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PUSH-PULL TESTS ON A REAL MASONRY-INFILLED RC BUILDING

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ABSTRACT :

Results from lateral-loading inelastic tests on a real masonry-infilled reinforced concrete building are presented and discussed. The first test has been carried out on the original building. Extensive damage to both masonry infill panels (MIPs) and reinforced concrete elements (columns and staircase structure) has been produced, with out-of-plane collapse of almost all the walls parallel to the loading direction. The second test has been carried out on the repaired building. Namely, perimeter MIPs were completely rebuilt and also strengthened by placing fiber reinforced polymer (FRP) rods into the mortar bed joints of masonry. The FRP system was applied with the basic aim to change the failure mode from diagonal tension to shear sliding. Experimental results show that this objective was successfully achieved. Extensive damage was produced also during this second test to all building components. Besides, back-analysis of experimental results has been carried out using simple analytical models including contribution from the staircase structure and the MIPs.

KEYWORDS: Reinforced concrete buildings, masonry panels, staircase structure, experimental tests, FRP strengthening, pushover response, analytical modelling

1. THE TESTED BUILDING

The investigated building was destined to demolition by the competent Authority within the dismantling process of the ILVA steel mill in Bagnoli (Naples, Italy). Two "push-pull" inelastic tests have been carried out: the first test (test #1) was conducted on the original building and brought the structure up to a very strong damage state. The second test (test #2) was carried out on the same building after repairing some damaged columns and re-constructing perimeter masonry walls, which were also strengthened with carbon fiber reinforced polymer (C-FRP) rods (Della Corte *et al.* 2008).

1.1. Building geometry

The building was built at the beginning of 1980s with a rectangular plan layout (18.50 m × 12.00 m) and two floors (Fig. 1 and Fig. 2) having heights on the ground equal to 4.60 m and 8.95 m (Fig. 3), respectively.
The floors' structure is made of mixed cast-in-place RC slabs and semi-hollow tile blocks. Floors are sustained by RC columns having a 300 mm × 300 mm cross-section. Columns are connected along the structure perimeter (and in some places on the internal sides) by rectangular beams having a 600 mm depth and a variable width (from 150 mm to 250 mm). Steel reinforcement of columns is constituted of 12 longitudinal ribbed bars with a diameter of 14 mm and transverse stirrups (ribbed bars) with a diameter of 8 mm spaced at about 200 mm along the column axis. Details of reinforcement of beams and floors' slabs are here omitted, because their influence on the obtained test results is deemed to be minimal as respect to columns and walls detailing. The perimeter masonry walls (Fig. 4) are faced-walls with the facing made of 100 mm thick semi-hollow tile blocks and backing made of 100 mm thick semi-hollow light-concrete blocks. The internal partition walls are made of 100 mm thick semi-hollow light-concrete blocks. All masonry walls were built-up by means of site-made mixed cement lime and sand mortar. The structural response is foreseen to be strongly affected by the presence of the staircase structure at the first level of the building. This staircase is made of two inclined RC slabs (150 mm x 920 mm) connecting the ground floor to the first floor, with an intermediate horizontal slab which is supported by a deep and short beam spanning 2 m between two columns at about the mid-point of the first-storey height (Fig. 3). Steel reinforcement of the slab connecting the ground floor to the half pace is constituted of 6 ribbed bars with 12 mm diameter plus 6 ribbed bars with 16 mm diameter, while the reinforcement of the inclined slab connecting the intermediate

half pace to the first floor is made of 6 ribbed bars with 10 mm diameter plus 6 ribbed bars with 16 mm diameter.

1.2. Building materials

The mechanical characterization of the building materials has been carried out on the basis of some experimental tests on materials, literature data and code recommendations.

Material properties of RC members have been assumed starting from both the results of some non-destructive tests (rebound index method) and knowledge of similar structures tested within a previous research project (Mazzolani 2006). The average cylindrical concrete strength has been assumed equal to 28 MPa and the concrete Young modulus equal to 30000 MPa. The average yield strength of steel has been assumed equal to 480 MPa.

Mechanical properties of masonry have been estimated on the basis of some compression tests carried out on masonry blocks. In particular, the following average compression strengths parallel to the holes were obtained: 17.1 MPa for semi-hollow tile blocks of the building in the original condition (test #1); 20.8 MPa for semi-hollow tile blocks used for the repairing (test #2); 3.1 MPa for semi-hollow light-concrete blocks (both test #1 and test #2). On the basis of these compression test results, assuming reasonable conservative values for the mortar strength (average compression strength = 2.5 MPa) and using literature or code recommendations for relating the block and mortar strength to the marony infill panels (MIPs) strength (Paulay and Priestley 1992, CEN 2005b, Italian Ministry of Public Works, 1987) the values of basic mechanical properties of MIPs have been assumed as follows:

- average compression strength parallel to the holes: in the range (4.3 – 9.5) MPa for brick masonry of the building in the original condition (test #1), (5.0 – 11.5) MPa for brick masonry used for the repairing (test #2), (1.3 – 2.0) MPa for light concrete masonry (both test #1 and test #2);
- average compression strength perpendicular to the holes: in the range (2.2 – 4.8) MPa for brick masonry of the building in the original condition (test #1), (4.8 – 5.8) MPa for brick masonry used for the repairing (test #2), (0.7 – 1.0) MPa for light concrete masonry (values are obtained as 50% of the compression strength parallel to the holes (FEMA 1999));
- average Young's modulus of elasticity: in the range (2365 – 5225) MPa for brick masonry of the building in the original condition (test #1), (2750 – 6325) MPa for brick masonry used for the repairing (test #2), (715 – 1100) MPa for light concrete masonry.

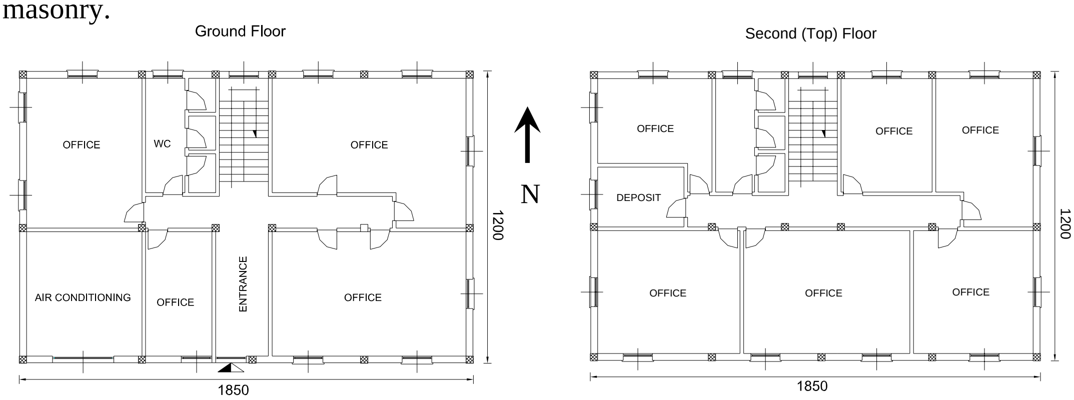

Figure 1. First- and second-floor functional layout of the building

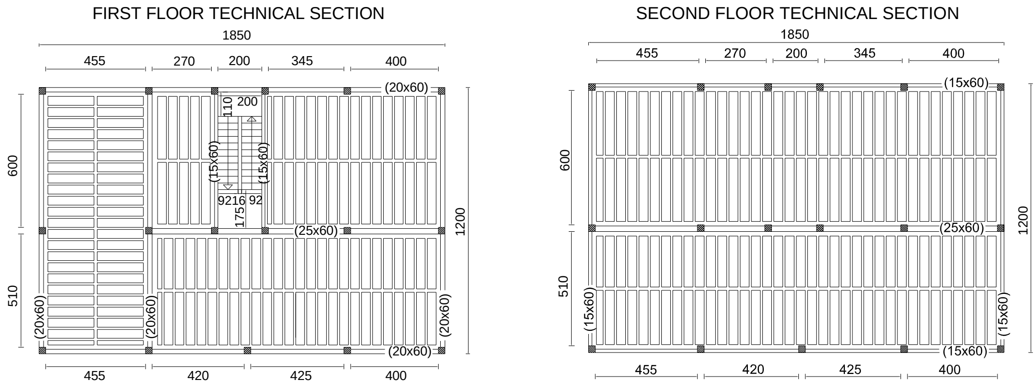

Figure 2. First- and second-floor structural layout of the building

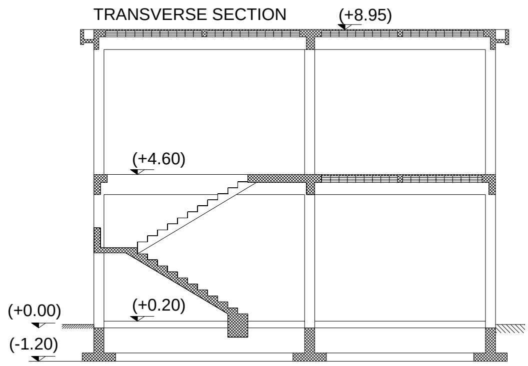

Figure 3. Transverse section of the building

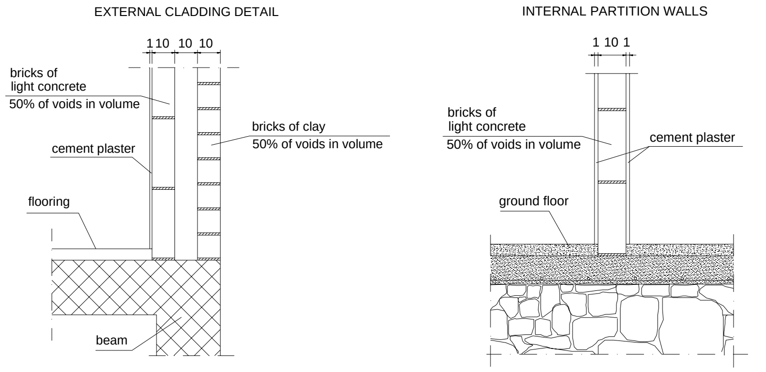

Figure 4. Details of external claddings and internal partition walls

1.3. Repairing and strengthening

After the first test, the building was partially repaired. In particular, only some damaged elements of the building were repaired (perimeter columns) or rebuilt and strengthened (external MIPs parallel to the loading North-South direction), while the other elements (internal columns, partition walls and staircase structure) were not repaired nor substituted. The perimeter MIPs with the plane parallel to the loading direction were rebuilt using materials having geometrical and mechanical properties as close as possible to those of the original elements. After the erection of the external MIPs, the facing walls were strengthened by means of the fiber reinforced polymers (FRP) structural repointing technique. This technique consists in placing composite FRP bars in the masonry bed joints, using either an epoxy paste or a cement mortar for bonding.

Materials used for the strengthening were: sand-blasted carbon fiber rods (“MBarTM Joint” by Degussa Construction Chemical) having 1.5 mm thick and 5 mm wide rectangular cross-section; characteristic tensile strength of 1300 MPa; average tensile modulus of 70000 MPa; average ultimate deformation of 1.8%. Pre-mixed, thixotropic, fiber reinforced, shrinkage compensated cement mortar (“Emaco® Formula Tixo” by Degussa Construction Chemical) having compressive strength (28 days) more than 60 MPa; adhesive strength more than 6 MPa; modulus of elasticity more than 28000 (± 2000) MPa.

The installation procedure consisted in the following phases (Fig. 5): (a) grinding of joints: this phase consists of the cutting out part of the mortar using a grinder; (b) installation of carbon fiber rods in the bed-joints previously raked out; (c) bonding of carbon fiber rods with the pre-mixed cement mortar.

The repairing and strengthening of the damaged end portions of the external columns was carried out by removing degraded concrete and reconstructing it with the “Emaco® Formula Tixo” pre-mixed cement mortar. The portions of the steel bars permanently bended have been removed and replaced (just one location over the whole building) using new ribbed bars having the same diameter and similar grade as the original bars. Lap splice joints of length equal to about 60 bars diameter were provided between old and new portions of bars. The column repairing is shown in Figure 6.

The main objective of the FRP strengthening system for MIPs was to change the failure mode from the predominant diagonal tension cracking, as it was observed during test #1, to predominant shear sliding. In fact, the diagonal tension cracking failure mode is deemed to be relatively more brittle than the shear sliding mode. Thus, having dominance of shear sliding would help the MIPs maintaining their integrity during the lateral loading, with a favorable effect on life-safety performance levels. Furthermore, the strengthening of MIPs can favorably affect high-level performance objectives, such as the “operational” level, as testified also by previous research on this topic (Calvi and Bolognini, 2001).

It was decided to place FRP bars in the bed mortar joints of only the external façade of the semi-hollow brick walls, i.e. a one side strengthening system was selected. This choice was made because of simplicity, low-cost and reversibility of the intervention. However, since available literature seemed to suggest limited effectiveness of one-side strengthening, it was decided to apply FRP bars in every joint, in order to get the maximum possible improvement of the shear strength. In addition, FRP rods were placed also in the secondary sub-panels, when windows are present, because of the aim of limiting deleterious effects of stress-concentration at corner joints.

(a) Grinding of joints (b) Installation of rods (c) Application of cement mortar

Figure 5. FRP structural repointing: installation procedure

Figure 6. Column repairing

2. TEST SETUP AND INSTRUMENTATION

The loading direction was established based on the local site conditions, namely depending upon the possibility and simplicity to erect a new structure serving the function to contrast the horizontal loads to be applied. Then, the North-South direction (Fig. 1) was selected as the most appropriate. At the same time, this loading direction is the most interesting one, because it involves the staircase structure to resist horizontal loads by axial forces combined with flexural actions, hence maximizing its "bracing" effect.
Test set up and instrumentation used in both tests are identical. The purposely designed reacting steel structure is shown in Figure 7. The lateral loads have been applied by means of hydraulic jacks, alternately pushing and pulling the structure through selected target storey displacements. Another triangulated steel structure was used to distribute the applied load between the two building floors, as shown in Figure 7. The load centre was fixed in such a way to approximately get an inverted triangular load distribution, which is a frequently adopted conventional load-distribution in theoretical pushover analyses of low-rise buildings. Storey displacements were monitored by using a diastimeter (Zeiss-Trimble S10) and reflecting prisms. The measuring points were fixed in correspondence of the two building floors, thus getting the average storey translation and its rotation about the vertical axis.

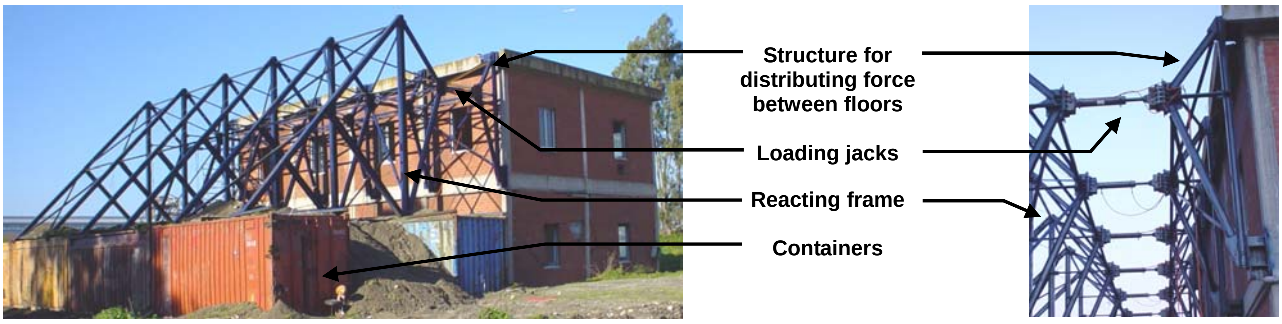

Figure 7. Reacting steel structure

3. RESULTS FROM TESTS ON THE ORIGINAL BUILDING

Figure 8 shows the base shear force vs. the first-storey average drift angles. The latter have been computed as the average values among the measuring points located at the floor.
After two cycles of fully reversed loading in the quasi-elastic field of response, the structure has been pushed up to a maximum first-storey drift ratio of about +5% (Fig. 8), which produced extensive damage in both MIPs and RC frame columns at the first storey of the structure. Figure 9 shows the damage state visible from outside of the building, at the point of maximum storey drift ratio, for the perimeter infill-walls having their plane parallel to the load direction. After reaching the maximum lateral displacement, the lateral force applied by the hydraulic jacks was fully reversed, now pulling the structure up to a lateral drift of opposite sign equal to about -3% (Fig. 8). At this lateral displacement the structure damage at the first storey was very extensive, with the out-of-plane collapse of almost all the MIPs (both external claddings and internal partitions) having their original planes parallel to the loading direction. After reaching the point

of minimum lateral displacement, the structure was again pushed in the positive direction and a few more small loading cycles were necessary to be applied for approximately re-centring the structure in its original position.
The main failure modes observed in the infills were (Fig. 9): diagonal tension cracking in the infill #1; masonry crushing, sliding and diagonal tension cracking in the infill #2; diagonal tension cracking in the sub-panel #3; sliding and diagonal tension cracking in the sub-panels #4 and #5; diagonal tension cracking in the infill #6. Along with the MIPs also the first-storey frame columns and the staircase structure were severely damaged during the loading test. Damage in the RC frame was characterised by flexural-shear plastic hinges formed at column ends, with the strongest shear effects occurring at the top end of columns located adjacent to the strongest masonry panels and for large inelastic displacements. The test has shown that damage in the MIPs was negligible up to an inter-storey drift angle equal to about 0.002 rad (at this point the first cracks appeared with amplitude less than 1mm). At 0.003 rad damage was very clear but still moderate, with amplitude of cracks of a few millimeters at maximum. From 0.004 rad through 0.005 rad damage in the MIPs became extensive, the size of cracks reaching several millimeters. At 0.005 rad, which approximately corresponds to the maximum lateral strength achieved in the test, also the corner crushing of MIPs was extensive and an evident gap between MIPs and columns occurred.

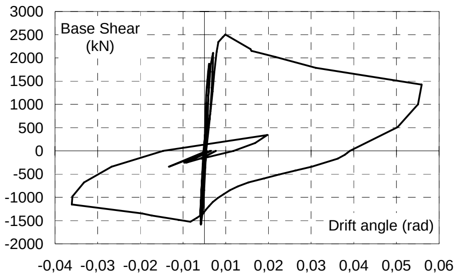

Figure 8. Base shear vs. average first-storey drift angle response

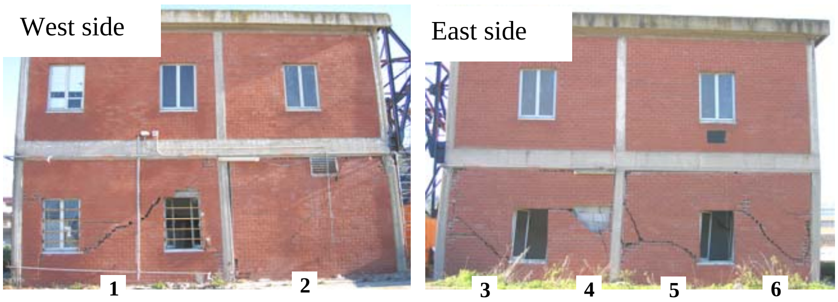

Figure 9. Damage of perimeter masonry walls at the point of maximum lateral displacement (+20 cm)

4. RESULTS FROM TESTS ON THE BUILDING PARTIALLY REPAIRED

Figure 10 shows the response of the repaired building in terms of base shear vs. average first-storey drift angles. The repaired building has been subjected to a preliminary single cycle of fully reversed loading in the quasi-elastic field of response; then it has been pushed up to a maximum first-storey drift ratio of about 7.5% (Fig. 10).
Figure 11 shows the damage state of the exterior façades parallel to the load direction for a drift ratio close to the peak value reached during the test on the original building. The dominant failure modes were: sliding in the infills #1, #3, #4 and #5; masonry toe crushing and sliding in the infill #2; sliding and diagonal tension cracking in the infill #6. Damage in the RC frame was similar to what observed during the test on the original building.
After reaching the maximum lateral displacement, the lateral force was reversed and the structure was pulled up to a negative drift ratio of about 6.5% (Fig. 10). At this point the structure damage at the first storey was very extensive, with the full or partial out-of-plane collapse of all the MIPs parallel to the load direction. Also in this test, after reaching the point of minimum lateral displacement, the building was again pushed in the positive direction and, finally, small loading cycles were applied for re-centring the building in its original position.
Comparing Figure 9 and Figure 11 it may be noted that repaired infill walls exhibited different failure modes, characterized by a reduced influence of the diagonal tension cracking mode. In addition, it has been observed that the damage level in the strengthened MIPs was smaller than that exhibited by the original MIPs at the same level of lateral displacement.
Comparing the maximum lateral strength obtained in the second and first test (Figure 10), a reduction of about 60% (+1021 kN vs. +2501 kN) and 50% (+-803 kN vs. -1527 kN) are obtained for pushing and pulling phases, respectively. This result can be attributed to the extensive damage produced during the first test to the staircase structure and to the partition walls. Also damage produced in columns may have contributed, because only marginal repairing was carried out for them (just the most damaged in shear and flexure, as shown for example in Figure 6). Therefore, the strength measured at large drifts during the test #2 is to be considered a residual value of the RC columns after the damage sustained during the previous loading cycles.

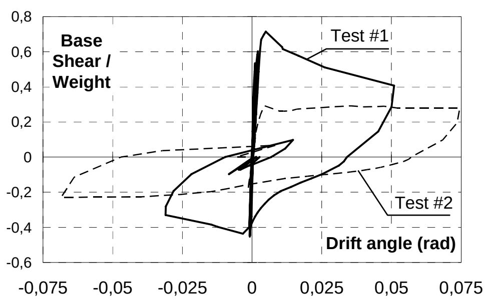

Figure 10. Normalised base shear vs. average first-storey drift angle responses

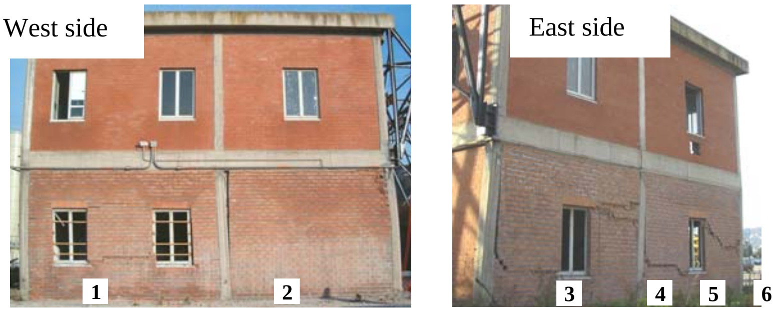

Figure 11. Damage of perimeter masonry walls at a lateral displacement of +20 cm

5. SIMPLIFIED ANALYTICAL MODELLING OF BUILDING RESPONSE

In order to investigate about the possibility to analytically predict the observed behavior, a simplified model has been developed. The following components have been considered as contributing to the lateral stiffness and strength: (1) the RC columns; (2) the staircase structure; (3) the perimeter MIPs (cladding panels parallel to the load direction); (4) the internal MIPs (partition walls parallel to the load direction).

The equivalent diagonal strut approach has been assumed to model the structural contribution of MIPs (Stafford-Smith 1966, Klingner and Bertero 1978). It is also assumed that MIPs are characterized by a negative post-peak stiffness equal to 5% of the initial stiffness and a residual strength equal to 10% of the peak value. Each MIP including windows has been subdivided in sub-panels. The "effective" height of the sub-panel has been defined according to the criteria suggested by Dolce (1991) for load-bearing masonry walls (Figure 12*a*).

The contribution of the staircase structure to the total stiffness and strength of the building has been estimated by schematizing the inclined RC slabs as additional diagonal struts. The simplified model is shown in Figure 12*b*. The inclined RC slabs constituting the staircase have been treated as beam-column elements, having the cross-section of the slab. The flexural response in the vertical planes parallel to the loading direction has been neglected, while the elastic restraining effect coming from the bending about the strong axis has been considered (rotational springs shown in Figure 12*b*). Two main non-linear events characterise this sub-structure: (i) yielding of the inclined strut and, subsequently, (ii) flexural plastic hinging of the supporting RC columns. Since the secondary stiffness (after yielding of the inclined elements) resulted to be a relatively small percentage of the initial stiffness, it has been neglected and an elastic-perfectly plastic model has been assumed for the staircase, using the initial elastic stiffness and the total strength of the substructure shown in Figure 12*b*.

The RC column contribution has been estimated assuming that they are fixed at the ends and subject to relative transverse displacements. An effective moment of inertia equal to 20% of the gross cross section property has been used (Elwood and Eberhard 2006). Besides, an effective column height has been assumed, smaller than the clear column length, because of the need to consider column-panel interaction. The effective height of columns has been assumed equal to the effective height used for computing the stiffness and strength of panels. The columns' contribution to the total strength is estimated assuming plastic hinge formation at each column end and locating plastic hinges at a distance from the effective column ends equal to one half of the column depths. A different approach was required for the column adjacent to the panel #2, where shear effects were non-neglgible (damaged column in Figure 6). In this case, the eccentricity of the equivalent diagonal strut, suggested by FEMA (2000) and discussed by Al-Chaar (2002), has been first computed, thus determining the presumed length of the portion of column subject to strong shear forces. This length resulted in a value of about 700 mm, which is in good accordance with the experimental observation of the length of column subject to strong damage (Figure 6). Henceforth, the stiffness of the column has been computed by considering the series composition of the flexural and shear flexibilities relevant to the portion of the column having a length equal to 700 mm. Eventually, the coefficient 0.2 has been applied to the total stiffness (to consider concrete cracking and bar slipping).

Results of the analytical computation of the building response during the initial part of test #1 are highlighted in Figure 13*a* and here compared with the experimental results. Three analytical curves are reported, corresponding to three different hypotheses on the strength of the materials consituting the MIPs: Paulay and Priestley (1992), CEN (2005), Italian Ministry of Public Works (1987). The force-deformation relationships analytically derived for each of the resisting components of the building are also reported in Figure 13*a*, with reference to the upper curve (i.e. the curve obtained using masonry properties computed according to Paulay and Priestley (1992)). It is seen that the analytical model results are quite close to the experimental response and the agreement can be considered very good if the simplicity of the analytical model is considered.

The analytical simulation of the response obtained with the test #2 is more difficult to be obtained, because of strength and stiffness degradation occurred in the RC members after test #1. For the sake of simplicity, it has been assumed that the staircase structure does not contribute anymore to the lateral response, while the column contribution has been degraded with empirical degradation factors equal to 0.8 for the strength and 0.5 or the stiffness. The analytical results are reported in Figure 13*b*, again with reference to the three different models for computing the strength of masonry panels. Analytical results are also in this case in good agreement with the experimental ones.

Based on the analytical models that more closely represents the response of the building during test #1, the contribution of the different components to the stiffness and strength of the whole building can be estimated.

Namely, the ratios between the analytical peak strength of each resisting component and the analytical peak strength of the whole building, for the test #1, are as follows: columns = 46%, staircase= 40%, masonry panels = 20%. As far as the stiffness is concerned the following percentage distribution is obtained: columns = 17%, staircase= 41%, masonry panels = 42%. Furthermore, the distribution of shear strength and stiffness between perimeter cladding panels and interior partition panels is as follows: interior panels = 25%, cladding panels = 75%, for strength; interior panels = 21%, cladding panels = 79%, for stiffness.

It is apparent that the calculated distribution of stiffness and strength among the different components cannot be generalized to every situation, because the percentage influence of both the staircase and masonry panels depends upon the number of RC columns, i.e. upon the plan area of the building. This is just a case illustrating that the contribution may be large and it could be appropriate to be taken into account, in order to get a correct picture of the seismic response for small-to-medium inter-story drift angles.

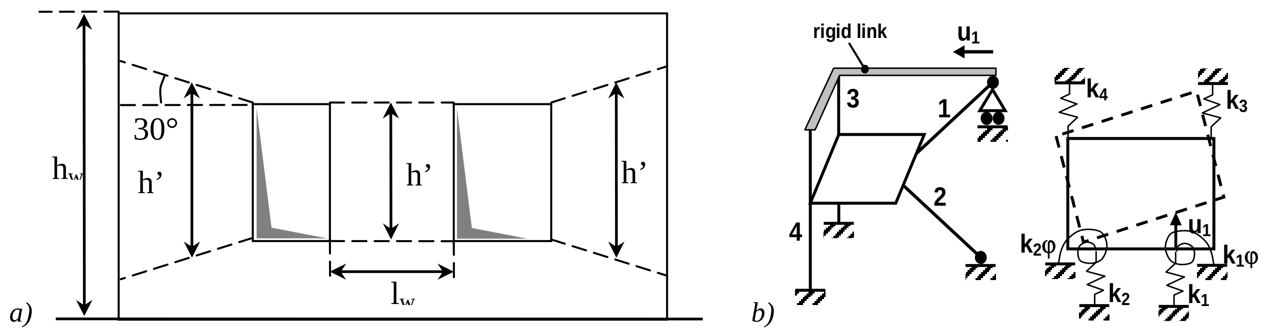

Figure 12. Basic assumptions for the simplified analytical model

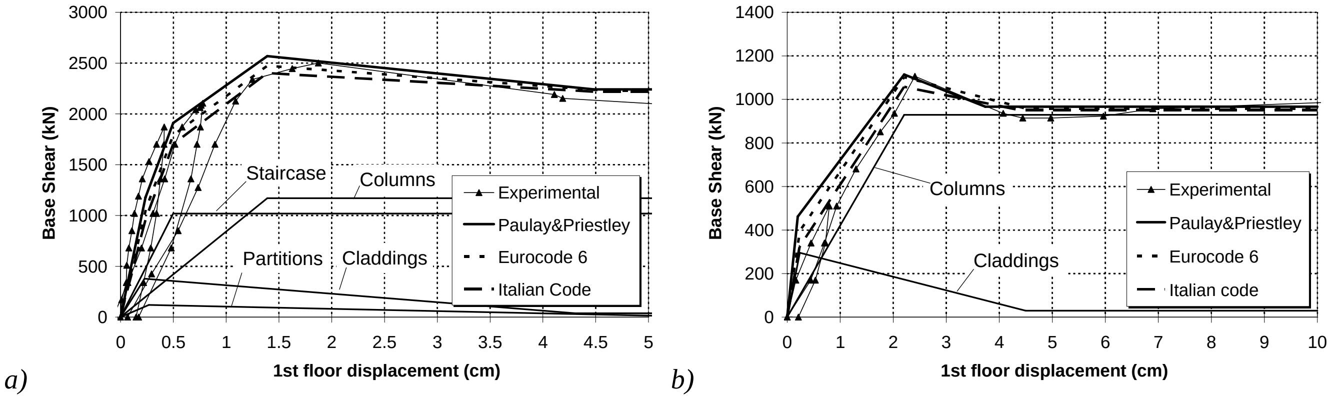

Figure 13. Comparison of analytical model vs. experimental results

6. CONCLUSIONS

Based on test results summarised in this paper, the following main conclusions can be drawn:

1. Damage to MIPs starts for a drift angle of about 0.002 rad, develops between 0.002 and 0.005 rad and becomes very large for drift angles in between 0.005 rad and 0.015 rad.
2. The near surface mounted FRP rods, placed in the mortar bed joints, revealed to be effective in changing the failure mode of MIPs from dominant diagonal tension cracking to prevailing shear sliding mode.
3. An analytical estimate of the pushover response of the whole building has been obtained by summing up the contribution of three parallel components: the RC columns, the staircase structure, the MIPs. A simple analytical model has been proposed for considering the effect of the staircase structure. Notwithstanding uncertainties related to material properties, the method resulted in a good representation of the response exhibited during tests. According to model results, the effect of both the staircase structure and MIPs on the initial building response may be very large.

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