SEISMIC DESIGN OF MASONRY BUILDINGS THROUGH MACRO-ELEMENTS

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SUMMARY

The paper focuses on some seismic design methods via macro-elements (RAN, SAM and 3MURI) analyzing the basic hypotheses, the calculus procedures, the criteria and the applicative capabilities. The comparison of the above methods has been accomplished through an application to a case-study. Linear and non-linear static analyses on the whole structure allowed to investigate and assess the structural behaviour of the construction under study. The following issues are discussed: (i) geometrical and mechanical modelling, (ii) collapse mechanisms characterizing masonry panels composing the wall; (iii) initial stiffness, peak load and ductile branch in the base shear versus top displacement diagram.

INTRODUCTION

In the last two decades a radical change in the computational approach to masonry structures, mainly related to some limits of Finite Elements (FE), occurred via the use of Macro-Elements. In fact, the complexity of the structural shapes and the impossibility of hypothesizing homogeneity, isotropy, iso-strength and elastic behaviour for the material, made the adoption of FE based methods extremely burdensome. Therefore, the need to formulate “simplified” analyses and design procedures with minor computational efforts and wider applicability to real structures rose. In order to meet such requirements without compromising the deepening level of the performed analyses, some design procedures have been formulated. Since the ‘80s, the seismic response of masonry buildings through analyses based on macro-modelling of walls, conceived as systems of masonry panels, has been studying.

MACRO-ELEMENTS METHODS

Macro-elements methods aim to the evaluation of the seismic response of masonry buildings modelling the opened walls as collection of bi-dimensional macro-elements (masonry panels). They are distinguishable, depending on their behaviour, in three different typologies (Fig. 1):

a) Pier panels, included between two openings in the vertical direction, having essentially the bearing function;
b) Spandrel panels, included between two openings in the horizontal direction, bearing the slab loads;

c) Cross panels, connection between pier and spandrel strips, carrying the slab loads and transmitting stresses between pier and spandrel panels.

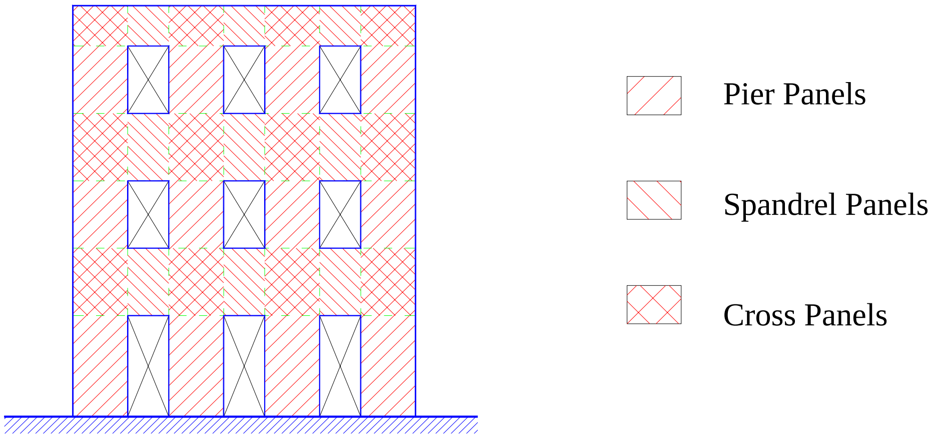


Figure 1: Wall division in macro-elements

In the field of analysis through macro-elements it is possible to distinguish a considerable number of methods that attempt to reproduce the behaviour of walls and masonry structures subjected to seismic and other kind of loads. This paper focuses on the analysis of the following methodologies:

1. **RAN method** (introduced by Raithel & Augenti, 1984);
2. **SAM method** (presented by Calvi & Magenes, 1997);
3. **3MURI method** (initiated by Gambarotta & Lagomarsino, 1997).

The RAN Method

The RAN method allows a linear static seismic analysis at each storey and a global non linear analysis until the failure attainment of the pier panels in the hypothesis of infinite strength and stiffness of spandrel strips (Augenti, 2004). The method overcomes the inconvenient linked to the execution of analysis at each floor which imply the not fulfilment of local and global equilibrium of walls. In this procedure, the overturning moment induced by horizontal actions is taken into account. This is achieved balancing its effects through the variations of the normal stress on the pier panels.

In the study of the single macro-elements, particularly of pier and spandrel panels, all the possible failure mechanisms are taken into account: composed flexural and axial load, tensile shear and sliding shear. In particular, for pier panels it is possible to conduct a check both in the fixed-double pendulum (namely *Grinter*) and in the cantilever restraint conditions (failure of the spandrel panels). In the analysis of pier panels, also a reduced section induced by the cracking of the material, is considered. Panels are analyzed both in the elastic and plastic fields. As a consequence, a curved trend of the corresponding characteristic curves shear force – displacement (V - δ) in the non proportional elastic and plastic parts is detected.

On the other hand, for spandrel panels, only the elastic behaviour is considered, in agreement with the restraint hypothesis.

Along the observations of the damages produced by real earthquakes, the internal cross panels are in fact immune by collapse mechanisms, although it is possible to determine the stress state and the interactions with the other resistant elements composing the walls.

The RAN method represents a valid tool for determining the ultimate strength offered by the wall, with reference both the elastic and plastic limit states. Through this instrument, therefore, it is possible the design of retrofitting interventions necessary to re-establish the strengthening and functionality of the masonry structure, as well as the evaluation of the effectiveness of these interventions employing different materials.

The SAM Method

The method uses the non linear static analysis as the preferential analysis tool of the seismic response of masonry buildings, and studies the structure globally satisfying immediately the equilibrium conditions (Magenes & Della Fontana, 1998; Magenes et Al., 2000). The analysis through macro-elements takes into account, also in this case, all the possible collapse mechanisms: eccentrical axial load, tensile shear and sliding shear. Spandrel panels have a reduced strength with infinite ductility which allows to consider these elements able to explicate the shear strength until the collapse of the whole wall. For pier panels, considered with limited strength, a variation of the distribution of the flexural stresses following the change of the restraint conditions is contemplated. They vary from the fixed-double pendulum typology to the cantilever one. The stresses distribution among panels depends on their linear elastic strength, starting from the initial condition of fixed-double pendulum restraint and iteratively proceeding to the stresses distribution within the wall until the failure of the first pier panel.

In the study of spandrel and pier panels the reduced section induced by cracking due to external loads is taken into account but, in the diagrams, the non linear behaviour is assimilated to a bilateral line. The cross panels are assumed stiff and unlimited strength and are modelled through rigid wings (offsets) introduced at the boundary of pier and spandrel panels. No failure mechanisms for the cross panels are contemplated.

Again, the possibility of using different materials and systems for the bearing structure and the slabs is included into the method.

The 3MURI Method

The 3MURI method allows to carry out linear and non-linear static, dynamic modal and non-linear dynamic analyses (Penna, 2001; Cattari et Al. 2005). The response of the masonry panel is evaluated taking into account the mechanical properties of the material, their potential degradation and the real restraint and stress conditions.

The analysis of macro-elements behaviour mainly concerns masonry piers and spandrel strips. The method analyzes all the possible failure mechanisms for them: composed flexural and axial load, tensile shear and sliding shear. Furthermore, the reduction of the section (induced by cracking) allows to evaluate the stiffness degradation which involves the panels stresses varying generated by external actions. A stresses redistribution on the reacting section follows so that the shear-displacement curve presents a curved line in the non-linear range.

Finally, cross panels are assimilated to bi-dimensional rigid elements able to transmit the actions along the degrees of freedom in the plane, not assuming any collapse mechanism.

The method allows to take into account many typologies of slab with different stiffnesses, varying from reinforced concrete or wooden slabs to masonry vault systems.

Together with macro-element models, the 3MURI method allows to model masonry panels via non-linear masonry cantilevers characterized, as per macro-elements, by a reduced strength and a stiffness degradation in the non-linear range.

CASE STUDY

The building considered for the comparison of numerical analyses has a rectangular plan view. Its dimensions are 10,80×14,60 m^2 and two storeys for a total height of 9 m. The inter-storey height is 5 m at the first floor and 4 m at the second floor. In Fig. 2 the plan view and the front elevation scheme with openings (double hatched in the building plan) is reported.

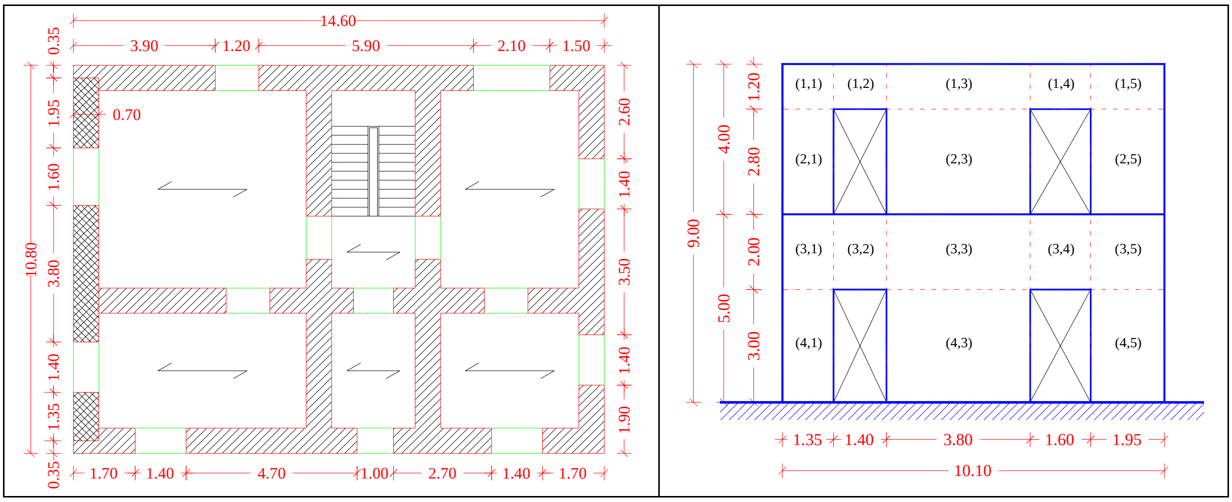


Figure 2: Building plan and panels schematization of one opened wall.

The vertical bearing structure is made of yellow tuff stones and cementicious mortar. The wall thickness is 70 cm at the first floor and 60 cm at the second one; the slabs, at the two levels, are assumed rigid in their plane.
The following values of the elastic and mechanical characteristics have been adopted in the numerical application (Table 1):

Table 1: Elastic and mechanical characteristics

Symbol	Definition	Value
γ	Specific weight	16 kN/m^3
f_k	Compressive characteristic strength of masonry	3,0 MPa
f_{vko}	Pure shear characteristic strength of masonry	0,3 MPa
f_d	Compressive design strength of masonry	1,0 MPa
f_{vdo}	Pure shear design strength of masonry	0,1 MPa
E	Normal elasticity module	3000 MPa
G	Tangential elasticity module	1200 MPa

The following vertical loads have been considered: (i) self weight of the walls; (ii) slab loads (Q = 8,10 kN/m^2) comprising self weight and dead loads (G_k = 6,10 kN/m^2) and accidental loads (Q_k = 2,00 kN/m^2).
Finally, in the determination of the seismic horizontal actions, an analysis according to the prescriptions of the Italian Code O.P.C.M. n. 3431, considering the building of class 1 (50 years service life) in the seismic zone 3 (a_g = 0.15g) on the ground foundation type D (shear wave velocity < 180 m/s, number of impacts through Standard Penetration Test <15, Tip Strength through Cone Penetration Test <70 KPa), has been conducted.

Application of the RAN Method

In the following, the procedure applied for the linear static analysis is described. In Fig. 3, the strength domains in the plane axial load – shear (N, V) for the pier panels of the wall in Fig. 2, normalized to the ultimate normal load (N_u), are reported.
The curves representing the borders of the domains, respectively are: elastic limit state for composed flexural and axial load (V_e); non proportional elastic limit state for eccentrical axial load (V_l); plastic limit state for composed flexural and axial load (V_p); tensile shear limit state (V_t); sliding shear limit state (V_{aapp} and V_{aeff}). The dotted vertical lines correspond to the normal load applied on pier panels taking into account the variation induced by the horizontal actions.

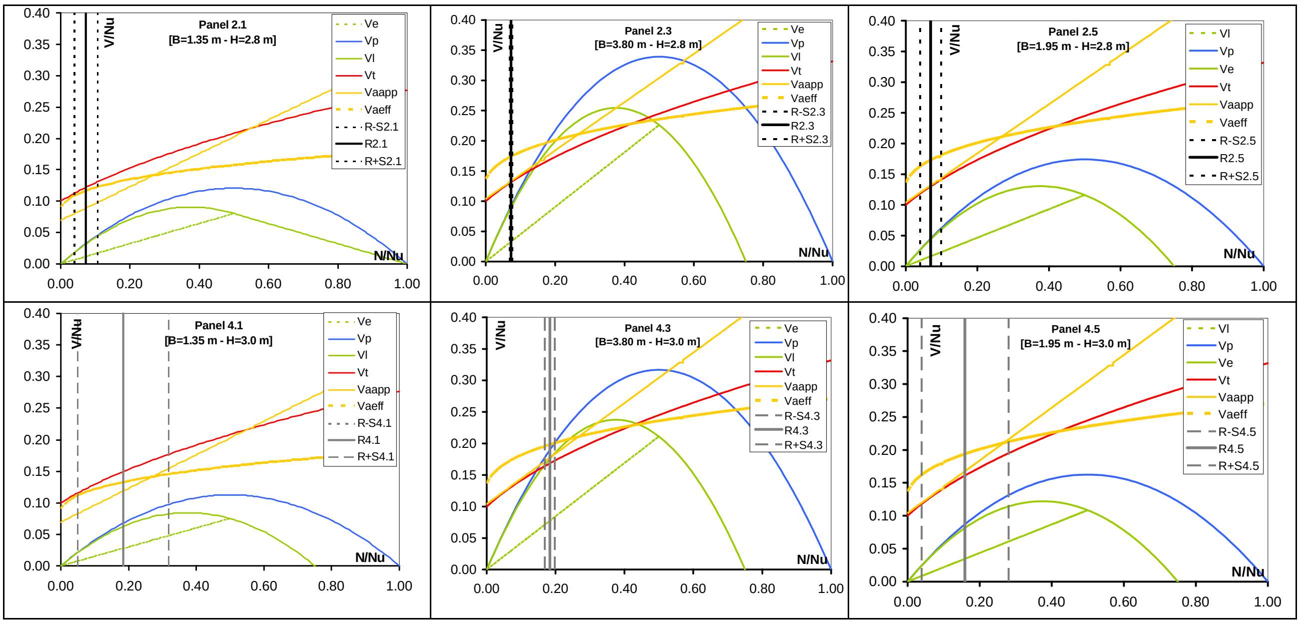


Figure 3: Pier panels strength domains of the wall in Fig. 2.

Check of panels do not stop to the equilibrium conditions since, panels belonging to the same level have the same displacements for congruence conditions. Therefore, the characteristic curve for each panel constituting the walls, relating the shear force V applied at the top and the dual displacement δ, has been determined.
Generally, it is composed of three branches (Fig. 4):

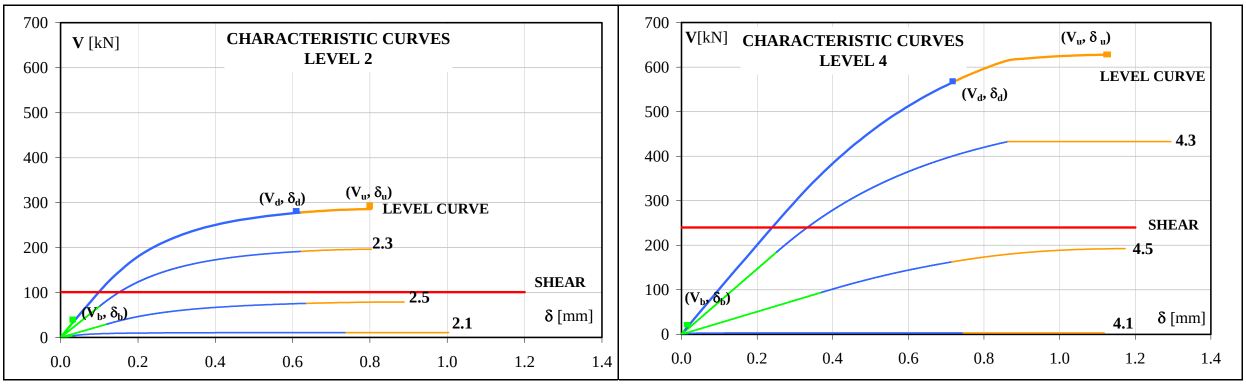


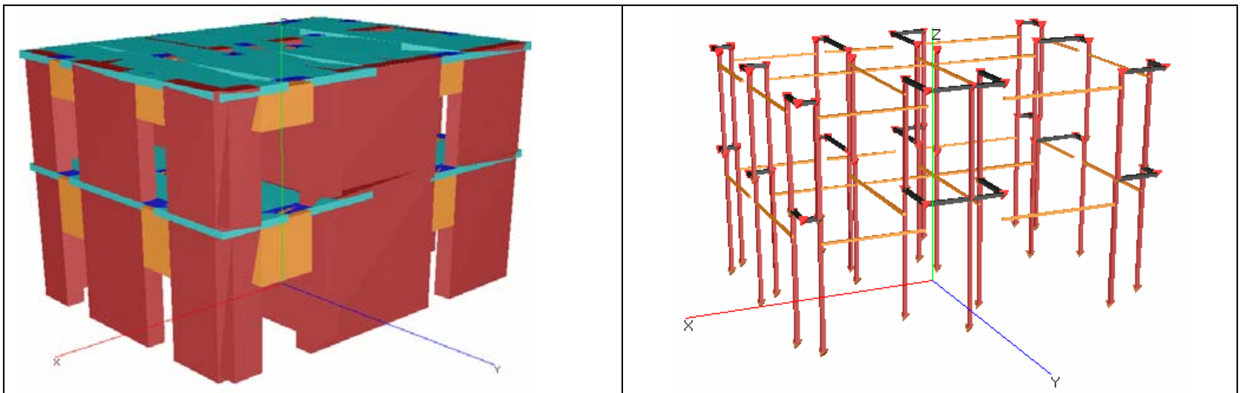
Figure 4: Characteristic curves at each level.

a) a first straight line, in the proportional elastic field, where sections are fully working;
b) a second curved line, in the non proportional elastic field, where reacting sections are reduced;
c) a third non linear branch, in the plastic field, related to the ductility of the panel.

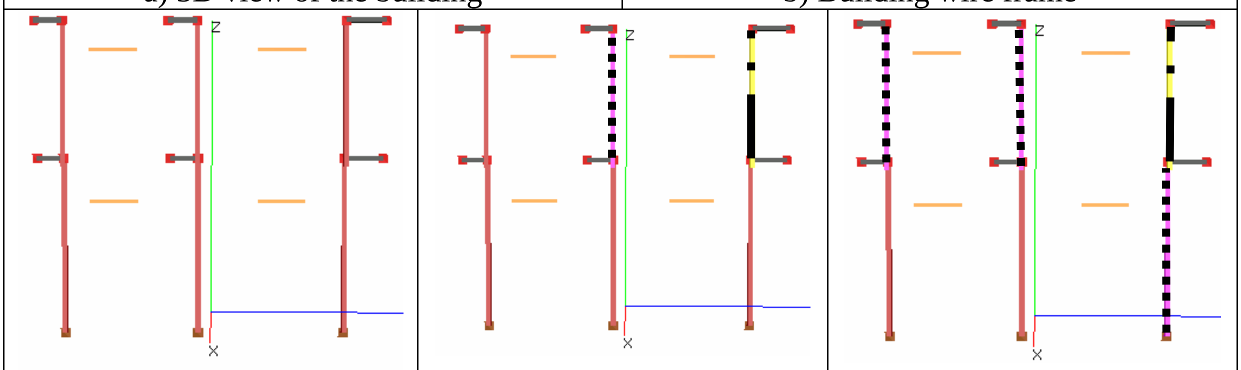
Subsequently, for each wall and each storey, the characteristic curves representing the relationship V-δ of the whole floor, have been determined. These curves allow to assess the maximum shear force borne by the level corresponding to the seismic design action. These curves stop at the displacement for which the weakest panel, belonging to each floor, is at failure.
Finally, the equilibrium of the spandrel strips has been checked, with a tolerance of 5% of the maximum acting stress.

Application of the SAM Method

For this numerical application the same geometrical, mechanical and load model has been implemented in the computer code available on the market (http:// www. crsoft.it/andilwall/andilwall.aspx).
In Fig. 5.a a 3D view of the building, and in Fig. 3.b the bare frame are given. A selection of some deformed shape of the wall of Fig. 2, following a static non linear analysis, are shown in Figs. 5.c, 5.d and 5.e; the magenta colour (dotted line) indicates that the pier panel collapses for eccentrical loads in both the extremities of the wall, the yellow colour (line-point line) that only in one extremity of the panel the limit value has been reached. Shear mechanisms are not detected.



a) 3D view of the building b) Building wire frame



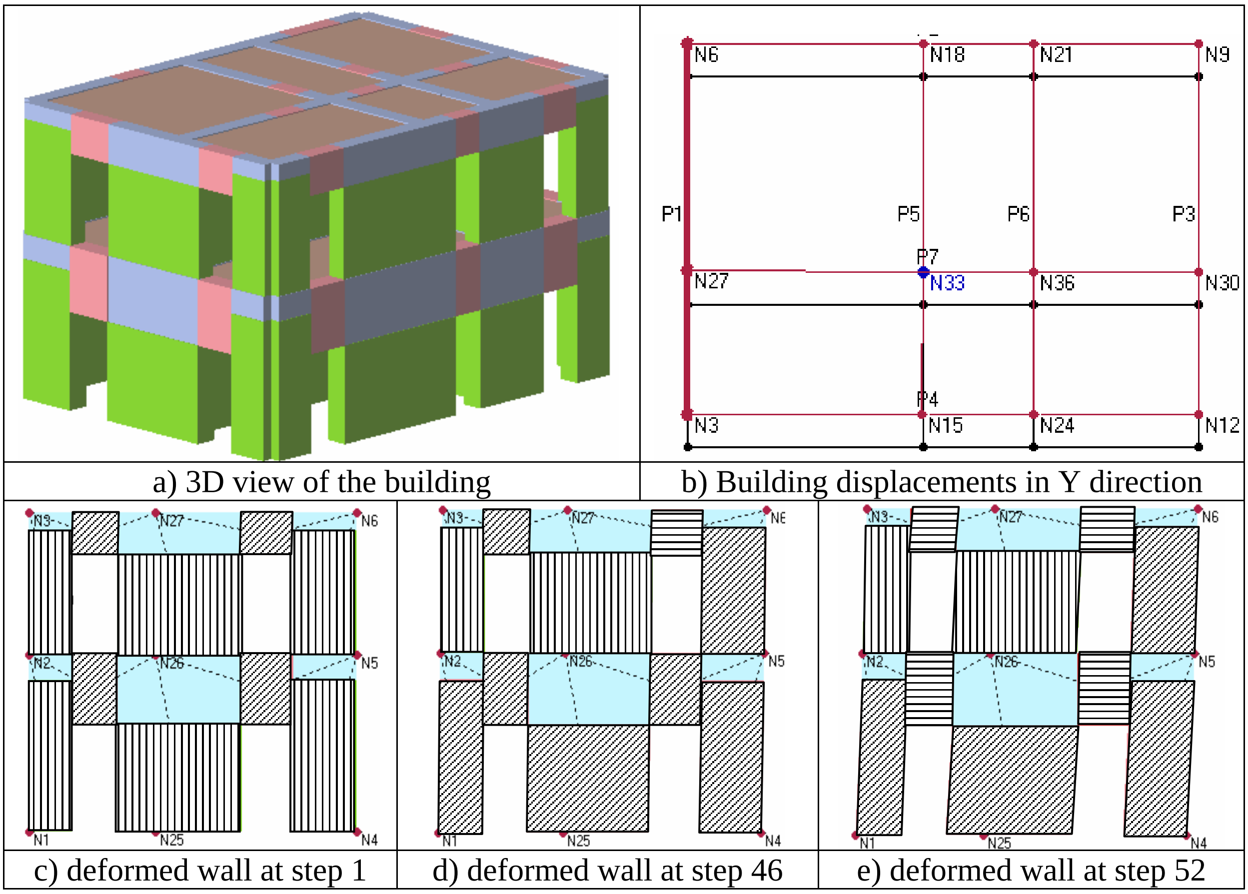
c) deformed wall at step 1 d) deformed wall at step 225 e) deformed wall at step 245

Figure 5: Building and wall model according to SAM.

The computer code allows to perform sixteen different checks, varying the following parameters: direction and sign of the applied actions; accidental eccentricity of the application point of actions, and typology of force distribution. Since the possibility of executing seismic checks taking into account the linear and non linear behaviour of the spandrel panels in the elastic field, both the two kinds of analyses have been performed. The results show how, at least in this case, the different behaviour of the spandrel strips is not excessively influent on the global behaviour of the structure. Furthermore, in both the applications, the displacement values for damage and ultimate limit states are very close.

Application of the 3MURI Method

The 3MURI computer code was then used (http://www.stadata.com). A 3D view of the building (Fig. 6.a), the building displacements according to the Y direction (Fig. 6.b) and a selection of deformed shapes of the wall of Fig. 2, are shown (Fig. 6.c, 6.d and 6.e). The colour green (vertically striped) means that the panel is undamaged, the pink one (diagonally striped) that the plastic limit state for bending and axial load has been reached, the red one (horizontally striped) that the collapse occurred for composed flexural and axial load. The cross panels, in light blue (plane), are not affected by failure mechanisms.



a) 3D view of the building | b) Building displacements in Y direction

c) deformed wall at step 1 | d) deformed wall at step 46 | e) deformed wall at step 52

Figure 6: Building and wall model according to 3MURI.

The computer code allows twenty-four different checks varying in function of the following parameters: direction and sign of the external action, accidental eccentricity and distribution of horizontal actions.

Two different applications, taking into account a seismic force distribution proportional to the first vibrating mode and another one proportional to masses, assuming zero eccentricity, have been performed. The conducted applications gave similar results, both in terms of stress and strain of masonry panels and failure mechanisms. Finally, a third application changing the geometrical model in the aim of comparing it to the one proposed by RAN, neglecting the variation of the height of the pier and cross external panels, has been performed.

COMPARISON OF RAN, SAM AND 3MURI

Once that the three kinds of analyses were completed, a comparison of the methodologies is carried out and discussed in the following.
The first difference among the procedures is the geometrical modelling of the panels in horizontal and vertical regular walls. The RAN method considers cross panels with a geometry equal to the intersection between spandrel strips and pier strips. The SAM and 3MURI, conversely, follow the modelling proposed by Dolce (1991), which allows a reduced height for cross external panels. This is due to the survey of some cross external panels which, being less confined than the internal ones, are more subjected to damages in case of earthquakes.

Regarding mechanical characteristics, the parameters required by 3MURI are practically the same of RAN whilst SAM, being mainly targeted to artificial stones, requires a more refined calibration in the perpendicular and parallel directions of load.
Furthermore, the possibility for the three methods of considering, inside the masonry walls, tensile resistant elements made of material different from masonry (beam elements, chains, etc.) can be investigated.

Another comparison of the methods is represented by collapse mechanisms of pier and spandrel panels. The 3MURI and SAM methods have not detected shear collapse mechanisms in any of the panels, but only mechanisms of flexural-axial load failure or uncracked panels. Conversely, in the application of the RAN method a shear collapse mechanism has been detected in the lower central pier, and flexural-axial stress mechanisms in the other panels of the wall. The presence of this kind of collapse mechanism revealed in the RAN method for the stocky panel appears more appropriate with the geometry and the load condition of the panel. It has to be admitted that a general disagreement about the failure modes is detected. For example, in the stocky panel at second floor 3MURI does not predict the failure, whilst for SAM and RAN the collapse occurs for flexural-axial load, due to the small amount of load.

Finally, a quantitative comparison of the three methodologies has been performed. Some non-linear static analyses of the whole structure, were carried out. In Fig. 7, the respective capacity curves (forces proportional to the first mode of vibrating, versus along +y direction, absence of additional accidental eccentricity between the mass and the stiffness centroids), are shown. As it can be observed, the maximum value of the base shear according the three methods is about 1600 kN. Some differences, however, are noticeable in the evaluation of the stiffness and the ductile branch. It is opinion of the authors that the RAN method does not compute the same ductility as the other two methods since the non-linear analysis has been conducted in force and not displacement control as SAM and 3MURI. About the global stiffness of the building in the elastic field, unfortunately the three methods give disagreeing results. The same considerations can be made for the analyses performed according to the uniform

distribution of the loads, except the fact that the peak load is about 7% higher than the triangular distribution.

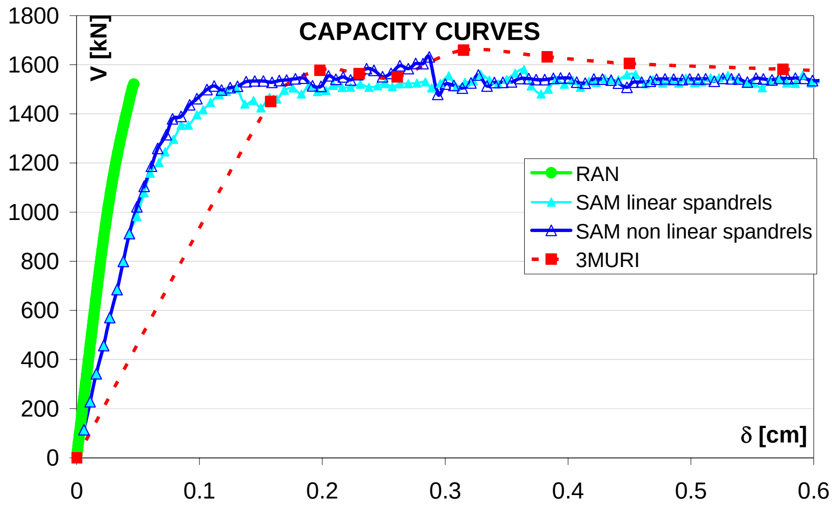


Figure 7: Capacity curve with RAN, SAM and 3MURI.

Finally, it can be observed that, whilst SAM and 3MURI, being implemented in computer codes, request the use of a "close" calculus, the RAN method allows the application through the use of spread-sheets. This implies a simpler use of the methodology and the possibility of adapting it to specific needs, although it is more time consuming.

CONCLUSIONS

This paper seeks in detail some well-known analysis methods using on masonry structures macro-elements. The hypothesis at the base of each, the procedure adopted, the criteria and the applicative capabilities have been analyzed. In particular, methods with similar characteristics that allow the execution of static non linear seismic analysis have been selected.

After a deep examination of these methodologies, it can be assert that the modelling through macro-elements allow a remarkable reduction of the degrees of freedom of rather regular structures. Consequently, the possibility of analyzing complex building with a computational effort considerably reduced in respect to Finite Elements is limited. Unfortunately, FE methods, at least for applicative purposes, are still characterized by many uncertainty for a correct modelling. Conversely, macro-element methods requires further analyses for the modelling of spandrel strips an buildings characterized by irregularities.

Another limit of these methods is represented by the hypothesis of no-tension behaviour of masonry. This kind of assumption, if from one side allow a further simplification of the procedure due to the difficulty of determining the experimental tensile strength offered by masonry, from the other side can lead to non sufficiently realistic results of the behaviour of structures both for the ultimate strength capacity and failure mechanisms of the elements.

Beyond the limits above cited, it can be affirmed that, for the applicative simplicity and the plurality of offered possibilities, macro-element methodologies are certainly a valid solution for the practitioner in the case of analysis of simple masonry residential buildings.

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