BINDA[3], Claudio MODENA[4]

SUMMARY

A valid approach for the limit analysis of existing masonry buildings in seismic areas concerns the application of single or combined kinematics models involving the equilibrium of structural macro-elements. They can be more reliable in describing the real structural behaviour than common equivalent static procedures, based on the "box" behaviour of the structure and on the elasto-plastic behaviour of the masonry. The main results of the application of different procedures for the static analysis of masonry buildings in seismic area are discussed in the paper. Urban centres suffering different levels of damage and different typologies of buildings are compared.

INTRODUCTION

One of the most recent seismic events occurred in Italy (1997 Umbria-Marche earthquake) struck several historic centres involved in retrofitting phases after previous similar events. The effects, in some cases, were disastrous, as many buildings were retrofitted with heavy interventions (substitutions of timber floors and roofs with reinforced concrete and hollow tiles mixed, jacketing, etc.) without taking into account the real behaviour of the structure, both in the original and modified conditions (Fig. 1, Fig. 2 and Fig. 3) (Binda et al. [1], Penazzi et al. [2], Penazzi et al. [3]).
Assessment methods suggested by the national standards, in fact, are based on hypotheses often not easy to be satisfied in old centres (effective strong connection among the structural components, presence of stiff floors able to transmit the horizontal forces to shear walls, etc.).
Recently, more reliable procedures for the evaluation of the seismic vulnerability, applicable both at global and local level, have been validated on the basis of extensive in-situ survey performed on the damaged areas by direct comparison of the obtained results with the real damage occurred (Bernardini et al. [4], Giuffrè [5]). They are based on the application of single or combined kinematics models involving

[1] Assistant Professor, Dept of Construction and Transportation Engineering, University of Padova, Italy. Email: valluzzi@caronte.dic.unipd.it
[2] Ph.D., Dept of Structural Engineering, Polytechnic of Milan, Italy. Email: cardani@stru.polimi.it
[3] Full Professor, Dept of Structural Engineering, Polytechnic of Milan, Italy. Email: binda@stru.polimi.it
[4] Full Professor, Dept of Construction and Transportation Engineering, University of Padova, Italy. Email: modena@caronte.dic.unipd.it

the equilibrium of macro-elements, composed by single walls or subassemblages (intersecting walls, walls and floors or roof, etc..).

Those models, calibrated on the real damaged sites, are usefully applied for prediction analyses of vulnerability for centres under seismic hazard, in order to prevent their future damage. Moreover, the simulation of possible interventions can be performed, both in damaged and non-damaged conditions (Valluzzi et al. [6]).

In the paper, the main results of the application of different procedures for the seismic analysis of masonry buildings are discussed. In particular, the extensive study both on centres suffering different levels of damage (Montesanto, Roccanolfi) and sites in current hazardous conditions (Campi) is proposed. The procedure validation is preliminarily demonstrated for a damaged isolated building (Montesanto), by comparing the results of the analysis with the surveyed crack pattern. As following step, the same procedures was applied on a more complex building (Roccanolfi): such phase pointed out some limits of the general methods working at global level, as the related required simplifications can be too far from the real configuration of the construction. Finally, the analysis was performed on the low damaged row buildings typology (Campi), in order to predict the current seismic vulnerability and to simulate some proposal of rehabilitation and improvement interventions.

Fig. 1: Collapse of the upper floor of a building due to the substitution of the timber roof with a heavier r.c. and hollow tiles mixed one, supported by poor masonry walls.

Fig. 2: Rear of the same building were a large overturning of the façade occurred, due to the high percentage of openings, combined with the presence of a heavy r.c. floor supported by a single layer of the poorly connected double-layer masonry.

Fig. 3: Loss of effectiveness of a wall strengthened with jacketing due to the scarce connection of the steel net through the masonry and the lack of overlapping of the reinforcement at the corners.

THE STRUCTURAL MACROMODELLING

Existing masonry buildings in historic centres, often do not satisfy the general conditions which allow the application of common equivalent static procedures, based on the “box” behaviour of the structure (which requires the presence of well-connected walls and floors and a proper horizontal stiffness of the floors) and on the elasto-plastic behaviour of the masonry. Common buildings in historic areas, in fact, are often realized following a traditional code of practice and according to typologies (multi-material masonry, multi-leaf walls) and constructive details (poor connection between intersecting walls, between walls and floors and even among the layers in the thickness), which in some cases can evidence fundamental deficiencies for the stability and the safety under seismic actions (Fig. 4) (Valluzzi et al. [7]).

a) b) c)

Figure 4: Out-of-plane mechanisms on existing building under seismic actions: collapse of the façade due to the lack of connection between floors and wall and among walls (a), expulsion of the external layer of the wall due to the multi-layer poorly connected constructive system (b), collapse of the corner due to the excessive nearness of the openings (c).

In such conditions the ultimate capacity of the building depends on the stability of its macro-elements, which is a portion of the structure bounded by the potential damage pattern (cracks, borders of poor connections, etc.) that can behave as a whole, following a kinematics mechanism (Giuffrè [5]).
Macro-elements are defined by single or combined structural components (walls, floors and roof), considering their mutual bond and restraints (e.g. the presence of ties or ring beams), the constructive deficiencies and the characteristics of the constitutive materials.
Once the critical structural configuration is defined, the subsequent step is the identification of the most probable collapse mechanism/s characterizing each macro-element.
Several studies based on the in-situ observations after seismic events allowed to systematize abacuses of the typical damages occurring in constructive typologies (buildings, churches), which led to the consequent systematization of the mechanical models able to describe their specific behaviour by kinematics models, both for in-plane and out-of-plane mechanisms.
Kinematics models provide a collapse coefficient $c=a/g$ (where a is the ground acceleration and g the acceleration of gravity), which represents the masses multiplier able to led the element to failure. In the simplified assessment procedures, the mechanism connected to the lowest value of c is the weakest one and, consequently, the most probable to occur. The collapse is thus due to a loss of equilibrium of its structural portions rather than for state of stress exceeding the material ultimate capacity. Both out-of-plane and in-plane mechanisms are considered.
Those models, calibrated on the real damaged sites, are usefully applied for analyses of vulnerability for centres under seismic hazard, in order to examine the current condition and to prevent their future damage (Valluzzi et al. [4]). Moreover, the simulation of possible interventions can be performed, both in damaged and undamaged conditions, evaluating their impact with the pre-existing situation (Valluzzi et al. [6]).
In the following, a compendium of the main mechanisms allowable in literature is given. Some of them have been implemented in automatic procedures (VULNUS, Bernardini et al. [8], Bernardini et al. [4]) able to combine different mechanisms for global vulnerability analyses of buildings with sufficient regularity (both in plane and in elevation) and limited height (three storeys or less), and that take into account the type of connection among the structural elements.

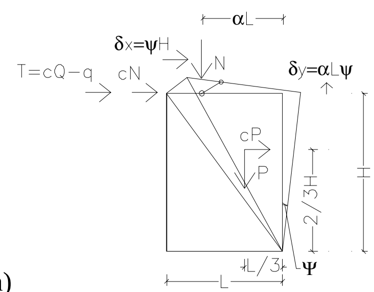
In-plane mechanisms

In-plane mechanisms concern the walls parallel to the seismic action. They are also named “second-way mechanisms” (Giuffrè [5]), because the related damage (shear cracks), is often not able to led the structure to the collapse, in comparison with the out-of-plane mechanisms. For that reason, in-plane mechanisms are characterized by collapse coefficients higher than the out-of-plane ones.

Kinematics chains describe the in-plane rigid rotation of the vulnerable structural portions of the building, defined by particular geometrical (dimensions, openings) and bond conditions (connection, presence of ties), under in-plane horizontal actions.

For each wall, the resisting sects and the related involved seismic forces are identified. Fig. 5 shows the single and multiple-panel cases. The latter one concerns the in parallel behaviour of the sects, bordered by the contour of the openings of the same wall. In such case, being the sects aligned to the upper edge and due to the presence of a longitudinal tie, the equilibrium equation requires the imposition of the horizontal displacements equality at the upper edge itself (Giuffrè [5]).

Symbols in the figure are as follows: N is the vertical load acting at a distance of αL from the compressed edge, P is the weight of the detaching portion and T is the tensile force in the tie, obtained by the difference between the weight cQ of the supported portion of the wall and the counteraction (q) allowed from the supporting base of the wall.



Equilibrium condition:

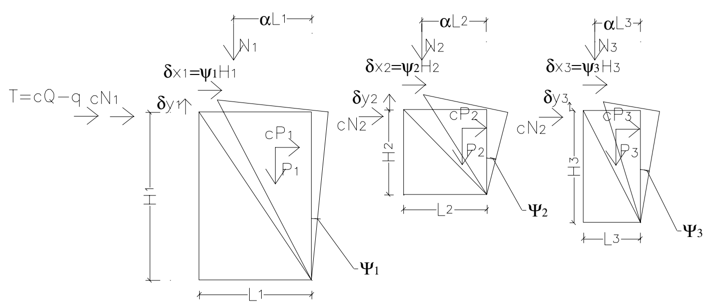
$$-N\times\alpha L-P\times\frac{L}{3}-q\times H+c(N\times H+Q\times H+P\times\frac{2}{3}H)=0$$

Collapse coefficient:

$$c=\frac{N\times\alpha L+P\times\frac{L}{3}+q\times H}{N\times H+Q\times H+P\times\frac{2}{3}H}=\frac{L}{H}\times\frac{N\alpha+\frac{P}{3}+q\frac{H}{L}}{N+2\frac{P}{3}+Q}$$

a)

b)



Equilibrium condition:

$$-N1\times 0.75L1-N2\times 0.75L2\times\frac{H1}{H2}-N3\times 0.75L3\times\frac{H1}{H3}-P1\times\frac{L1}{3}-P2\times\frac{L2}{3}\times\frac{H1}{H2}-P3\times\frac{L3}{3}\times\frac{H1}{H3}-q\times H1+$$

$$+c(N1\times H1+N2\times H2\times\frac{H1}{H2}+N3\times H3\times\frac{H1}{H3}+P1\times\frac{2}{3}\times H1+P2\times\frac{2}{3}\times H2\times\frac{H1}{H2}+P3\times\frac{2}{3}\times H3\times\frac{H1}{H3}+Q\times H1=0$$

Figure 5: Scheme of the in-plane kinematics model for a wall under in-plane actions: a) single wall, b) multiple-wall system (Giuffrè [5]).

Out-of-plane mechanisms

Out-of-plane mechanisms, also named of “first-way” collapses (Giuffrè [5]), involve walls subjected to horizontal actions orthogonal to their plane. The overturning is the main action, which is counteracted by the possible presence of connection elements (ties, ring beams) or intrinsic resisting effects (e.g. arch effect of the wall in its thickness). The method is based on equilibrium equations which can take into account also the strength of the materials (crushing of masonry, tension in the tie, ..). In Fig. 6 and 7, their classification performed by grouping the mechanisms involving horizontal and vertical strips of the wall, is shown. Nearby the kinematics scheme the formulation of the collapse coefficient is given.

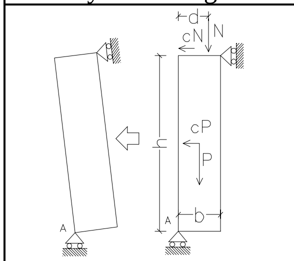
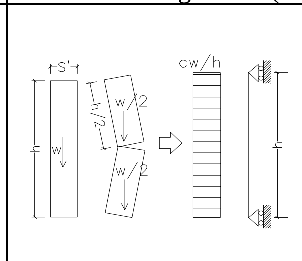
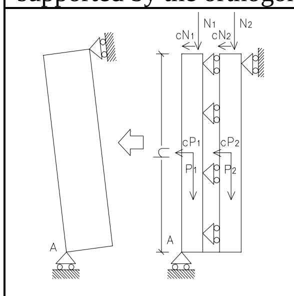
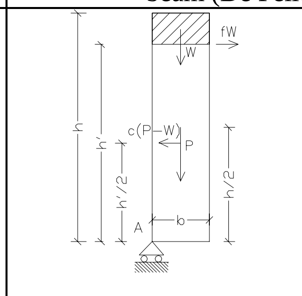
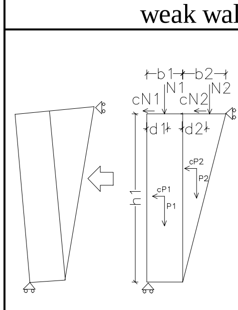
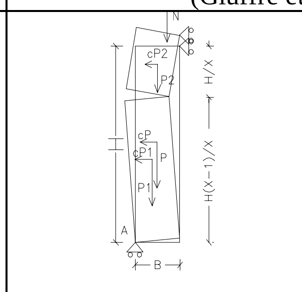
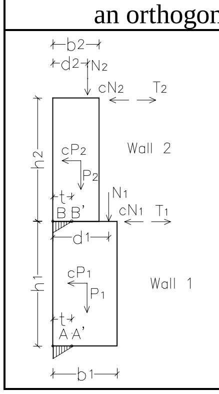
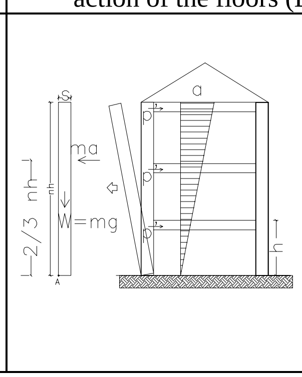
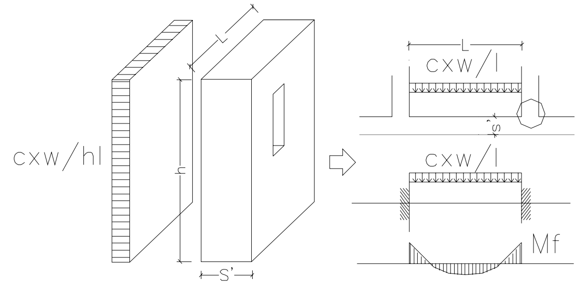
Overturnig of a monolithic wall simply supported by the orthogonal wall (Avorio et al. [9])		Out-of-plane collapse of a wall subjected to high confining forces (Bernardini et al. [8])	
	$c=\dfrac{P\times\dfrac{b}{2}+N\times d}{P\times\dfrac{h}{2}+N\times h}$		$c=\min\left(\dfrac{2s'}{h};\dfrac{(\sigma_t+\frac{w}{2})\frac{4}{3}s'^2}{wh^2}\right)$
Overturning of a double-layer wall simply supported by the orthogonal wall (Avorio et al. [9])		Overturning of a wall restrained at the top by a ring beam (De Felice et al. [10])	
	$c=\dfrac{P_1\times\dfrac{b_1}{2}+N_1\times d_1}{(P_1+P_2)\times\dfrac{h}{2}+(N_1+N_2)\times h}$		$c=2f\dfrac{h-h'}{h}+\dfrac{bh}{h'^2}$
Overturning of a wall connected to a perpendicular weak wall (Avorio et al. [9])		Overturning of a wall restrained at the top by a tie (Giuffrè et al. [11])	
	$c=\dfrac{P_1\times\dfrac{b_1}{2}+N_1\times d_1+P_2\times\left(b_1+\dfrac{b_2}{3}\right)+N_2\times(b_1+d_2)}{P_1\times\dfrac{h_1}{2}+N_1\times h_1+P_2\times\dfrac{2h_1}{3}+N_2\times h_1}$		$c=\dfrac{B}{H}\cdot\dfrac{2x+\left(\dfrac{N}{P}\right)(x+1)x}{x-1}$
Overturning of multi-floor walls not connected to an orthogonal wall (Avorio et al. [9])		Global overturning of a wall with the counteracting action of the floors (Bernardini et al. [8])	
	$c=\dfrac{P_2\times\dfrac{b_2}{2}+P_3\times\dfrac{b_3}{2}+N_2\times d_2+N_3\times d_3}{P_2\times\dfrac{h_2}{2}+N_2\times h_2+P_3\times\left(h_2+\dfrac{h_3}{2}\right)+N_3\times(h_2+h_3)}$		$c=0.75\dfrac{s}{h}\times\dfrac{1}{n}+\dfrac{p'}{W}n$

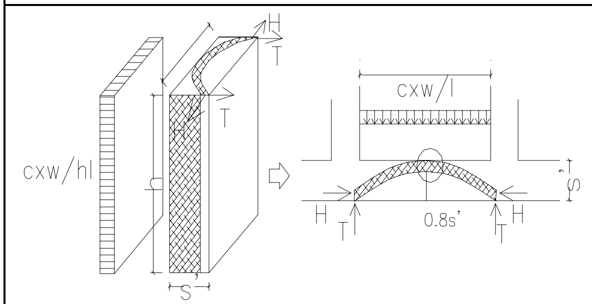
Figure 6: Kinematics models for out-of-plane mechanisms: vertical strips.

Fixed beam mechanism



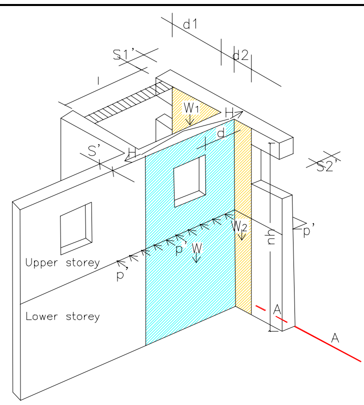
$$c = \frac{2\sigma_t \times h \times s'^2}{w \times l}$$

Arch effect in the thickness of the wall: ultimate condition for masonry crushing



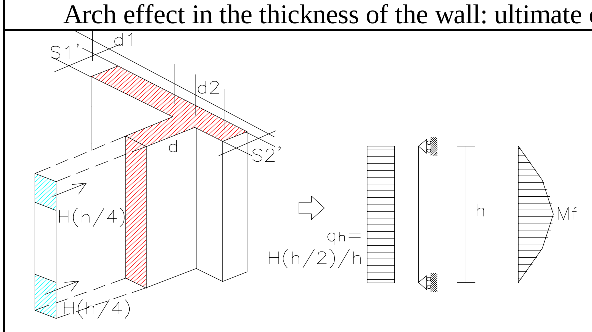
$$c = 1.28\frac{\sigma_c \times s'^2 \times h_{netta}}{w \times l}$$

Arch effect in the thickness of the wall: ultimate condition for abutments overturning



$$c = \frac{P_e\left[\frac{d_1^2 s_1^2 + d_2^2 s_2^2}{2} + (s+l)^2 \frac{s''}{8}\right] + p'(d_1 + d_2)\frac{2}{3}n}{P_e \frac{l^2}{6.4}}$$

Arch effect in the thickness of the wall: ultimate condition for compressive failure in the section



$$c = \frac{(\sigma_t + P_e \times \frac{h}{2}) \times (2d \times \frac{d_1 s_1' + d_2 s_2'}{3} + d\,\frac{ds'}{6})}{\frac{P_e}{64} s \frac{l^2}{1.6s'} h^2}$$

Figure 7: Kinematics models for out-of-plane mechanisms: horizontal strips (Bernardini et al. [8]).

CASES STUDY

Isolated building (Montesanto)

The extensive in-situ survey of some masonry building centres damaged during the Umbria-Marche earthquake in Italy (1997), performed in the last years by the Polytechnic of Milan, the University of Genova and the University of Padua (Binda et al. [1]), allowed to systematize the possible mechanisms of collapse (both in original and retrofitted conditions) in a reference abacus. Consequently, the assessment of the reliability of the above mentioned procedure by direct comparison of the obtained results with the real damage occurred, was possible (Valluzzi et al. [7]).

As an example, the analysis of an isolated building located in Montesanto (Fig. 8) showed that the kinematics models which correspond to the lowest *c* coefficients (values lower than 0.28, which corresponds to the safety limit for the considered seismic zone prescribed by the national standards) are consistent with the main collapse mechanisms ascribable to the real damage (Fig. 9). In particular, they concern out-of-plane effects like the overturning of the most damaged façade (East) and of the corners; as confirmation, the presence of typical shear sloped cracks is related to in-plane mechanism connected to higher *c* coefficients (Fig. 9).

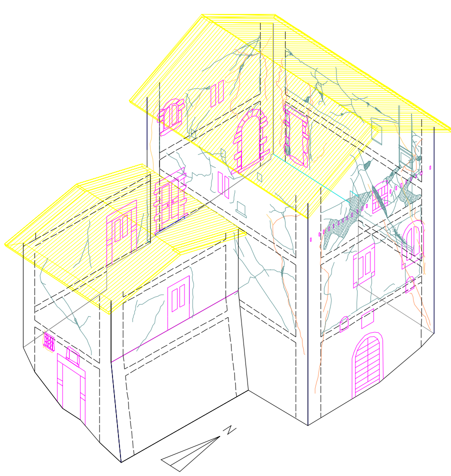


Figure 8: View of the building with survey of the crack pattern: the most damaged portions are the Eastern wall and the Nord-Eastern corner (overturning effects).

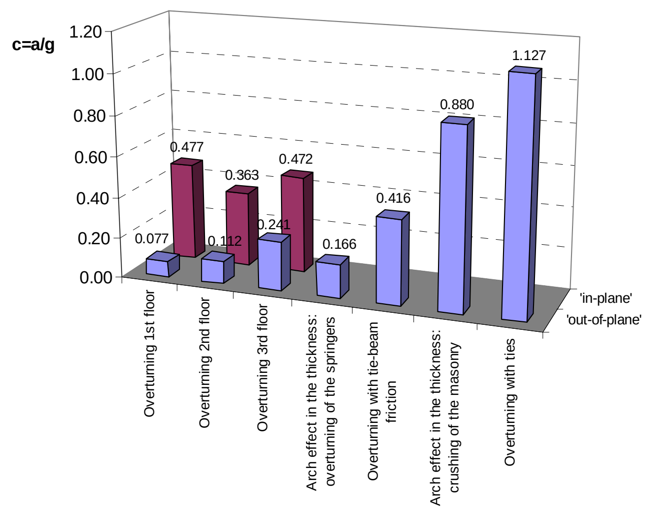


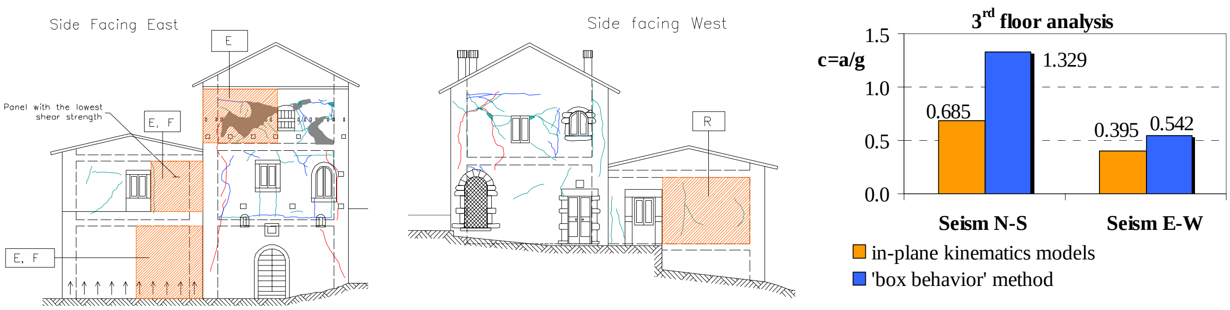
Figure 9: Comparison among the single kinematics models: the lowest seismic coefficients are related to the out-of-plane mechanisms involving the most damaged portions of the building.

The building was retrofitted after a previous earthquake occurred in 1979, so the situation before the new event occurred in 1997 was related to the presence of stiff floor and roof (mainly r.c. and hollow tile mixed with tie-beam along the borders) and consolidated walls (even if only partially at the first floor) (Fig. 10). Such conditions allow the application of typical assessment methods based on the "box behavior" of the structure, which take into account only the in-plane shear strength of the masonry panels composing the walls, as suggested by national standards.

Nevertheless, the comparison of such method with the results obtained by the application of kinematics models, showed that it is not able to detect the sects of the walls wich correspond to the actual damage of the building (Fig. 11). Moreover, as expected, the related seismic coefficients are higher than the ones obtained by the application of the single elementary mechanisms models, so the assessment with that method could be unfavourable for the safety (Fig. 11.c).

a) b) c)

Figure 10: Interventions performed before the last earthquake on floors (a: execution of the r.c. tie-beam, b: detail of the r.c. floor) and walls (c: phase of injection) in the building located in Montesanto (Perugia, Italy).



a) b) c)

Figure 11: Results of method adopting the “box behavior” of the structure: (a) the most damaged panels (East side) are given as simply reaching the elastic (E) or first crack limit (F), whereas the panel led to rupture (R) belong to a very low damaged wall (b); comparison between the two methods (c).

Complex building (Roccanolfi)

For buildings characterized by more adjacent constructions (rows, complex) the general procedure for the vulnerability assessment is to perform first of all the global analysis and to control some local aspect by using the single kinematics models.

Nevertheless, some cases detectable in historical centers can have architectural and constructive aspects very complex, so a critical analysis of the results obtained at general level is necessary. In such connection, the study of a large complex located in Roccanolfi was considerd (Fig. 12 and 13), with reference to one of the most damaged aggregates of buildings (Fig. 12). Such complex is particularly irregular both in plan than in section and can be subdivided in eleven units, considered separately in the study (Fig. 14).

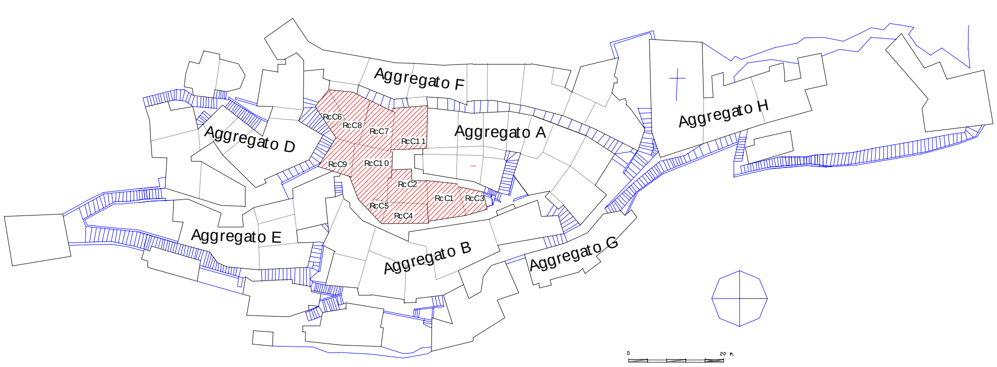


Fig. 12: Plan of Roccanolfi, where the hatched area corresponds to the “aggregate C”.

Fig. 13: View of Roccanolfi.

Fig. 14: Longitudinal section of the “aggregate C”.

The analysis is performed both at global level by the use of the procedure VULNUS (Bernardini et al. [8]), and at local level, by applying the single kinematics mechanisms. By VULNUS is possible to define two indexes, I1 and I2, related to the in-plane and out-of-plane collapse mechanisms, respectively. The significant parameter, both for the above-mentioned indexes and for the application of the single mechanisms is still the collapse coefficient c=a/g, as described above.
Results shoved that a prevalent number of units composing the complex have out-of-plane index lower than the safety limit imposed by the national standards (c=0.28) (Fig. 15) and it corresponds to the most damaged portions, as observed by the in-situ survey of the building (Fig. 16 and Fig. 17, and Tab. 1).

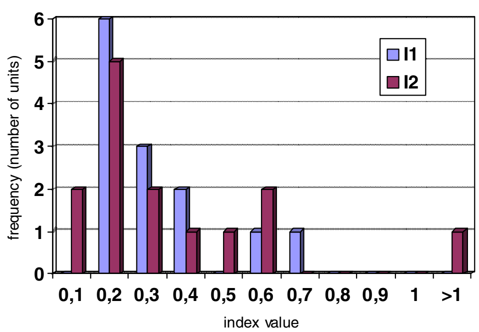


Fig. 15: Comparison between the two indexes and frequency of the results obtained for the variuos buildings included in the same complex.

Fig. 16: View of the units named RcC9, RcC10, RcC5 and RcC4 with evidence of damage on masonry walls and roofs.

Table 1: Damage observed on the complex building.

Unit	Out-of-plane mechanisms of the walls					Out-of-plane mechanisms triggered off by the roofs		In-plane mechanisms
	Global overturning	Partial overturning	Detachment between perp. walls	Overturning of the corners	Flexural deformation	Collapse of the tympanum	Collapse of lateral walls	Shear failure of horiz. strips
RcC1	X						X	
RcC2		X	X	X		X	X	
RcC3								
RcC4				X	X		X	X
RcC5								X
RcC6								
RcC7		X				X	X	
RcC8						X	X	
RcC9							X	
RcC10							X	
RcC11					X			

By operating with simulation of three degrees of seismic intensity, it was also possible to identify the vulnerability classes related to the several units of the building. Fig. 18 shows the synthesis of the analysis, by taking into account the different seismic hazard levels. It is worth to notice that units belonging to high vulnerability class suffered severe damage, especially as collapse of the upper floors, whereas for lower vulnerability classes units evidenced lower damage. Nevertheless, for some units (e.g. RcC1 and RcC10), having very irregular configuration, excessive simplifications were adopted to describe conditions not easy to foresee in automatic procedures (floors at different heights, presence of arcades and loggias, very bad quality of masonry walls in the thickness). As results, those elements conducted to very low coefficients, which even if increasing the global safety, are not responding to the real conditions detectable in-situ.

Fig. 17: View of some damaged roofs of the building.

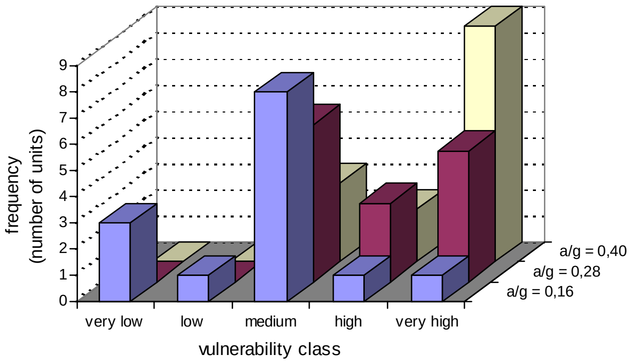


Fig. 18: Estimation of vulnerability for different classes of limit seismic coefficient.

Row buildings (Campi)

As for row buildings, an interesting site is Campi Alto di Norcia, a castle perched on a slope surrounded by walls, whose buildings are arranged in concentric terraces and narrow streets connected by short radial flights (Fig. 8 and 9). After the 1979 earthquake that caused many damages to the building structures, the centre of Campi has undergone several interventions of retrofitting that unfortunately changed almost all the original medioeval masonry features. Most of the times they were of utmost importance in the preservation of the historic centre after the 1997 earthquake, even if the seismicity in the valley (Castorian Valley) was of minor entity. In fact the damages found in Campi after 1997 were of irrelevant nature, and mainly located in those buildings that were not repaired since a very long time.

Fig. 8: View of Campi Alto.

Fig. 9: Plan of Campi Alto with evidence of the current standing buildings.

The global analysis with VULNUS conducted on the rows showed that the lowest collapse coefficients are referred to out-of-plane mechanisms (I2 index, see also Fig. 11). The global “survival” percentage of the buildings, with reference to the limit coefficient (c=0.28) is around the 81%. As expected, results denote a particular sensitivity of the most brittle mechanisms (overturning) for the head buildings of the rows. This was detected also by the application of the single collapse mechanisms: as an example, Fig. 10 shows the analysis performed on a row composed by four units, where the weakest mechanism (overturning of the upper floors), was found.
For the same row, the simulation of several intervention as the strengthening of the masonry walls with injections (where applicable), the possible filling of the openings too close to the corners of load bearing walls, and the rehabilitation of wooden floors and roofs with stiffening compatible techniques, can induce a significant improvement. This is quantifiable with a proper reduction of the specific vulnerability, as shown in Fig. 11 (in the figure, the only reduction of the coefficient is related to the proposal of a rebuilding with original stones of a panel which was previously substituted with clay bricks; it is possible to notice that that changing is still assuring a proper safety level of the building).

Fig. 10: Localization of the panels with lowest seismic coefficient (global overturning of a two floor wall) by simple kinematics models.

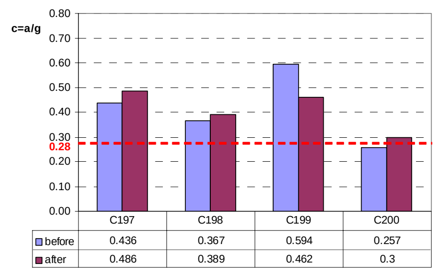


Fig. 11: Simulation of intervention with “Vulnus”: comparison of the I2 index (out-of-plane mechanisms).

CONCLUSIONS

The application of single or combined kinematics models involving the equilibrium of macro-elements revealed a better agreement with the poor structural characteristics often detected in existing masonry buildings, in comparison with standard assessment methods based on hypotheses often not easy to be satisfied in old centers. The general procedure can be used both for assessment of buildings in seismic areas and for prediction of the vulnerability or for simulation of proper interventions. It foreseens some simplification in the schematization of the real configurations of the existing buildings, therefore particular attention both in the application method and in results interpretation phases has to be paid for complex aggregates or irregular constructions.

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