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ULTIMATE STRENGTH EVALUATION OF UNREINFORCED MASONRY BUILDINGS VIA NUMERICAL SIMULATIONS

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ABSTRACT

The paper deals with the F.E. modeling of a full-scale two-storey unreinforced-brick-masonry (URM) prototype, which was built and tested at the Department of Structural Mechanics of the University of Pavia. Experimental tests and numerical simulations were part of a coordinate research project on the behavior of URM existing buildings when subjected to horizontal seismic actions. An important goal of the project consisted in the assessment of numerical approaches in order to correctly represent the actual behavior of existing masonry structures.

To verify the capabilities of numerical approaches, destructive tests performed on the prototype were simulated numerically, in order of permitting direct comparisons between experimental and numerical results. After performing destructive tests, the structural rehabilitation of the prototype will be performed by means of injections; a strengthening intervention will be also designed, in order to evaluate related variations in the ultimate strength of the prototype and in its overall behavior. Numerical models have been setup simulating several hypothesis for the strengthening of the construction. Influences of various interventions on the strength and the overall behavior of the prototype are evaluated numerically in order to check their effectiveness.

KEYWORDS

Unreinforced masonry ; strengthening of buildings ; finite elements ; nonlinear behavior.

INTRODUCTION

The evaluation of the structural behavior of masonry buildings represents an important task in the assessment of the safety of new constructions or for the conservation of the existing building heritage. For these reason, in the last decades an increasing number of researchers devoted the major attention to the modeling of the masonry material behavior, collecting a large amount of experiences both as numerical and experimental results (see f.i. Chiostrini and Vignoli (1994), Chiostrini (1993) and Chiostrini and Vignoli (1992)).

The present work deals with the F.E. modeling of a building prototype in real scale, subjected to a series of destructive tests (see Calvi and Magenes (1994 a) and (1994 b)). The building has a rectangular base of 600 × 440 cm and overall height of 643.5 cm. A "perfect defect" was introduced, through a separation from the "window" and the "door" walls, as shown in Fig. 1. Two flexible diaphragms were built at heights of 283 and 577 cm from the base, composed by I beams, disposed along the short side of the base, constituting in this way a join between the two parts of the prototype. More details about the construction and the instrumentation of the prototype are reported in Calvi and Magenes (1994 b).

According to the experimental tests setup, in the numerical simulation a displacement story was imposed along the x direction at the second floor level to both the two parts of the prototype (see the coordinate system in Fig. 1); the same force applied to the second floor (deriving from the imposed displacement) was at the same time imposed also at the first floor, as to simulate seismic actions, in which, due to the same masses of the two floors, the same inertia forces are applied at both the two floor levels.

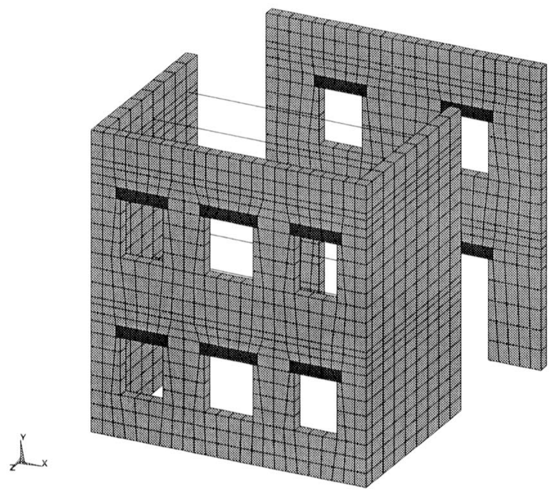


Fig. 1. F.E. mesh and representation of the two part of the prototype : door and window walls

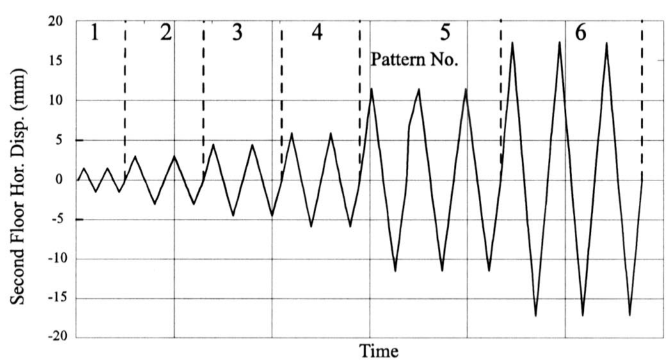


Fig. 2. Displacement patterns considered in the numerical simulations

Table 1. Displacement patterns characteristics

Pattern	Drift (%)	Second Floor Disp. (mm)	No. of cycles
1	0.025	1.442	2
2	0.050	2.885	2
3	0.075	4.328	2
4	0.100	5.770	2
5	0.200	11.54	3
6	0.300	17.31	3

Fig. 2 and Table 1 show the displacement story considered in the numerical simulation, which has been divided into six patterns. Each pattern is composed by two or three displacement cycles of the same amplitude. The experimental test program comprehended also some low-level cycles between two consecutive displacements patterns, in order to assess the conditions of the prototype before performing the following run. Such low-level cycles are not considered in the numerical simulations, as in this case they do not assume any relevance.

The coordinated research project which comprehend both experimental and numerical tests was supported by the Gruppo Nazionale per la Difesa dai Terremoti del Consiglio Nazionale delle Ricerche (CNR-GNDT (1992 a)) in order to produce a contribution to the better understanding of the behavior of URM existing buildings, with a particular regard to the strength with respect to horizontal seismic action. Another aim of the project was the evaluation of the reliability of various numerical approaches, developed by several research unit from different Universities, in representing the actual ultimate behavior of masonry elements.

The experimental part of the project was divided into two phases : in the first one, a complete mechanical and physical characterization of materials (bricks and mortar) was performed, also with tests on small assemblages (triplets and piers) ; in the second phase, tests on elements (walls) and on the whole prototype were executed. Numerical analysis comprehended initial forecast simulations and then a refinement phase, developed when the experimental results of the prototype were known, permitting in this way the better setup of the various models. More details concerning characteristics and dimensions of the prototype, experimental tests and other results are reported in CNR-GNDT (1992 b) and (1993).

MODELING OF MATERIAL BEHAVIOR

Masonry F.E. modeling was performed through an eight-node tetrahedron (the "stiff65" library element of ANSYS (1993)) provided of three translational degrees of freedom for each node and eight integration points. The following characteristics were implemented: *i)* cracking was permitted in three orthogonal directions at each integration point; *ii)* when cracking occurred at an integration point, cracking was modeled through an adjustment of material properties, hence cracking was assumed as a "smeared band" of cracks; *iii)* the masonry material was initially (before cracking) assumed to be isotropic; *iv)* in addition to cracking (in tension) and crushing (in compression) a plastic behavior was defined, accordingly to the classic Drucker-Prager failure surface.

The brittle behavior of masonry was modeled through an appropriate failure criterion, here defined by means of only two material parameters: f_t (uniaxial tensile strength) and f_c (uniaxial compressive strength); cross section of the assumed failure surface was defined with a cyclic symmetry about each 120^0 sector of the deviatoric plane. Both cracking and crushing failure modes were accounted for. The presence of a crack at an integration point was represented through modification of the stress-strain relations by introducing a plane of weakness in a direction normal to the crack face. Also, a shear transfer coefficient β was introduced (depending on the crack status: open -β_t- or re-closed -β_c-), representing a shear strength reduction factor for those subsequent loads inducing sliding (shear) across the crack face.

Rate independent plasticity : Frictional and dilating behavior of masonry

In order to reduce the number of the parameters employed to represents the nonlinear behavior of masonry, a Drucker-Prager perfectly-plastic criterion was employed in the model, avoiding the need for definition of an hardening rule. In this way cohesion c and angle of internal friction ϕ were assumed as the only two material parameters required to define the yield surface. A non-associated flow rule was adopted for determining the direction of plastic straining, which was calculated assuming the angle of dilitancy ν instead of the angle of internal friction. The dilitancy (which represents a third material parameter) controls both the volumetric expansion during the plastic straining and its deviation from the associated flow rule; the correct setting of the dilitancy value, moreover, permits to define the relevance of the frictional behavior of the material, as $\nu = 0°$ signifies a pure frictional behavior with no volume change during the plastic flow, while $\nu = \phi$ indicates a purely dilating or perfectly plastic material with zero friction and the validity of the flow rule. Several ways are possible in order of representing relationships between ϕ and ν (see f.i. Bishop (1950), Rowe (1962) and Davis (1968)), focusing the major attention on the physical meaning of the parameters, as represented in Fig. 3, where the trace of the yield function is depicted in a meridian plane in the $\tau - \sigma$ stress space.

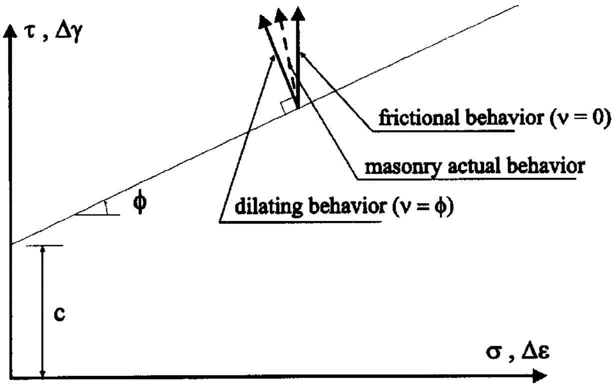


Fig. 3. Frictional and dilating behavior : flow rule for masonry material.

Failure surface

The failure criterion here adopted (William and Warnke (1975)), initially defined for concrete, accounts for both cracking and crushing failure modes trough a smeared model. Despite the need for five constants in order to define the criterion, in most practical cases (thereby when the hydrostatic stress is limited by $\sqrt{3}f_c$) the adopted failure surface can be specified by means of the only two constants : f_t and f_c (respectively the uniaxial tensile and compressive strength). The failure surface here adopted is depicted in Fig. 4.

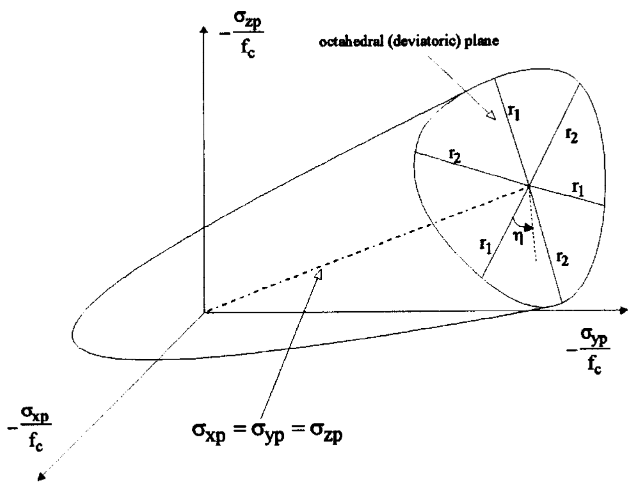


Fig. 4. Failure surface

NUMERICAL MODEL RESULTS

A series of numerical simulations was executed in order of reproducing most of the experimental tests actually performed for the prototype. In particular, the experimental project comprehended seven displacements patterns each of them composed by several cycles with constant maximum displacements. Numerical simulations regards six of the seven experimental patterns (see Fig. 2), as the seventh one resulted at the same time the most cumbersome to be simulated (due to the large number of elements in the cracked state, with the connected convergence difficulties) and the less interesting from a structural point of view (as all the cracking mechanism resulted already achieved in the preceding patterns). Results of the numerical simulations were accurately compared with the experimental ones. In particular, for each simulation comparisons were performed on the following aspects: *i)* horizontal displacement along the vertical sides of each wall of the prototype; *ii)* vertical stress distribution at the base of the walls; *iii)* shear in individual piers of the walls; *iv)* vertical displacements at the top side of the walls; *v)* plots of base shear versus top displacement of the walls; *vi)* crack patterns in the walls. All the different aspects of the results were found in a satisfactory agreement with experimental data, confirming in such way the effectiveness of the model.

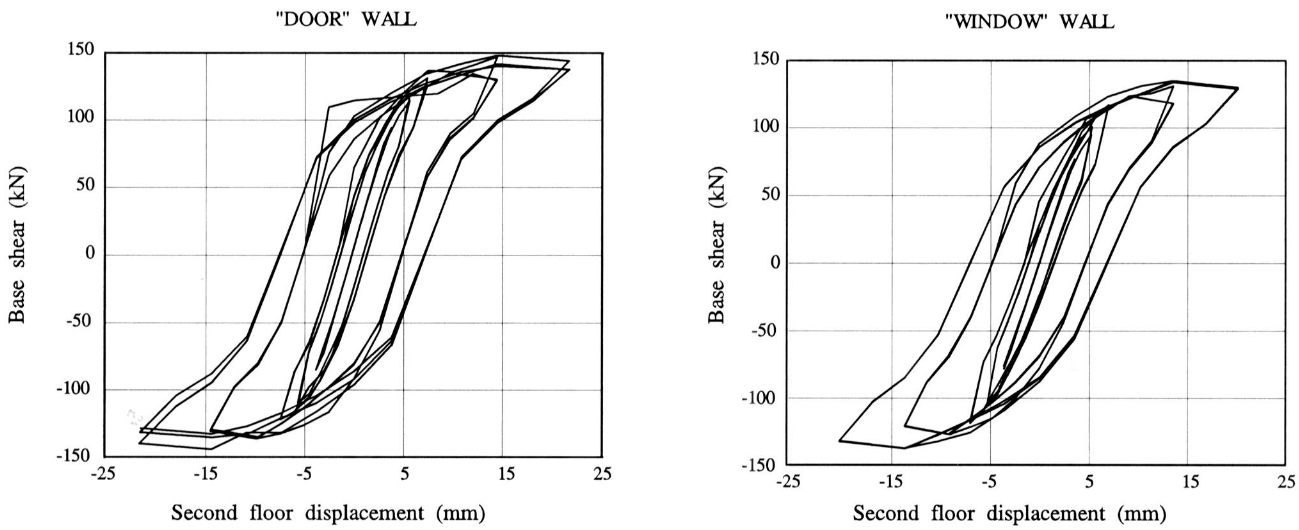


Fig. 5. Load-displacement plots for the six patterns considered in the numerical simulations

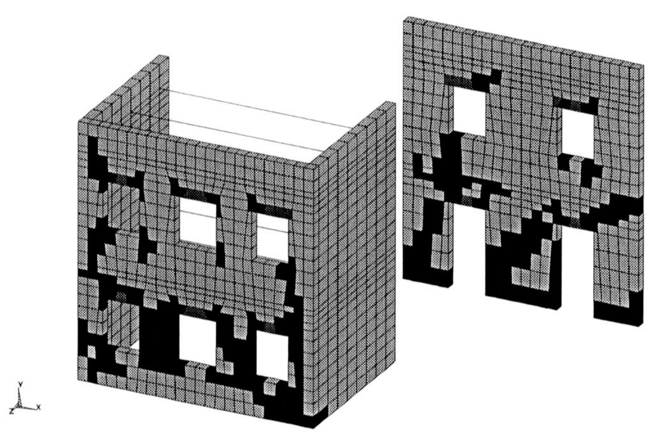


Fig. 6. Cracked elements after the first cycle in pattern no. 6

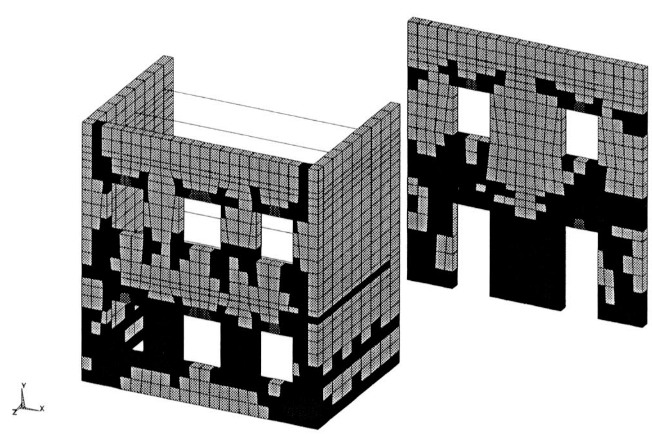


Fig. 7. Cracked elements after the last cycle of pattern no. 6 (last cycle in the numerical analysis)

Fig. 5 shows the base shear against the imposed displacement on the second floor level ; Figs. 6 and 7 show numerical results concerning cracking of the prototype, permitting a comparison with experimental ones. For comparisons it is important to consider that, due to the smeared crack model employed, the whole finite element is considered cracked when the boundary of the strength dominion is reached in an integration point (distinguished with dark hatch in the figures).

EFFECTS OF VARIOUS STRENGTHENING HYPOTHESIS

The effectiveness of several strengthening interventions is evaluated via numerical simulations ; due to the overall good accordance between numerical and experimental results, in fact, the numerical model can be assumed as representative of the actual behavior of the construction and can thereby be utilized also in the design of the strengthening of the building. Simulations are performed considering only the "door" wall and applying two forces of the same intensity at the two floor levels but controlling the upper floor horizontal displacement.

Each of the strengthening hypothesis is then evaluated trough the corresponding strength and ductility of the wall, compared with the same values of the unreinforced wall (for which an ultimate base shear T_u = 162 kN and an ultimate horizontal displacement s_u = 21.5 mm are obtained). In the understanding of the results is important to remember that in the numerical simulations the ultimate displacement results the one corresponding to the last step of the numerical analysis in which the convergence is achieved. Obviously, increase of tolerances and of the maximum number of equilibrium iterations allowed can result in a prosecution of the analysis and in a corresponding larger ultimate displacement. Due to this, results concerning the ductility of the model can be assumed with some attention.

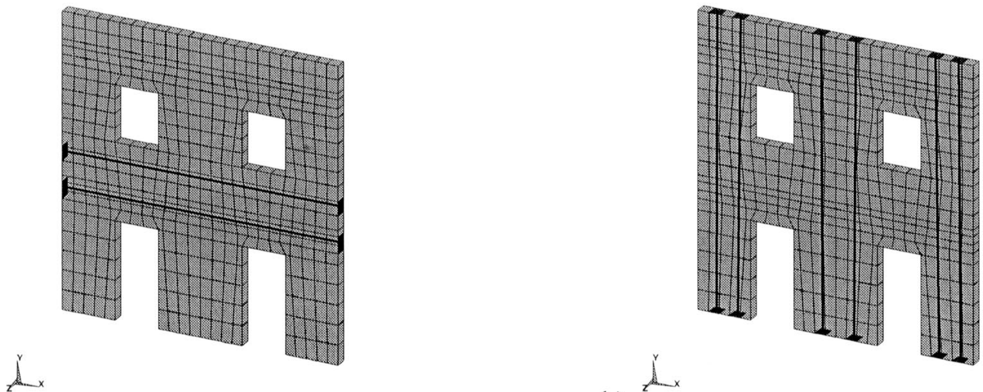
Application of r.c. lintels

The first strengthening hypothesis concerns the adoption of reinforced concrete lintels. The first cracking of the prototype, in fact, was observed just in the zones above the openings ; the possibility that the masonry lintels (with vertical mortar bed joints) could facilitate the start of cracking is thereby checked trough the introduction of resistant elements above the openings. For this strengthening hypothesis the numerical analysis showed a slight increase of the strength (T_u = 173 kN), without appreciable variations of the ultimate displacement (s_u = 21.6 mm).

Application of horizontal tie bars in the floor band

The second intervention considered was the application of steel bars in the horizontal masonry band corresponding the first floor (see Fig. 8a); also such choice was made in order to prevent the first cracking of the wall just above the door openings, this time trough the application of a confining horizontal restraining, acting in those zones where the pressure of the vertical dead loads is not present (thereby between the door and window openings in the horizontal floor band). The steel bars are applied on two levels, so to provide a better distribution of the confining action and to form a bending-reinforced horizontal beam corresponding to the masonry band.

Trough the first numerical simulation it was demonstrated that the application of horizontal bars did not provide an effective structural reinforcement, as the confining action of the bars was recognized acting only after the occurrence of large cracks, thereby when the displacements grew enough to produce appreciable reactions in the steel bars. The application of prestressing in the horizontal steel bars (so as to produce a compression of 20 N/cm² in the horizontal masonry band) caused some modifications in the cracking mechanisms, inducing a behavior similar to the "shear type" model, but without remarkable improvements of the strength and ductility of the wall ; on the contrary, the ultimate displacement resulted as $s_u = 20.8$ mm, thereby inferior to the one of the unreinforced model. Simulations concerning the above strengthening intervention can be summarized with the conclusion that the only application of horizontal steel bars does not produce, for the examined structure, any appreciable benefit to the ultimate structural behavior.



a) b)

Fig. 8. Truss and plate elements for the modeling of horizontal and vertical tie bars

<u>Application of vertical tie bars</u>

The application of vertical reinforcements (see Fig. 8b) demonstrated as the better strengthening hypothesis between the ones here considered, resulting the one producing the larger improvements of strength and ductility.

The vertical reinforcements, in fact, demonstrated capable of stiffening the resistant mechanisms of the masonry walls (diagonal cracking) without appreciable variations of the overall structural behavior. Moreover, such upgrading was found effective also without prestressing (see Table 2). As in the case of horizontal bars, the prestress in the vertical ties corresponded to a compression of 20 N/cm² in the vertical masonry bands.

Table 2. Effects of vertical reinforcements

Ultimate base shear T_u (kN)	Ultimate displacement s_u (mm)	Type of vertical reinforcements
$T_u = 162$ kN	$s_u = 21.5$ mm	none
$T_u = 213$ kN	$s_u = 24.2$ mm	not prestressed
$T_u = 246$ kN	$s_u = 28.8$ mm	prestressed

CONCLUSIONS

Results from several numerical simulations are reported concerning the ultimate behavior of unreinforced masonry constructions. Comparisons with experimental results demonstrate the overall reliability of the approach in simulating actual structural behavior until collapse. Due to the macro-modeling adopted, in which a single finite element comprehends several bricks and mortar joints, experimental results from test on panels are required in order to achieve a satisfactory reference for the material parameters to be considered in the numerical model. Despite the need mentioned, it is felt that the simulations performed can represent an encouraging result about the possibility of representing the most important characteristics of URM building structural behavior.

As introduced, the numerical model requires seven parameters to identify the nonlinear behavior of masonry: three for the definition of the plastic behavior (cohesion, angle of internal friction and dilitancy) and four for the strength dominion and the behavior in cracked condition (uniaxial tensile and compressive strength, open and re-closed crack shear transmission coefficients). The numerical model setup was performed taking into account the main aspect of minimizing the number of parameters required to define the material behavior, so to ensure an easier determination of such parameters. Performances of the model and the good agreement between experimental and numerical results confirmed the effectiveness of such choice.

Discussion concerning the effects of various of the most common strengthening interventions on masonry buildings are reported, furnishing elements for the assessment of design criteria in the problem of upgrading existing masonry buildings.

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