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NON LINEAR BEHAVIOUR OF MASONRY BUILDINGS UNDER SEISMIC ACTIONS

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ABSTRACT

We present a modelling with frames for masonry buildings endowed with rigid floors and R.C. platbands that is a result carried out in Naples with P.P.Rossi.
The modelling, considering the cracking of wall panels, allows to follow, by an iterative analysis, the non linear behaviour of this kind of buildings and underlines the progressive stiffness reduction of the structure by a continuous change of its geometry.
We carry out a comparison between the experimental results obtained by a laboratory test and the theoretical predictions provided by the proposed modelling.

KEYWORDS

Masonry buildings ; Non linear analysis ; Panel cracking

INTRODUCTION

By the term of masonry buildings we denote building typologies belonging to the entire history of construction up to the first decades of the 1900, when the reinforced concrete technology first integrated and then almost entirely replaced the load bearing function of masonry in buildings.
With time, masonry buildings exhibited different structural typologies based on constructive techniques and static principles that have been evidence - time after time - of different structural knowledge and technological levels as well as of different historical periods. Hence, the necessity of classifying the various typologies from their static behaviour standpoint in order to propose analysis models suitable for interpreting their specific behaviour.
The "masonry material", formed by quoins bound by mortar often of reduced affinity, is frequently reported in the scientific literature as a material endowed with unilateral strength just because of the mild and precarious cohesive capacity of the mortar.
With reference to a well known structural classification (Pagano, 1970) this study will analyse the more recent building typology consisting of masonry load bearing walls and of stiff floors, resistant in their own plane, formed by cement and brick floor material.
The wall box results in such a case connected with each stress-resistant floor which produces an overall behaviour of the building comparable to that of a spatial frame with rigid floors where the walls represent the frames.

The wall where the openings are regularly arranged represents a plate with holes which can be usefully outlined (Morlando and Ramasco, 1984) as a frame provided with rigid segments at its ends for which shear deformability is not negligible at all when compared with the bending one.
The presence of curbs and of platbands on the open rooms gives bending rigidity to the horizontal masonry bands independently of the axial force, while only compression can ensure bending capacity to the wall panels.
The technical and scientific attitude also reflected by regulations suggests that this static behaviour is the result that has to be achieved by means of interventions of static rehabilitation that can be applied also to the most ancient buildings belonging to different typologies.
The deformability of this building - at least vis-à-vis weak horizontal forces - appears comparable with that of normal RC framed buildings (Lenza, 1987).
When the horizontal forces increase, and thus with the growing eccentricities of normal forces in the wall panels, the first cracking occur. Hence, cracks make the building more and more deformable up to the collapse caused by excessive compression or, more frequently, by the turnover of wall panels of a storey.
The first non linear model deriving from the original frame has been described in (Lenza, 1989) and is formed by reticular patterns that replace the beams.
The limitation of this model belonging to the so-called "variable geometry models" (Lenza et al. 1989)- that thanks to their simplicity have also permitted dynamic analyses (at the pitch) in a non linear system (Lenza et al., 1991) - consists in the reticular schematization where the structural fibres of the sections are - for the sake of simplicity - concentrated in linear elements which as a whole give the same deformability as the entirely reacting wall element.

A MODEL MORE CONSISTENT WITH REALITY

Mechanical Features

The type of modelling described in this paper still belongs to the "variable geometry models" and - making always reference to the class of mixed structure buildings - is based on the frame schematization of the masonry wall. This solution suggested by the natural succession of voids and solids that often characterises the buildings of the last centuries, permits to identify horizontal masonry bands and panels as it commonly happens in the case of normal structures in reinforced concrete (FIG. 1-2).
The intersection areas between horizontal masonry bands and panels due to the confinement exerted by the contiguous elements are considered as undeformable and indefinitely resistant.
This assumption appears less credible for the joints at the wall ends due to the partial confinement restraint action which makes it useful to consider also the hypothesis of a smaller extension of the rigid areas.
The generic panel delimited by the contiguous rigid areas - even though it results to belong to the sphere of bidimensional elements due to its geometric dimensions - is schematised by beam elements only.
It is necessary to consider the influence of shear stresses on the deformability which permits to better simulate the behaviour of a "squat" element such as the masonry panel by means of beam elements.

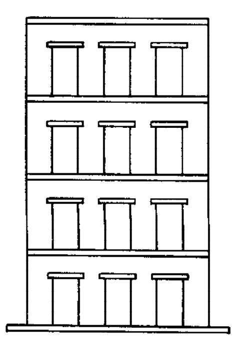

Fig. 1 Typical example of a slender masonry wall with regularly distributed voids and solids.

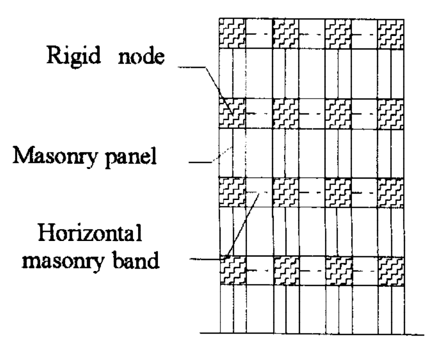

Fig. 2 Schematization of the masonry wall as a frame; identification of horizontal masonry bands, panels and rigid areas.

If we assume that the dead load of the masonry panels is brought back to the level of the floors, the axial force is kept constant along the wall panel and its eccentricity varies linearly. It is possible to define the sections of the really reactive panels with reference to the theory of beams subjected to compressive and bending stresses formed by a material which is not resistant to tensile stress.

The individual wall panel is ideally devided into three segments of which the central one, corresponding to that part of the panel that fully reacts, has a constant section whereas the other two, which are possibly cracked, have a variable section linearly along the axis.

The generic masonry panel will exhibit unchanged resistant sections compared to the initial ones due to the stresses included in the central core of the section (FIG. 3), while it will reduce its size and change its shape vis-à-vis cracking-producing stresses, i.e., vis-à-vis tensile stresses at the edges greater than the pre-fixed maximum ones (FIG.4). Therefore, the width of the panel's resistant section will be equal to the initial one both for zero tensile stress and for a finite value stress, if the resultant of the external forces is constrained in the central core of inertia of the section:

$\sigma_t \geq 0$ $$d=1 \tag{1}$$

while it will be calculated by the mentioned formulas derived from the beam theory if the panel cracking has occurred (Abrams)

$\sigma_t=0$ $$d = \frac{3l}{2} - \frac{3M}{P} \tag{2}$$

$\sigma_t>0$ $$d = \frac{P}{\sigma_t} - \sqrt{\left(\frac{P}{\sigma_t}\right)^2 - \left(\frac{3Pl - 6M}{\sigma_t}\right)} \tag{3}$$

where:

- P is the normal force
- M is the bending
- t is the tensile stress strength
- d is the new width of the generic section

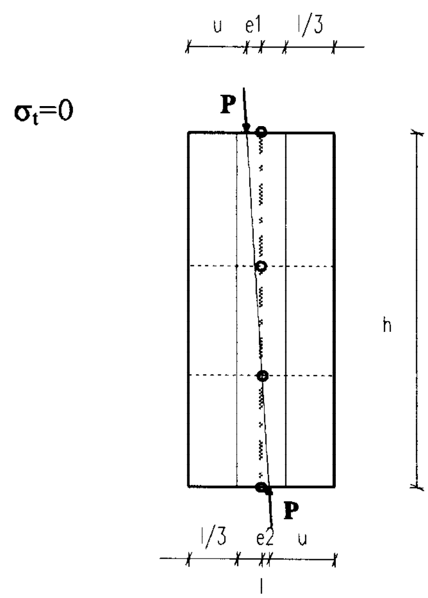

Fig. 3 Masonry panel with the pressure center included within the central core of inertia of the section

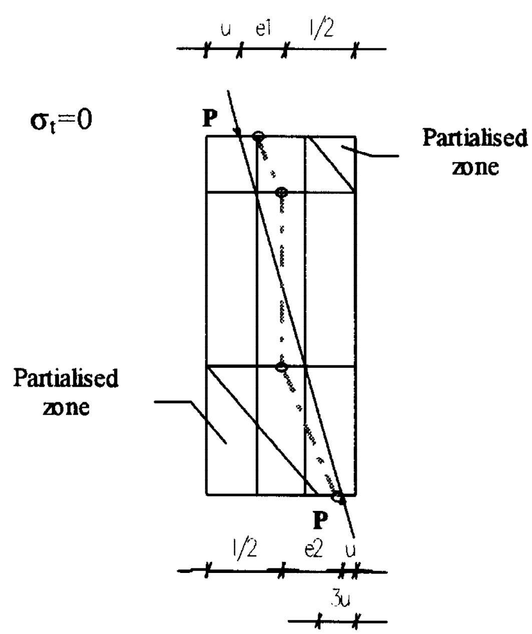

Fig. 4 Cracking of the masonry panel due to compression and bending stress with the pressure centre is external to the central core of inertia of the section.

When the stresses increase, the size of the extreme sections of the wall panels becomes smaller neglecting the part where the tensile stress exceeds the dead zone, while the extension of the central segment changes in order to represent the resistant section zone.
When cracking progresses, the resultant of the loads at the ends of the wall panels tends to slide towards the compressed edges; at the same time the panel tends to assume the geometry of a strut having a section restricted at the ends and a sloping axis (FIG. 4-5)

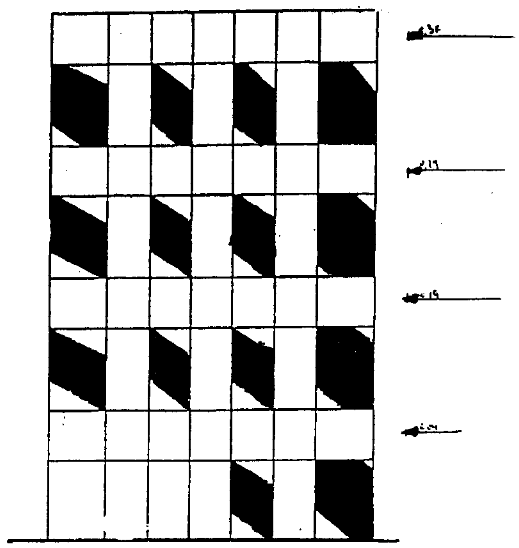

Fig. 5 Resistant configuration of the panel near to the collapse of the brick frame.

However, if by means of frame schemes the rigid end segments and the shear strain make the analysis of the plate with holes reliable from the deformation viewpoint as already shown in (Morlando and Ramasco 1984) the same reliability has not been demonstrated when considering that the extension of cracking holds on the basis of the linear law deriving from the classical hypothesis of the plane conservation of the section assumed for the beam.

Calculation procedures

In the modelling proposed, the resistant geometry results to be function of the tensile state in turn depending on the structure geometry, so that the solution must be iteratively found.
At present a calculation procedure has been developed aimed at performing a non linear analysis on a personal computer.
Such a procedure will represent a basic modulus for a dynamic analysis that- due to the marked non linearity of the model will necessarily have to be carried out by direct step by step integration of motion equations.
In each iteration the pattern of the previous iteration is taken as the reacting pattern and the latter is modified on the basis of the resulting eccentricities. The iterative procedure ends when the reactive pattern assumed is confirmed by the calculations. Thus, for each loading step the stresses of all structural elements are known, in particular in the K.C. seams and platbands.
The uniqueness of the solution found derives from the assumption of the plane conservation of the sections "borrowed" from the beam theory.
When the horizontal actions, and thus eccentricity, increase, the procedure does not lead to any solution. Under these conditions equilibrium is impossible. This corresponds to a state of internal lability deriving from the unilateral resistance of the material. Lability reveals itself as the sliding or turnover of the floor panels. In each loading phase the admissibility of the compression tensile state must be controlled which can anticipate the crisis due to the structure lability.
The automated calculation procedure uses as the basic modulus for the solution of the model within the framework of a linear analysis a finite elements programme that is iteratively recalled and used (Wilson and Habibullah) The procedure is completed by a routine that, once having acquired the results of the latest

analysis, changes the reacting geometry of the wall panels on the basis of the calculated eccentricities and replaces the latest model with the updated one, thus permitting, in the short run, to reach for each loading step the solution of the problem subject to conditions (1)-(3).
It has been made operationally possible to take also into account the reduced tensile stress strength of the masonry considered as prone to brittle fracture.

TESTS OF THE PROPOSED MODEL

Intermediate Scale Experimental Structure.

The tests have involved a model of an intermediate scale masonry building built at the Engineering Faculty of Naples which is still under study.
The masonry constructed with small blocks of yellow tuff - squared and bound by cement mortar - is connected with each storey by floors built in compliance with rigidity and strength assumptions, using highly resistant cement mortar, the same as in the case of the platbands placed on the openings.
The four-storey model consists of two identical parallel walls (FIG.1)connected with each storey by rigid and resistant floors having the following geometrical features:

Table 1 Geometrical features of the model of the masonry building under study.

total height (cm)	total length (cm)	height of the panel 1st floor (cm)	height of the other panels (cm)	basis of the panel (cm)	length of the horiz. masonry band (cm)	height of the horiz. masonry band (cm)
158	91	22.5	24.5	13	13	15

In a first series of lab tests on masonry panels (they too on a small scale) values have been derived concerning the breaking axial strain under tensile stress and compression, while due to some problems arising during the tests in calculating a reliable value for the Young modulus it was decided to rely on empirical and regulatory relationships for a first evaluation. The traverse modulus has been assumed equal to 1/6 of the longitudinal one

Table 2 Main mechanical characteristics of the panels of the model under study

σc (N/mm²)	σt (N/mm²)	Young Modulus (N/mm²)	Traverse modulus (N/mm²)
3	0.3	2000	333

The model undergoing a vertical load of 1.75 KN at the last floor and of 1.25 KN at the others, has been stressed with horizontal forces, in correspondence with the floors. These forces are equal for each floor and are applied almost statically according to a monotonous law except for short initial loading and unloading cycles.
The crisis, basically occurring as turnover of the first order wall panels, occurred vis-à-vis a horizontal load of about 0.95 KN per floor, with a deformation line at the last floor shown in FIG.6.

Numerical Results Referred to the Structure Subjected to the Tests

The first investigation conducted within the program checking on the reliability of the model employed made the simplifying hypothesis of a non-tensile stress resistant material, an often realistic assumption concerning the real capacities of masonry.
In the light of the comparison made with another analysis performed by considering the effect of a reduced

resistance to the tensile stress evaluated by lab tests and introduced as stated in (3), the above assumption does not seem to have remarkable affected the previous results though it has caused a small increase in the collapse load (FIG.6).
The comparison between the experimental data collected by the loading test described above and the theoretical predictions obtained through calculation models is shown here in terms of horizontal displacements of the last floor since, basically, the differences and the observations made hold in the same way as for the other floors.
The result of the non linear, iterative and numerical analysis sufficiently correspond to the real shape of the experimental curve due to the progressive reduction in rigidity of the structure caused by panel cracking while according to the authors the last part is not affected by the uncertainty contained in the definition of the Young modulus which at present has not been evaluated by lab tests and has been set according to regulations.
The experimental curve is then compared with the one derived from a previous model of reticular calculus (Lenza, 1989) with frame schematization of the masonry walls. The deformable part of the frame is conceived as a set of pendulum beams having unilateral strength organised in bridles and crossed diagonals that mechanically correspond to the relative masonry element. When the load increases the number of reacting beams decreases and the structure, having a variable resistant structure, springs out of the shape and exhibits a decreasing rigidity.

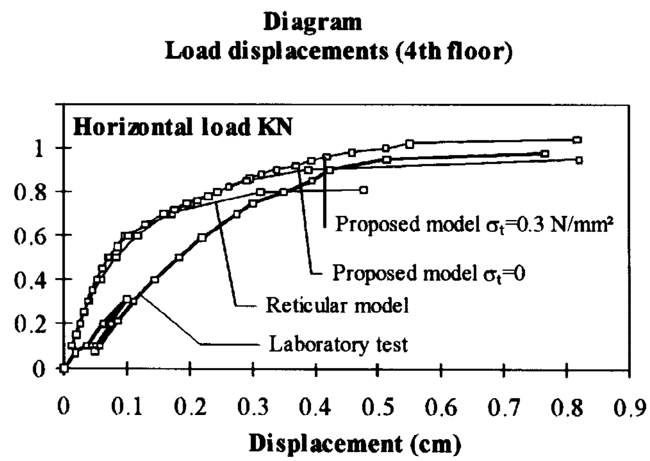

Fig. 6 Comparison between experimental data and theoretical results. The reliability of the production is verified in terms of displacements of the last floor when the floor loading force varies.
The numeral analysis is performed both in the assumption of a material that is not tensile stress resistant and in the case when there is a mild tensile stress

A further analysis was made in order to evaluate the influence of the extension of the areas considered as rigid. In fact, if it may seem reasonable to consider as very rigid and resistant the intersection areas between masonry panels and horizontal bands in the external joints - where actually the confinement action by the contiguous elements can lead to such an evaluation. This assumption clearly does not hold in the end joints of the structure, where part of the intersection area is not confined.
It has therefore been decided to consider a reduction in height first by 25 per cent and then by 50 per cent, only in relation to the latter.
The results have shown the weak influence of such a parameter on the structural response (FIG. 7).

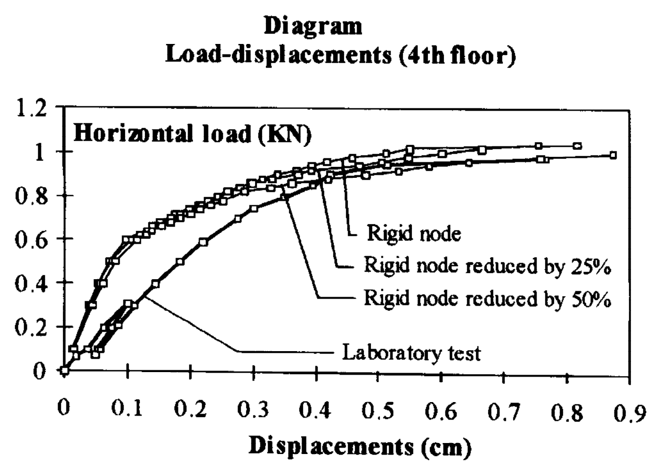

Fig. 7 Variation of the structural response vis-à-vis a reduction in the extension of the rigid zones of the end knots which are not confined on all sides.
The influence of such a parameter appears only as a small change in the response without considerable differences in the first loading stages

POSSIBLE DEVELOPMENTS

Modelling that studies the influence of panel cracking on the structural behaviour has shown to effectively evaluate the behaviour of the structure by revealing a good concordance even in predicting the collapse load corresponding to the turnover of the wall panels of one storey.

It should be noted that the comparison made with the experimental results shows a certain margin of error basically due to some uncertainty in the definition of the Young modulus.

Further developments of modelling will have to aim at checking its reliability in the case of "squat" masonry buildings by introducing a calculation routine predicting different collapse mechanisms.

REFERENCES

PAGANO M. (1970). Teoria degli edifici: edifici in cemento armato, Liguori Editore, Napoli

MORLANDO G. and R. RAMASCO (1984). In tema di verifica sismica degli edifici in muratura, Quaderni di Teoria e Tecnica delle strutture, Università di Napoli - Istituto di Tecnica delle Costruzioni n° 562

LENZA P. (1987). Prospettive di analisi modale per edifici in muratura, Ingegneria Sismica, Pàtron Editore, n° 3

LENZA P. (1989). Un modello reticolare per l'analisi non lineare di edifici in muratura, Ingegneria Sismica, Pàtron Editore, n°1 -

LENZA P., B. CALDERONI and M.PAGANO (1989). Attuali prospettive per l'analisi non lineare degli edifici in muratura, Proc.of the 4° National Congress "L'Ingegneria sismica in Italia", Milano,

LENZA P., A. GHERSI and B. CALDERONI (1991). Sul comportamento dinamico degli edifici in muratura, Proc.of the 5° National Congress "L'Ingegneria sismica in Italia", Palermo

ABRAMS D.P. and W.XU: Evaluation of in-plane shear resistance of cracked, unreinforced brick walls

WILSON E. and L. A.HABIBULLAH: SAP90, Programma di calcolo agli elementi finiti

LENZA P. and P.P.ROSSI (1995). Una modellazione a geometria variabile per l'analisi non lineare di edifici in muratura, Proc.of the 7° National Congress "L'Ingegneria sismica in Italia", Siena 25-28 September