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NUMERICAL SIMULATION OF THE EXPERIMENTAL BEHAVIOUR OF MASONRY PANELS AND WALLS UNDER HORIZONTAL LOADS

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ABSTRACT

The response of masonry panels and walls under vertical and lateral loads, calculated through the no-tension, multi-fan panel element is compared with test results. Two experimental researches on panels and one on a full-scale brick masonry building prototype are considered in this study. The numerical curves of lateral load vs. lateral displacement are compared with the experimental skeleton curves under cyclic loading, and conclusions about the accuracy and the field of applicability of the element are drawn.

KEYWORDS

Masonry; panels; walls; tests; numerical models.

INTRODUCTION

Safe and economic retrofitting of masonry buildings requires an accurate prediction of their response under seismic actions. Therefore, comparisons between numerical and experimental results represent a fundamental step in evaluating the reliability of theoretical models.

When modelling a material like masonry, which exhibits a markedly non-linear and anisotropic behaviour, accuracy and economy of the model are often in antithesis. Apart from non-homogeneous models, which are able to represent the real discontinuities of the material but are too expensive, homogeneous models remain the compulsory way for the analysis of usual structures. Two extreme approach are presently available in the field of homogeneous models. On one hand, refined models use non-linear plane stress elements, very expensive from the computational viewpoint and difficult to use, particularly owing to the determination of mechanical parameters and interpretation of results. On the other hand, models which are simplified and easy to use are, as a result, excessively rough and unable to take into account the effects of structural irregularities and of some widely used erection and retrofitting techniques.

The "no-tension multi-fan panel element" (Braga and Liberatore, 1990), hereafter simply named "panel element", is a reasonable compromise between the two aforementioned approaches. It is based on a no-tension formulation using a multi-fan stress pattern in a rectangular part of masonry with free lateral edges (panel). Only one single element is required to model a panel. The material behaviour is assumed linear elastic in compression (Dhanasekar *et al.*, 1985) and non-reacting in tension. The occurrence of crushing can be detected on the basis of the maximum compressive stress in the panel. The upper and lower faces of the panel are assumed rigid. For given displacements and rotations of the upper and lower faces, the multi-fan pattern is determined minimising the total complementary energy of the panel. The panel element is able to model the local structural characteristics with acceptable accuracy and, at the same time, very low computational burden. These features make it well suited for the analysis of complex masonry structures. On the other side, it is unable to reproduce the hysteresis loops and the degradation of the material. Therefore, the load vs. displacement curve calculated through the panel element should be interpreted as the skeleton curve of the real hysteresis loops.

The simplifying assumptions on the material behaviour require a careful calibration of the mechanical parameters in order to fit the experimental response. The panel element only requires the identification of the Young modulus, and of the compressive strength if crushing is suspected, in contrast to other more complex material models which require several physical parameters, rarely available in practical analysis.

The aim of this study is to compare the response calculated by means of the panel element with experimental results. The investigation considers a number of panels and two full-scale walls with different openings arrangement which underwent various combinations of vertical and lateral loads.

SIMULATION OF TESTS ON PANELS

The results of two different experimental researches on brick masonry panels have been used to verify the numerical response of the panel element. These researches investigated panels manufactured using the same brick type, and with different height/width ratios, thickness and boundary conditions. Both the experimental researches studied the response of masonry panels under combinations of vertical and lateral loads in quasi-static conditions.

Panels tested at ELSA

The former experimental research was carried out at ELSA (European Laboratory for Structural Assessment) (Anthoine *et al.*, 1994). Two two-wythes panels, hereafter named W1 and W2, were tested. They have width d = 1.00 m, thickness t = 0.25 m and height h = 1.35 m (W1) or 2.00 m (W2). The panels are made up of bricks "Rosso Classico" manufactured by RDB Terrecotte SpA. The dimensions of the bricks are $250\times120\times55$ mm and their mean compressive strength is 19.72 MPa (Magenes, 1992). These bricks are widely used to repair and strengthen Italian historical masonry buildings. Hydraulic lime mortar was used. A schematic representation of the testing set-up is shown in Fig. 1. Two vertical jacks apply a vertical load of 150 kN to a horizontal steel beam fixed to the upper face of the panel. The lateral load was applied by a third jack acting in the horizontal direction. During the test, the forces of the vertical jacks were adjusted in order to keep constant the total vertical load and, at the same time, to keep the upper face of the panel horizontal. Clearly, the test conditions reproduce those of the classical racking test. The test was driven in control of lateral displacement. The test results consist of curves of lateral load H vs. lateral displacement u.

In order to compare experimental and numerical results more clearly, the skeleton curves have been extracted from the hysteresis loops of lateral load vs. lateral displacement. Data points have been selected graphically from the field of positive load and displacement, for a total of 10 and 9 data points for W1 and W2, respectively.

The model adopted for the numerical simulation makes use of a single 4-node panel element (Fig. 2). The model is completed by very stiff horizontal and vertical trusses whose purpose is to display the lateral load and the vertical base reactions, respectively. The rotations of the upper and lower faces of the panel have been restrained, and the vertical load kept constant. The numerical simulation has been carried out prescribing monotonically increasing lateral displacements to the upper face. The Young modulus E has been assumed equal to 3000 MPa for both W1 and W2.

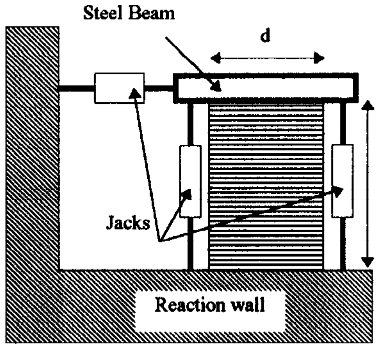

Fig. 1. Testing set-up at ELSA.

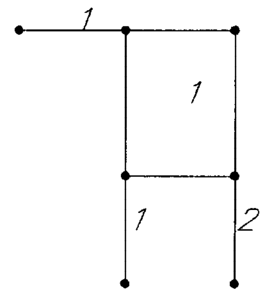

Fig. 2. Numerical model of the tests at ELSA.

The comparisons between numerical and experimental *H-u* curves are shown in Figs. 3 and 4 for W1 and W2, respectively. A good agreement between calculation and test can be noticed for W1 up to the maximum experimental load which is reproduced almost exactly by the model. Overestimation of the experimental response occurs at higher displacements. It can be ascribed to material degradation, which is not taken into account by the panel element. As regards panel W2, an excellent agreement can be noticed for $u \leq 3$ mm. At higher displacements, the model underestimates the experimental curve with errors below 10%. No clear explanation has been found for this behaviour.

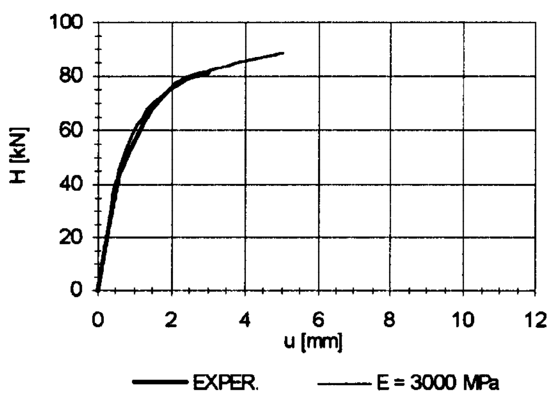

Fig. 3. Lateral load vs. lateral displacement (panel W1).

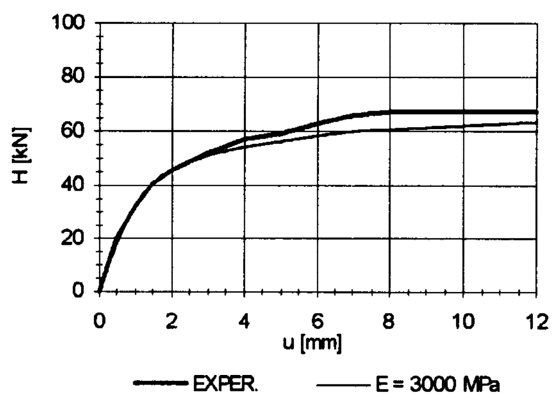

Fig. 4. Lateral load vs. lateral displacement (panel W2).

Panels tested at the University of Pavia

The latter experimental research was carried out at the University of Pavia (Magenes, 1992). Four three-wythes panels, hereafter named MI1, MI2, MI3 and MI4, were tested. They have width $d = 1.50$ m, thickness $t = 0.38$ m and height $h = 2.00$ m (MI1 and MI2) or 3.00 m (MI3 and MI4). Bricks "Rosso Classico" were used, together with hydraulic lime mortar with lime/sand ratio equal to 1/3 and compressive strength of 4.33 MPa. They underwent a shear-compression test in double bending conditions (Fig. 5). The initial vertical load was applied by means of two hydraulic jacks whose valves were subsequently closed. The initial vertical stress was equal to 1.2 MPa for MI1 and MI3, and to 0.4 MPa for MI2 and MI4. The lateral cyclic load was applied by means of a third jack under manual control acting on a RC base fixed at the bottom of the panel and sliding on a steel roller with negligeable friction. During the application of the lateral load, the valves of the vertical jacks were kept closed and the variations of oil pressure recorded. Notwithstanding the high stiffness of the jacks, small rotations of the lower face of the panel occurred during the test.

The complete digitized results for a total of more than 1000 data points per test were available for the simulation. The extraction of the skeleton curve from the hysteresis loops of lateral load vs. lateral displacement has been carried out numerically. For each data point, the values of lateral displacement u, lateral load H, vertical load N and base bending moment M were available.

The model adopted for the numerical simulation (Fig. 6) differs in some important features from that used for the panels tested at ELSA: the lateral displacement is prescribed at the lower — rather than upper — face, and this face is free to rotate. The simulation has been carried out prescribing monotonically increasing lateral displacements to the lower face. The corresponding forces recorded in the vertical jacks during the test have been applied to this face as well. E = 2600 MPa was assumed for all the panels.

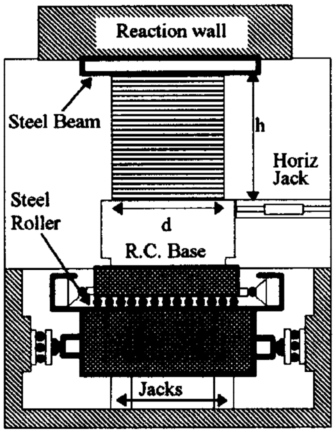

Fig. 5. Testing set-up at the University of Pavia.

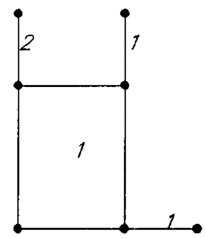

Fig. 6. Numerical model of the tests at the University of Pavia.

The comparisons between calculation and test are shown in Figs. 7-10 in term of curves *H-u*, *H/N-u*, *H-N* and *H-M*. An excellent agreement between calculation and test can be noticed for panel MI1. As regards MI2, the agreement is good. In the terminal part, the model underestimates the experimental curve with errors below 10%. The numerical and experimental curves of panel MI3 tally very well for $u \leq 6$ mm. At higher displacements, the experimental load is nearly constant, while the numerical load continues to increase up to 15%. This discrepancy can be ascribed to material degradation. Finally, panel MI4 exhibits a good agreement between calculation and test. A tendency to underestimate the experimental response, with errors below 7%, can be noticed at high displacements.

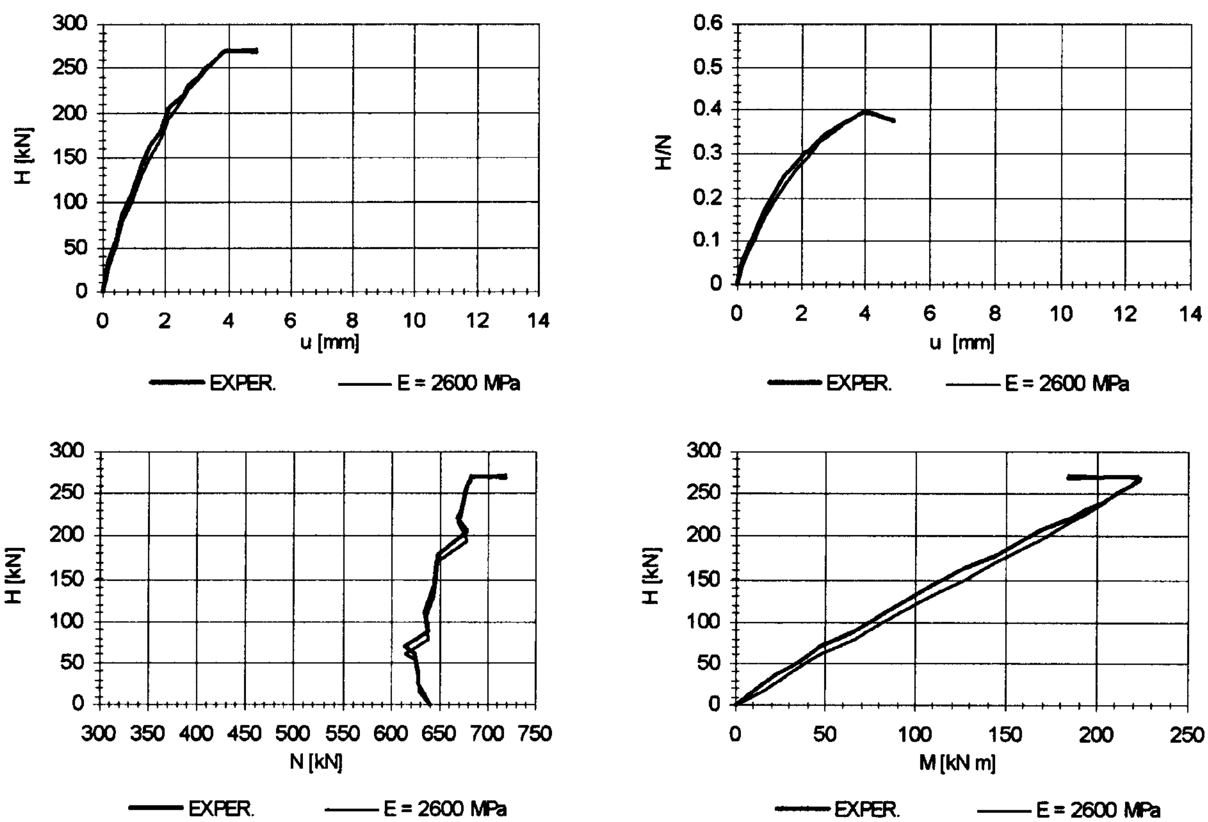

Fig. 7. Experimental and numerical results of the tests at the University of Pavia (panel MI1).

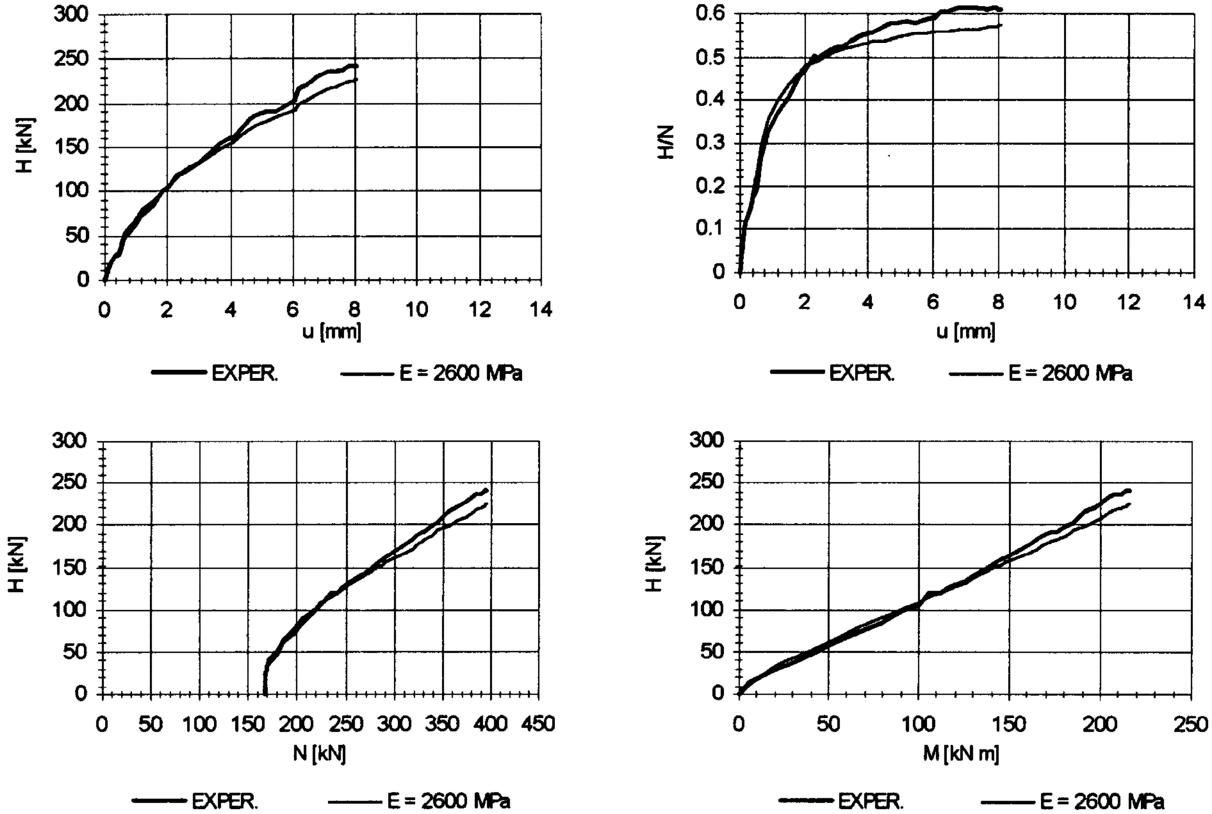

Fig. 8. Experimental and numerical results of the tests at the University of Pavia (panel MI2).

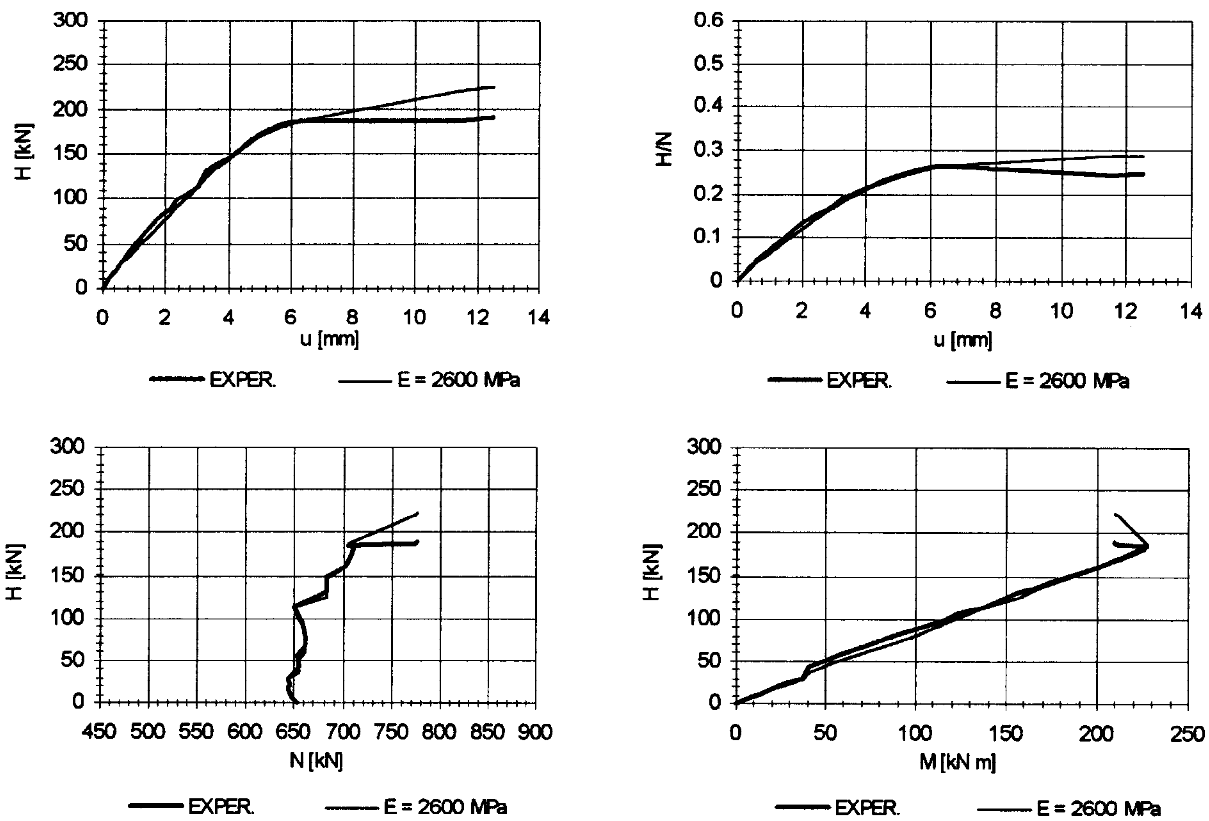

Fig. 9. Experimental and numerical results of the tests at the University of Pavia (panel MI3).

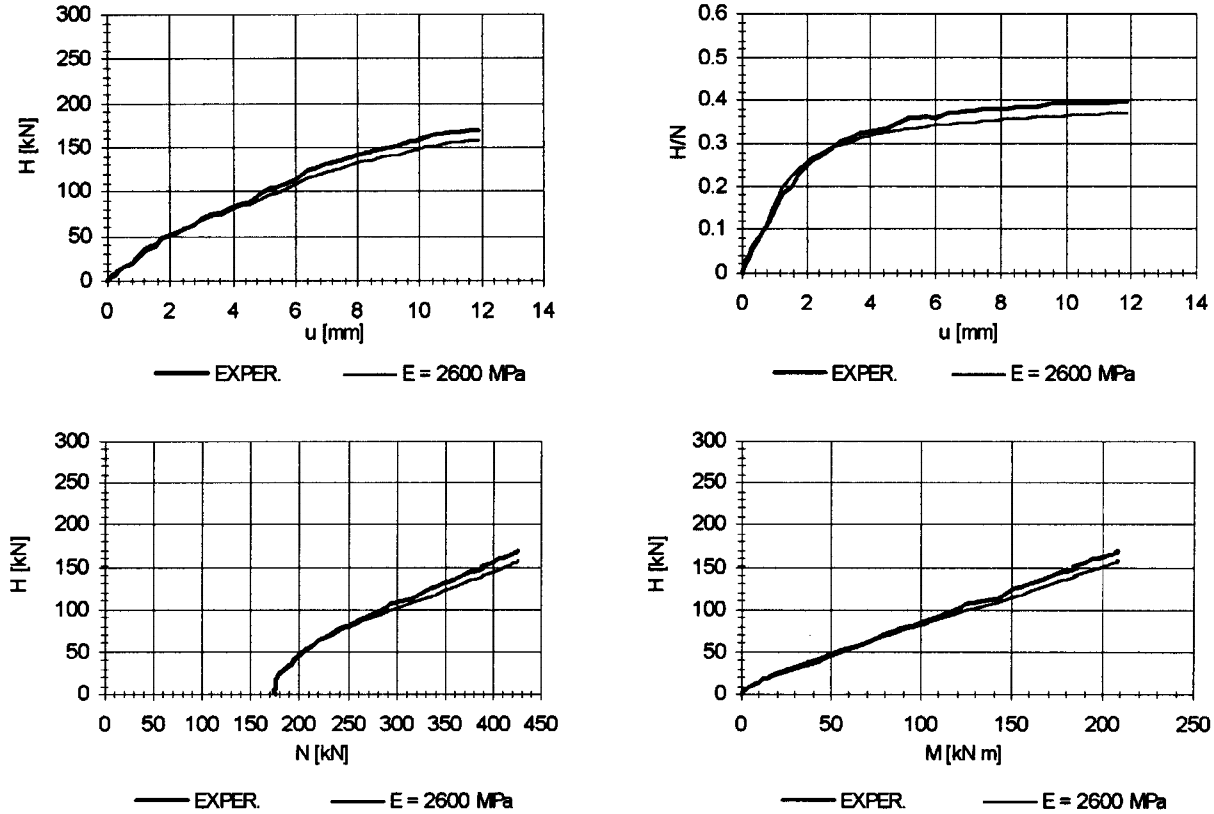

Fig. 10. Experimental and numerical results of the tests at the University of Pavia (panel MI4).

SIMULATION OF TESTS ON WALLS

The full-scale brick masonry building prototype tested at the University of Pavia is formed by two walls, named B and D. Wall B is connected to two orthogonal walls, named A and C. The plan dimensions are 6.0×4.4 m, and the height is 6.4 m. The test was driven in quasi-static conditions. For a detailed description of the prototype and of test results, the reader is referred to the literature (Calvi *et al.*, 1992; Magenes *et al.*, 1995).

The model used to *predict* the experimental response of the masonry prototype is shown in Fig. 11 (Braga *et al.*, 1995). The multi-fan element is used not only for the panels, that is the parts between the openings, but for all the rectangular parts in which the wall is decomposed. Using panel elements which assume that two edges are free is correct only for the panels between openings. However, since the displacement field and the base shear are mainly determined by these elements, the model can be regarded as acceptable, if only the global response is looked for.

Wall B presents additional problems due to the interaction with the orthogonal walls A and C. Since it was not easy to foresee whether the connection would be maintained during the test, two different models have been set up: (a) wall B alone and (b) three walls A, B and C constrained at the interfaces by the equality of vertical displacements.

A rigid link on the horizontal component of displacement at the storey levels has been imposed by constraint equations in order to account for the high axial stiffness of the steel bars which apply the lateral load to the walls. Horizontal trusses at the storey levels and vertical trusses at the base have been added in order to display the lateral applied loads and the foundation nodal reactions, respectively. Two values of E have been adopted for test predictions: 1491 MPa and 2094 MPa (Braga *et al.*, 1995).

Six drift values have been analysed (0.05%, 0.10%, 0.20%, 0.30%, 0.40% and 0.50%) for each of the three models considered (wall B, walls A+B+C, wall D) and for each value of the Young modulus (E = 1491, 2094

MPa), for a total of 36 cases. The lateral displacement at the first storey level has been determined by the *regula falsi* iterative scheme in order to make the lateral loads at the two levels equal, according to the test.

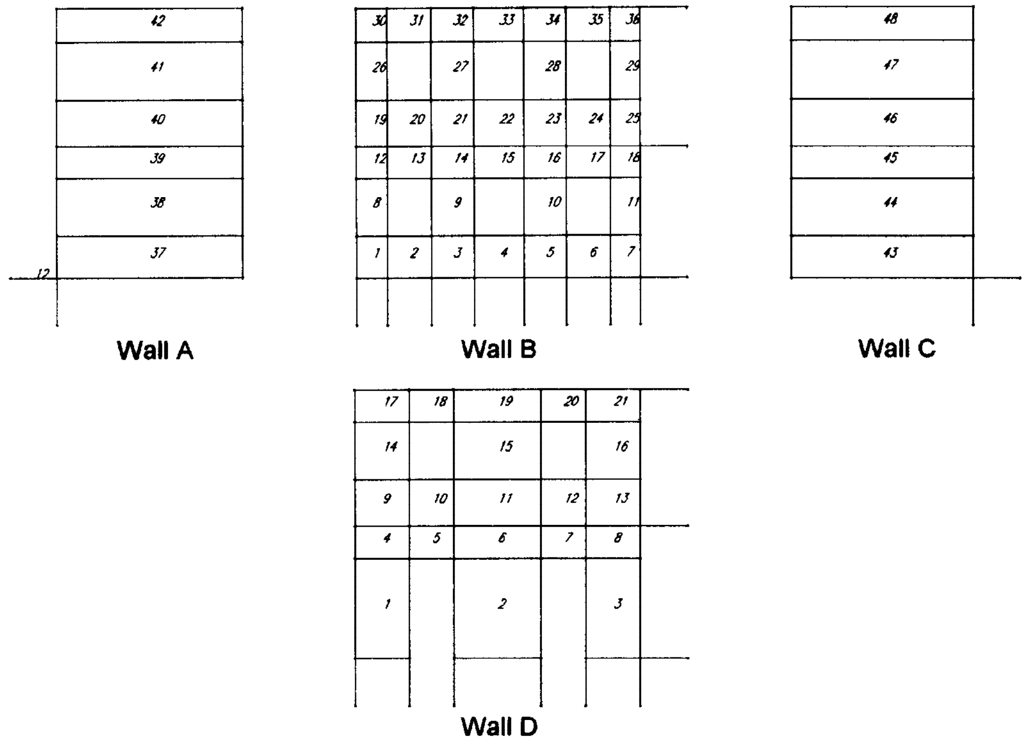

Fig. 11. Numerical model of the full-scale brick masonry building prototype.

The experimental and numerical curves of base shear H vs. top displacement u are shown in Figs. 12 and 13 for walls B and D, respectively. Underestimation of the experimental curves occurs at low displacements, and overestimation at high displacements. The maximum experimental base shear of wall B is equal to 134 kN and takes place at u = 10 mm. At this displacement the different models analysed present errors ranging from -5% to 17%. At higher displacements the numerical curves continue to increase, while the experimental curve denotes material degradation. As regards wall D, the maximum experimental base shear is equal to 150 kN and takes place at u = 11 mm. The numerical curves exhibit an overestimation ranging from 3% to 11%. At higher displacements, the experimental base shear is nearly constant, while the numerical one continues to increase.

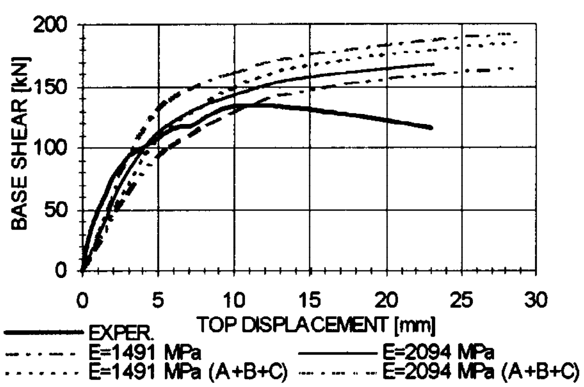

Fig. 12. Base shear vs. top displacement (wall B).

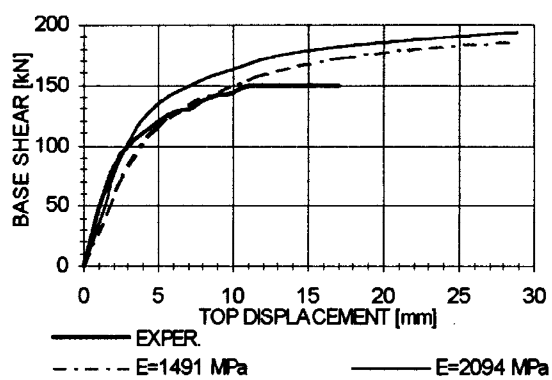

Fig. 13. Base shear vs. top displacement (wall D).

CONCLUSIONS

The main advantage of the panel element is the very low number of degrees of freedom, compared to that usually required by standard finite elements. The corresponding computational burden is considerably small and permits the analysis of large and complex masonry structures.

The ability of the panel element to reproduce the skeleton curves of masonry panels under cyclic lateral loads, up to the maximum lateral load, is verified through comparisons with the experimental response of panels tested at ELSA and at the University of Pavia. However, the assumptions on the material behaviour (no-tension and linear elastic in compression) do not permit to evaluate the starting and the development of material degradation.

The prediction of the experimental results of the full-scale brick masonry building prototype tested at the University of Pavia demonstrated that the panel element can be effectively employed to model masonry walls. However, besides the drawbacks described for the panels, modelling masonry walls involves two additional sources of inaccuracy. The former is the assumption that the two non-free edges of the panel are rigid, and leads to overestimate the base shear. The latter emanates from the interaction with the orthogonal walls. Since no separation took place between walls A, B and C during the test, the model of walls A+B+C would appear more appropriate. However, it is necessary to point out that this model yields a considerable overestimation of the stiffness of walls A and C, because of the assumption of rigid horizontal edges of their panel elements. A more realistic model, which releases this assumption should be devised in order to better reproduce the experimental results. Nothwithstanding the drawbacks described, at the displacement value corresponding to the maximum experimental base shear, this was predicted with errors ranging from -5% to 17% for the different models. This results is broadly satisfactory, considered the complexity of material and structural behaviour.

In conclusion, the degree of accuracy of the panel element, together with its high computational efficiency, encourages further studies on this element and its utilization in practical design.

ACKNOWLEDGMENTS

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