POLYMERIC GRID FOR A COST EFFECTIVE ENHANCEMENT OF THE SEISMIC PERFORMANCE OF MASONRY BUILDINGS

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ABSTRACT:

The performance of masonry walls reinforced using polymeric grid embedded into plaster layers as a tools for the seismic enhancement of brick masonry buildings has been investigated by experimental tests. The results of the experimental campaigns are summarised and discussed in the paper. Based on the experimental data and on the results of numerical simulations, simplified models to be used as tools for the design of the retrofitting intervention are proposed. The models properly consider the so called "first" and "second" collapse mechanisms as well as the grid effect in the evolution of the above mentioned mechanisms.

KEYWORDS: Masonry buildings, seismic enhancement, polymeric grid, cost-effective

1 INTRODUCTION

Retrofitting techniques based on the reinforcement of plaster layers in masonry buildings cannot be considered as novel: as a matter of a fact, starting from the '70s, masonry buildings have been strengthened using plasters reinforced with steel grids. This method has been widely used, for instance, in Italy for the rehabilitation of constructions after the 1976 Friuli earthquake and it is still applied, even if often criticised for some intrinsic contraindications and limitations mainly related to its actual efficiency and durability. More recently, other plaster reinforcements have been proposed in literature and are available on the market, like those using light carbon grids included into thin cement plaster layers. Polymeric reinforcements could also be employed as a retrofitting tool in the seismic upgrading of masonry buildings and could represent an alternative retrofitting system able of overcoming some deficiencies of the other reinforcement systems (i.e. corrosion of the steel) while being more effective in terms of costs-benefits ratio.

2 SEISMIC BEHAVIOUR OF MASONRY BUILDINGS

Notwithstanding masonry buildings usually represent the simplest constructions configuration and require a very poor constructive technology, their seismic behaviour show elements of complexity greater than those typical of new structural configurations associated to the modern materials, like steel and reinforced concrete frames. In masonry constructions, the ordinary structural configuration consists of a three-dimensional assembly of mass-distributed plane elements that are characterised by a twofold behaviour when horizontal inertia forces are induced by earthquake attacks. The forces in the plane of the panels, combined with those exerted by vertical loads, induce a membrane behaviour with in-plane normal and shear stresses, while the actions orthogonal to the panel induce a plate behaviour with out-of-plane bending and shear. The stress status in the members is complex and strongly influenced by the in-plane deformation of the element and by its interaction with the neighbouring elements. Masonry is indeed a composite material whose behaviour depends on the macroscopic mechanical characteristics: tensile and compressive strength, elastic and shear modulus. The tensile strength is practically null; this can easily induce cracks on the surfaces subjected to tension, causing a subdivision of the panels into separate portions that can transform sections of the building in kinematisms, so that each rigid portion can move with respect to the other till the structure's collapse. Such mechanisms can develop in the plane of the panels as

well as orthogonally, giving rise, respectively, to the so called "first" and "second mode" collapse mechanisms (Giuffrè, 1990). Furthermore, the texture of the masonry, associated with the dimensions and the organization of the blocks, can induce other behaviours characterised by the loss of integrity of the structural elements that can collapse for the breaking of the internal links among the blocks.
The seismic behaviour of a masonry building depends on three fundamental performance issues to be guaranteed:

- the preservation of the global integrity, with no separation into macro-elements, thus allowing for a box-like behaviour with a redistribution of the horizontal forces among all the resisting elements;
- the capability of all the members to resist the forces induced by the seismic actions, i.e. the capability of sustaining the induced forces without reaching the ultimate displacement;
- the capability of the panels not to develop collapse mechanisms associated to the evolution of the kinematisms and the loss of integrity.

3 SYSTEM DESCRIPTION

The retrofitting system presented in this paper consists in the insertion of a polymer grid embedded into a thin lime based mortar plaster. The grid is the "RichterGard RG TX" one, a stiff monolithic polymer grid with integral junctions, characterised by an isometric geometry resulting in apertures of equilateral triangles. Figure 1 shows the grid and its installation.

(a) grid (b) installation of the grid (c) plastering

Figure 1 Characteristics of grid and its installation

The basic underlying idea in the use of polymeric grid is that it could improve the performance of the masonry by increasing its strength and its ductility

4 EXPERIMENTAL ACTIVITIES

A pre-requisite for the use of the proposed retrofitting technique is the assessment of the effectiveness of plaster reinforcement as strengthening tool. A series of tests, mainly aimed at evaluating the actual influence of the grid in the mechanical behaviour of the reinforced element, with particular attention to its response to the horizontal actions, were carried out. Four testing campaigns, each one including one or more series of tests, were planned and executed; they consisted in:

- diagonal compression tests on 18 brick masonry squared panels;
- shear compression tests on 12 brick masonry rectangular panels;
- out-of-plane (stability) tests on 12 brick masonry large panels;
- out-of-plane (stability) tests on 4, previously damaged, repaired panels.

A detailed description of the tests is given in (Mezzi et al., 2006) and (Dusi et al., 2007a); for sake of conciseness, in the following Tables 1, 2, 3 and 4, a summary of the tested panels characteristics is reported along with some comments on the main outcomes.

Table 1 Characteristics of panels subjected to diagonal compression tests

Group	Number of panels	Panel's ID	Plaster	Grid
1	3	#1, #2, #3	No	No
2	3	#4, #5, #6	Yes	No
3	3	#7, #8, #9	Yes	One side
4	3	#10, #11, #12	Yes	Both sides
5	3	#13, #14, #15	Yes	Both sides
6	3	#16, #17, #18	Yes	Both sides

Table 2 Characteristics of panels subjected to shear-compression tests

Group	Number of panels	Panel ID	Plaster	Grid	Compression stress [MPa]
1	2	#1, #2	No	No	0.50
	2	#3, #4	No	No	0.75
2	2	#5, #6	Yes	No	0.50
	2	#7, #8	Yes	No	0.75
3	2	#9, #10	Yes	Both sides	0.50
	2	#11, #12	Yes	Both sides	0.75

Table 3 Characteristics of panels subjected to out-of-plane tests

Panel ID	Plaster	Grid	Panel ID	Plaster	Grid
F1	Both sides	No	F7	No	Tension side
F2	Both sides	No	F8	No	Tension side
F3	No	No	F9	No	Tension side
F4	No	No	F10	No	Compression side
F5	Both sides	Tension side	F11	Both sides	Tension side overlap
F6	Both sides	Tension side	F12	Both sides	Tension side overlap

Table 4 Characteristics of repaired panels subjected to out-of-plane tests

Panel ID	Plaster	Grid
R1	Both sides	Both sides
R2	Both sides	Tension side
R3	Both sides	Both sides
R4	Both sides	Tension side

4.1 Effects of the grid reinforcement

Diagonal compression tests, carried out according to the ASTM standard (American Society for Testing Materials, 1981), showed that the ultimate deformation of reinforced panels was increased by a factor of 2 to 3, thus indicating that the grid's presence adds a positive contribution the global ductility of the masonry wall. Conversely, no significant strength increment were noted.
The results from shear-compression tests basically confirmed those from the diagonal-compression tests. The comparative observation of the unreinforced and reinforced panels' conditions at failure, though presenting similar modalities, showed very "clean" cracks diagonally orientated in the unreinforced panel, whilst the reinforced panels were characterised by widespread nets of cracks; this suggests that the reinforced panels' collapse requires the formation of a large number of failure surfaces and, therefore, an higher value of the ultimate strength. The shear-compression tests also confirmed the positive effect of the grid on the ductility of the panels as well as a significant energy dissipation capacity given by the grid. A non-negligible increment of the maximum resistant shear force was found.
The stability tests clearly demonstrated the positive effects of the grid reinforcement on all the significant mechanical parameters of the panels, i.e. ultimate load, ultimate displacement and energy dissipation. The distribution of the crack patterns, already observed in shear compression tests, put again into evidence the beneficial contribution of the grid, related to the mitigation of the damage peak and to the increase in energy

dissipation due to the spreading of the damaged areas. The analysis of the experimental data also confirmed the expected beneficial effects of the grid on the out-of-plane behaviour of the panel, proven by a significant enhancement of the resistance against the collapse mechanisms.

5 EVALUATION OF THE GLOBAL SEISMIC ENHANCEMENT

The effects of the plaster reinforcement consist in an increase of both strength and ductility.

5.1 Strength increase

Different results on the strength increment have been obtained from the different tests carried out.
Diagonal compression tests didn't showed significant shear strength increase of the panels; experimental results reported in literature confirm this behaviour (Drdácký and Lesák, 2001). On the contrary, from shear-compression tests some non negligible increment of the maximum resistant shear force were found, but with peak effects. The discrepancy can be explained by considering that, besides the different testing modalities, diagonal-compression tests were carried out on masonry panels built with high strength bricks, while in the shear compression tests, use of medium strength bricks was made. Taking into account that existing buildings are often made of poor quality masonry, it can be stated that the polymeric grid reinforced plaster gives a shear strength increase. Such a strength increment can therefore be assumed in the design, with values ranging from 1.0 (no increase) up to 1.5 (maximum experimental finding); a 1.2 value is prudentially suggested by the authors.

5.2 Ductility increase

The increment in the ductility capacity, given by the presence of polymeric grid, is a constant result of all the tests carried out, therefore the fact that the reinforcement increases significantly the ductility is proven and must be considered in the design of the retrofit interventions. This effect can be directly taken into account when nonlinear static or dynamic analyses are carried out, for which the force-deformation relationship of each structural element is explicitly defined. According to the experimental results, the force-deformation diagrams of the reinforced panels can be characterised by an ultimate deformation of 0.4% (shear distortion). On the contrary, the maximum distortion of the tested unreinforced panels turned out to be approximately 0.2%. It can be observed that the ultimate displacements obtained from the tests carried out, are lower than those found in literature for either new and ancient masonry (Anthoine et al., 1995), (Chiostrini et al., 2000). The more pronounced plastic behaviour shown in the literature cases could be attributed to the mortar characteristics and to the masonry organization: this aspect will be clarified by further tests, already planned, on panels made with low quality masonry.

5.3 Modality for seismic check

The increase in ductility of the reinforced masonry, as well as the strength increase, can be taken into account within an analysis method for the seismic assessment of the building, through the amplification of the structure factor q. Considering, for an existing building, a strength increase k_s (i.e. 1.2) and a ductility increase k_μ (i.e. 2.0), an amplification $k_q = k_s \times k_q$ could be assumed by the designer for the structure factor. If, more realistically, an energy-based correlation between ductility and structure factor is assumed, the amplification factor is given by

$$k_q = k_s + \sqrt{\frac{k_\mu\left(q_0^2+1\right)-1}{q_0^2}} \qquad (5.1)$$

where q_0 is the basic structure factor.

6 ASSESSMENT BASED ON LOCAL COLLAPSE MECHANISMS

The experimental activities have been supported by theoretical and numerical investigations aimed at interpreting, reproducing the results and defining suitable simplified models to be used as tools for the design of the retrofitting interventions (Dusi et al., 2007b).

As previously said, in the existing masonry buildings partial collapses for seismic actions often happen, generally for loss of equilibrium of masonry portions. The verification with reference to the in-plane and out-of-plane collapse mechanisms, is meaningful only if a monolithic behaviour of the masonry wall is guaranteed. Under this assumption, the verification can be carried out by resorting to the limit analysis of the equilibrium, according to the kinematic approach, based on the identification of a collapse mechanism and on the evaluation of the horizontal force that activate such a mechanism. The application of the proposed verification method presupposes, therefore, the analysis of the local mechanisms, deemed significant for the construction. These latter can be assumed from the knowledge of the seismic behaviour of similar structures, already damaged from the earthquake, or can be defined by considering the presence of cracks patterns, even if not directly related to seismic actions. The quality of the connections among the walls, the masonry organisation and texture, the presence of tie-bars, the interactions with other elements of the construction or the neighbouring buildings must also be considered.

The kinematic approach also allows for the determination of the horizontal force evolution that the structure is progressively able to withstand meanwhile the mechanism evolves. Having defined α as the ratio of the horizontal forces applied to the correspondents weights of the structural masses, such an evolution can be represented by a curve of α multiplier as a function of the displacement d_k of a reference point in the system. The curve, determined up to the annulment of any capability of sustaining the horizontal actions ($\alpha = 0$), can be transformed into a capacity curve of an equivalent single-degree-of-freedom system, for which the ultimate displacement capacity of the local mechanism can be defined and compared to the displacement demand requested by the seismic action.

For the application of the analysis method, the following assumptions are generally made:

– the masonry tensile strength is null;
– absence of sliding among the blocks;
– the masonry compressive strength is infinite.

However, for a more realistic simulation, the following parameters shall be considered, even if in a simplified way:

– the sliding between the blocks, by taking into account the friction;
– the connections (even if of limited resistance) among the walls;
– the presence of metallic tie-rods;
– the presence of the polymeric grid within the plaster and on the corners;
– the limited compressive strength of the masonry, by considering sets of hinges adequately located;
– the presence of multi-leaves walls.

6.1 Linear kinematic analysis

In order to get an evaluation of the horizontal loads multiplier α_0 that leads to the activation of the local mechanism, it is necessary to apply to the rigid blocks composing the kinematic chain the following forces:

– the self weights of the blocks (applied to their centres of mass);
– the dead and live vertical loads acting on the blocks;
– a system of horizontal forces proportional to vertical loads (if such forces are not transmitted to others walls);
– other external forces (such as those transmitted by metallic tie-rods);
– other internal forces (such as the actions related to the connection between blocks and the grid presence).

Assigning a generalized virtual displacement to the generic block k of the kinematic chain, one can determine, as a function of the rotation and geometry of the structure, the displacements components of application points of the various forces applied in their respective directions. The multiplier α_0 is then obtained by applying the

Principle of the Virtual Work and equating the work done by the internal and external forces applied to the system acting through the virtual displacements:

$$\alpha_0\left(\sum_{i=1}^{n} P_i\delta_{x,i} + \sum_{j=n+1}^{m} P_j\delta_{x,j}\right) - \sum_{i=1}^{n} P_i\delta_{y,i} - \sum_{h=1}^{o} F_h\delta_h = L_{fi} \tag{6.1}$$

where:

n is the number of all the forces applied to the various blocks of the kinematic chain;
m is the number of forces not directly acting on the blocks, generating horizontal forces on the elements of the kinematic chain;
o is the number of external forces, not associated to masses, applied to the various blocks;
P_i is the vertical force of the generic block;
P_j is the generic vertical force, not directly applied to the blocks;
$\delta_{x,i}$,$\delta_{x,j}$ are the horizontal virtual displacements of the points of application of the forces P_i and P_j;
$\delta_{y,i}$ is the vertical virtual displacement of the point of application of load P_i;
F_h, δ_h are, respectively, the external force applied to a block and the displacement of the application point;
L_{fi} represents the work of the internal forces.

6.2 Nonlinear kinematic analysis

In order to evaluate the displacement capacity of the structure, up to its collapse, in the considered mechanism, the horizontal load multiplier α_0 can be determined not only with reference to the initial configuration, but also to modified configurations of the kinematic chain, representatives of the evolution of the mechanism and defined by the displacement d_k of a system's reference point. The analysis must be carried out up to the configuration corresponding to the annulment of the α multiplier, corresponding to a displacement $d_{k,0}$. For any configuration of the kinematism, the value of α_0 can be determined from Eqn. 6.1, properly rewritten by referring to the modified geometry. The analysis can be carried out with the use of graphic methods, by defining the system geometry in the different configurations up to its collapse, or with analytical-numerical methods, by considering a set of virtual displacements and rotations to be progressively updated based on the system geometry evolution.
If the forces involved are kept constant during the kinematism evolution, the resulting curve is almost linear. When the progressive variation of the external forces with the kinematism evolution is taken into account (e.g. when considering the elongation of the grid within the plaster), the curve could be linearized by segments, evaluating the curve in correspondence to displacements for which significant variations happen (i.e. grid yielding, ultimate deformation of the grid, etc.).

7 OUT OF PLANE COLLAPSE MECHANISM

In the following is reported, for its significance, an example of application of the proposed analysis approach to out-of-plane collapse mechanism, validated on the basis of the experimental stability test results. The analysis is based on the following assumptions: the collapse mechanism consists of an horizontal crack located at an unknown level. The crack divides the masonry panel into two "rigid" portions, as shown in Figure 2.
Given the virtual rotation θ_1 of block 1, the rotation θ_2 of block 2 can be easily computed. The equilibrium condition can therefore be derived from the Principle of Virtual Works by equating the total work done by the external and internal forces that are applied to the considered system along the correspondent virtual act of motion. The forces doing the work along the system's virtual displacement are:

– the weight of the lower and upper portions, P_1 and P_2

$$P_1 = \frac{a_1}{h}P = \frac{x-1}{x}P \qquad\qquad P_2 = \frac{a_2}{h}P = \frac{1}{x}P \tag{7.1}$$

- the lateral seismic forces related to the self weight, F_1 and F_2, given as a function of the α_0, multiplier of the actions that activate the mechanism

$$F_1 = \alpha_0 P_1 \qquad\qquad F_2 = \alpha_0 P_2 \tag{7.2}$$

- the vertical reaction at the top of the wall, V (including the loads originating from the overhanging walls and floors);
- the resistant moment of the cracked reinforced section M_{res}, evaluated considering the grid's presence. M_{res} can be computed considering the ultimate limit state of the section with the grid in correspondence of the stretched edge. The calculation also allows for the definition of the neutral axis position, which identifies the axis around which the rotations θ_2 develop.

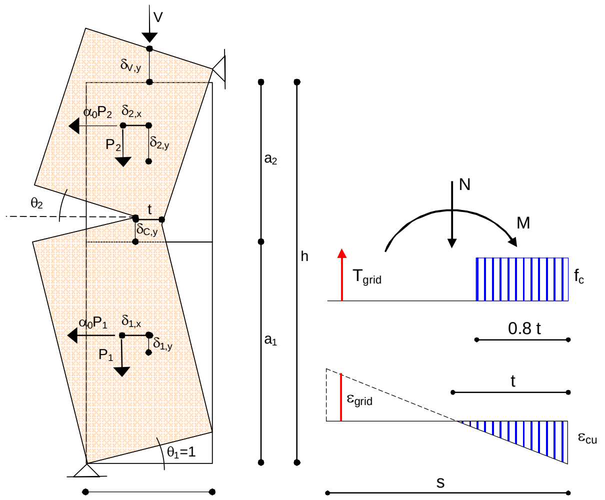

Figure 2 Out-of-plane collapse of a panel

The displacements of the forces' application points, i.e. the rotations of the moment application axes, can be easily derived from geometric considerations on the kinematism (Figure 2).
From the application of the Principle of Virtual Works, it can be obtained:

$$F_1 \cdot \delta_{1x} + F_2 \cdot \delta_{2x} - P_1 \cdot \delta_{1y} - P_2 \cdot \delta_{2y} - V \cdot \delta_{vy} - M_{res} \cdot \beta_z = 0 \tag{7.3}$$

from which, by simple substitutions, the value of the α_0 seismic multiplier of the actions activating the mechanism can be obtained:

$$\alpha_0 = \frac{s}{h(x-1)}\left[2x + \frac{Vx}{P} + \frac{Vx^2}{P} - \frac{2t \cdot x}{s} - \frac{2Vtx^2}{sP} + \frac{2x^2 M_{res}}{Ps}\right] \tag{7.4}$$

From a general point of view, the value of α_0 shall be compared with the seismic design coefficient, therefore the verification is satisfied when:

$$\alpha_0 \geq \lambda\, S_d \tag{7.5}$$

being S_d the value, in g, of the design spectrum, i.e. the value of the elastic spectrum divided by the proper q factor, and λ is the reduction factor of the seismic forces.
From the tests carried out applying loads orthogonal to the panel (Dusi et al., 2007a), a value of q = 3 can be assumed. In this case, α_0 is defined as a function of x, the elevation of the collapse section, that can be estimated by minimising the value of the α_0 multiplier, by imposing

$$\frac{d}{dx}\alpha_0 = 0 \tag{7.6}$$

from which

$$\alpha_0 = \frac{s}{h(x_0 - 1)}\left[2x_0 + \frac{V \cdot x_0}{P} + \frac{V \cdot x_0^2}{P} - \frac{2 \cdot t \cdot x_0}{s} - \frac{2V \cdot t \cdot x_0^2}{s \cdot P} + \frac{2x_0^2 \cdot M_{res}}{s \cdot P}\right] \tag{7.7}$$

Because $x_0 \geq 1$, the contribution of the term

$$\frac{2 \cdot x_0^2 \cdot M_{res}}{P \cdot h \cdot (x_0 - 1)} \tag{7.8}$$

associated to the resisting moment of the cracked section, is always positive and therefore represents an increment of the load factor α_0.
Similar approaches can be extended to nonlinear analysis and to overturning as well as in-plane collapse mechanism.

8 CONCLUSIONS

Plaster reinforced with polymeric grid has been proven to be an effective, simple and a low-cost solution for the seismic performance enhancement of common masonry buildings as well as for historical buildings, thanks to complete chemical and physical compatibility and to the not invasive installation procedures. The grid shows its positive effect after the masonry failure, avoiding the collapse and crumbling of separated portions. The experimental data clearly demonstrated significant increase of ductility and a non negligible increase in strength. Based on the experimental evidence, suitable simplified models to be used as practical tools for the design of the retrofitting interventions using the plaster reinforced with RichterGard grid have been developed and validated. The proposed retrofitting technique has been successfully used for pilot applications.

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