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EXPERIMENTAL RESPONSE AND NUMERICAL MODELLING OF TWO-STOREY INFILLED RC FRAME

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Abstract

Recent devastating earthquakes pointed out the high vulnerability of existing reinforced concrete (RC) buildings and the critical role of infills. The presence of stiff infills may significantly modify the lateral response of RC moment resisting frame buildings and cause severe damage on the surrounding RC members due to seismic actions transmitted at level of beam-column joints. Furthermore, due to their brittle response, they commonly exhibited severe damage often leading to high economic losses. In this context, effective seismic retrofit strategies should aim at both increasing the shear strength of RC members and reducing the damage to infills. However only few tests are available in literature addressing the seismic strengthening of existing RC frames accounting for the infill-to-structure interaction. To fill this gap a comprehensive experimental program of pseudo-dynamic tests on full-scale two-storey infilled RC frames has been designed and is currently ongoing at the full-scale laboratory testing of the University of Napoli Federico II.

This paper reports the preliminary experimental results and numerical analyses carried out by using available non-linear models accounting for the infills contribution. The comparison between theoretical predictions and experimental results provides useful insights to improve the numerical models to reproduce the infill-to-strut interaction. Finally, a retrofit strategy to improve the seismic performance of the structural system and reducing the expected damage to infills is herein outlined.

Keywords: pseudo-dynamic, full-scale, shear failure, FRP

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1 INTRODUCTION

Recent devastating earthquake pointed out the high vulnerability of existing reinforced concrete (RC) buildings designed with obsolete code provisions. This resulted in number of fatalities, injuries and massive economic losses [1]. Severe damage were commonly found on hollow clay brick infills (see Figure 1a) [2] and at level of beam-column joints with marked shear cracks involving in many cases also the top end of the columns; this latter effect is caused by the interaction with stiff infills (see Figure 1b). Thus, stiff infills have a crucial role in the seismic response of existing RC buildings, modifying their dynamic response [3–5], increasing the shear actions transmitted to surrounding RC members [6], and, last but not least, experiencing significant damage and high economic losses due to their brittle response [7]. This remarks the need to properly account for the infill contribution to the lateral response of RC buildings and for the infill-to-structure interaction that may lead to premature shear failure of poorly detailed columns.

(a) (b)

Figure 1: Case study building: front view (a); damage to RC column due to infill-to-structure interaction (b).

In the last decades, a significant research effort was devoted to the experimental testing [3,6,8–10] and numerical modelling of RC building or frames with hollow clay brick infill and partitions [5,6,11,12]. This resulted in number of analytical and numerical approaches to assess the infill contribution to the lateral response of RC buildings. Nowadays different numerical models are available [3,6,9,13–16]. They can be used to assess the damage state of the infills and the action transmitted to the surrounding frames. Furthermore, refined analytical models are recently proposed to capture the column shear failure due to the infill-to-frame interaction.

Although number of studies can be found in literature to assess the contribution of infills to the lateral response of RC buildings, only few researches addressed the experimental behavior of full-scale RC frames and their seismic strengthening to resist the infill action [17,18]. This paper presents the preliminary results of pseudo-dynamic experimental tests carried out on infilled full-scale RC frames. The experimental outcomes are reported and compared with the preliminary results of a numerical model. Then, the model is used to assess internal actions in RC members that can be used for the design of suitable strengthening solutions.

2 EXPERIMENTAL PROGRAM

The experimental program consists of pseudo-dynamic tests on a full-scale two storey infilled RC frame (see Figure 2). The frame reproduces a perimetral frame of a real building damaged during the L'Aquila earthquake (2009), see Figure 1, and re-built due economic inconvenience of repair and structural retrofit [1]. Indeed, the severe damage to structural and non-structural components (see Figure 1) resulted in very high repair costs.

Figure 2: Front view of the prototype infilled RC frame.

The selected frame is extracted from the building depicted in Figure 1. It is 6.86 m high and 4.50 m long. The interstorey height is about 3.10 m and the foundation block is 0.56 m high. The square columns, 0.40 m side, are reinforced with 8 ϕ 16 mm longitudinal bars and ϕ 8 mm transverse reinforcement 250 mm spaced. The beam is 0.55 m high and 0.40 m width reinforced with 6 ϕ 16 mm and 4 ϕ 16 mm bars at top and bottom side, respectively. The joint panel have no transverse reinforcements as commonly observed in existing buildings built in the last century in the Mediterranean area.

The concrete compressive strength of the tested frame is about 8 MPa corresponding to a very poor-quality concrete as often found in many existing buildings in the Mediterranean area [19].

Hollow clay brick stiff infills are used at both floors. Square bricks with 250 mm side and 200 mm thick are used. Furthermore, 10 mm joints of M10 class mortar are used to build the infill walls. The mechanical characterization tests on wall samples showed a shear strength of about 0.35 MPa.

The experimental program consists of pseudo-dynamic tests at increasing intensities. In particular, the AQG record in the Est direction (PGA=0.45 g) of the 2009 L'Aquila earthquake is used as input acceleration. The Alpha-OS [20] integration algorithm is used for the numerical solution of the equation of motion and calculate step-by-step the displacement profile to apply at the different floors. The mass matrix is defined considering two lumped mass at the level of the two floors. They are calculated making simple considerations on the substructuring of the test. In particular, it is assumed that the portion of the floor mass belonging to each frame is function of the elastic stiffness of the frame. In this case, exterior frames are considered infilled and interior frame bare. The test results in terms of recorded displacement histories at the two floors as well as in terms of global hysteretic response are reported in the following and compared with the numerical predictions.

3 NUMERICAL MODELLING

A non-linear FEM model has been developed in the SAP2000 environment [21]. An overview of the adopted model is reported in Figure 3 along with the selected capacity models used to characterize the structural members and the infill walls.

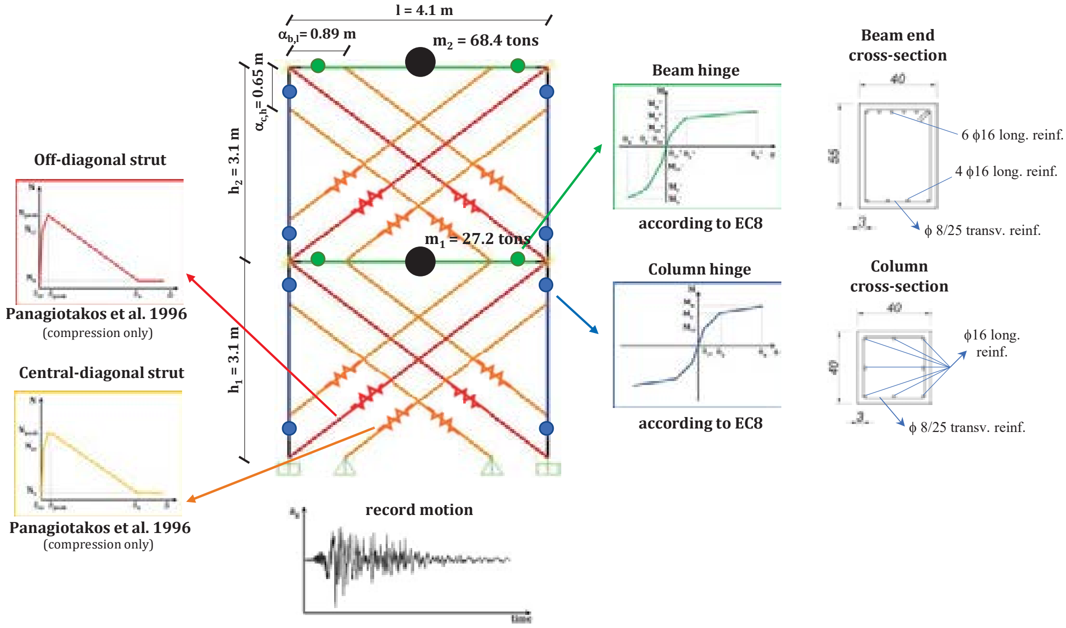
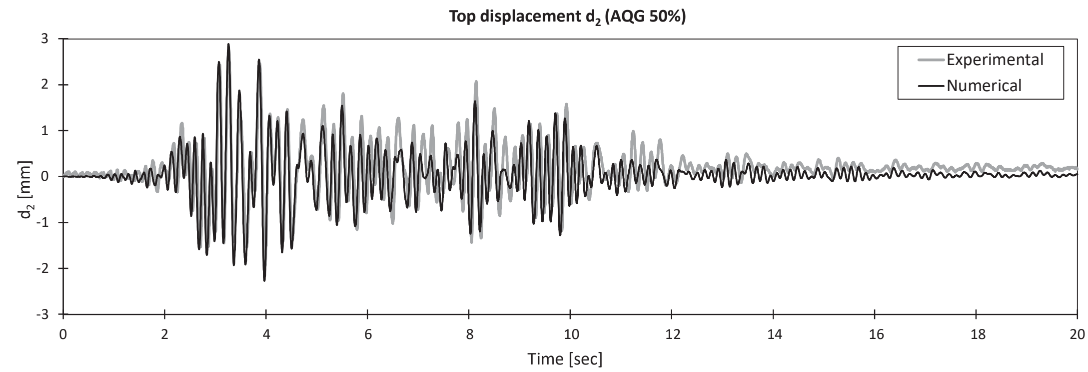


Figure 3: Overview of the adopted numerical model.

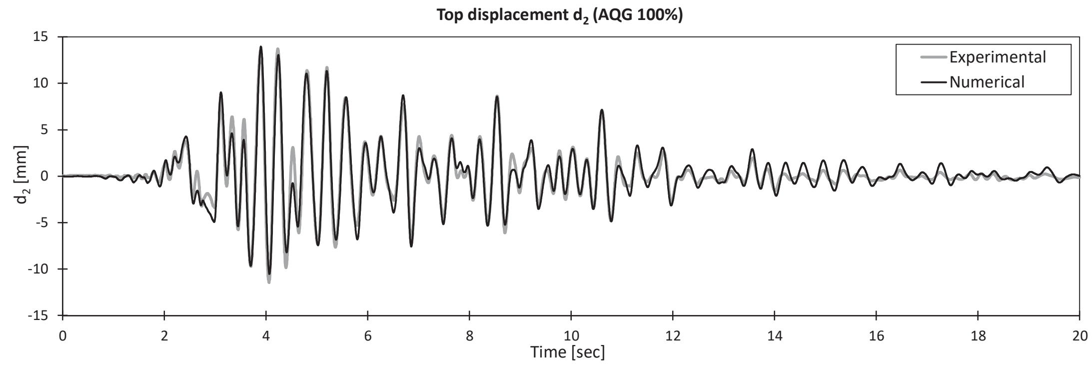
The frame is represented by using the member axis. All the nonlinearities are lumped at the member ends. The plastic hinge properties are characterized by using the capacity models suggested by the Eurocode 8 [22] and Italian building code [23,24] for the plastic hinge rotation at the yielding and at the ultimate limit state. The contribution of the joint non-linear response to the building lateral deformation is neglected at this stage. The infill walls are included in the model by means of diagonal compression struts in the perimetral frames. A three-strut model is considered for this study. Indeed, it is demonstrated that this model can reproduce the actual force distribution transmitted by the infills to the surrounding frame members [6]. The mechanical properties of the strut are defined according to the Panagiotakos and Fardis model [3]. The mechanical properties of the hollow clay brisk masonry wall are assumed according to mechanical characterization tests (i.e. τ_{cr} = 0.35 Mpa). The distribution of the portion of the infill lateral load absorbed by each strut assumed in this work is compliant with the proposal by Jeon, Park, and DesRoches [25], while the mechanical characterization of the off-diagonal struts have been reproduced according to the proposal by Chrysostomou et al. [26]. Furthermore, the use of a three-strut model allow to identify the actual distribution of shear on the column due to the interaction of the RC frame with the infill walls [6]. Nonlinear time-history analyses (NLTH) are carried out imposing the input motion at the base and concentrating the masses in the center of the beam. A mass of about 27.2 tons is applied at the first level, while a mass of 68.4 tons at the second level. The reference mass of the selected frame is calculated distributing the total floor mass of the reference building (see Figure 1) as function of the elastic stiffness of the frame respect to the total floor stiffness of the building. The masses of the third and forth floor are lumped at the second level assuming that they behaved as rigid floor. This assumption is made according to those made in the substructuring of the pseudo-dynamic test as discussed in the previous section.

4 COMPARISON OF EXPERIMENTAL AND NUMERICAL RESULTS

A direct comparison of the experimental and numerical response in terms of measured top displacement, d_2, during the pseudo-dynamic test at 50% and 100% of the 2009 L'Aquila earthquake is depicted in Figure 4.



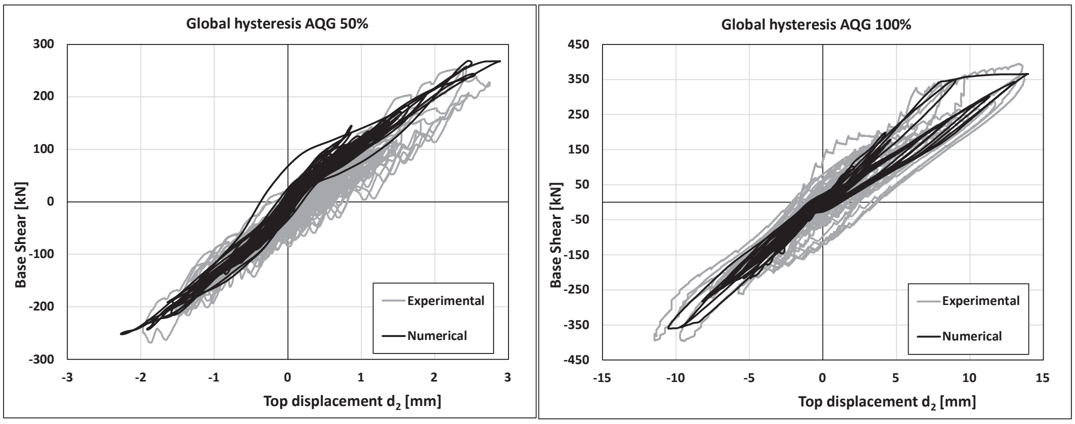
(a)



(b)

Figure 4: Comparison of experimental and numerical results in terms of top displacement: 50% of the selected record motion (a); 100% of the selected record motion (b).

The comparison outlines the accuracy of the proposed modelling procedure that is capable of providing reliable estimations of the top displacement at all the steps of the test. In particular, the best match between experimental response and theoretical model at 50% of the earthquake intensity (Figure 4b) is attained in the first 5 seconds of the test. Also, the peak displacement of about 3 mm is well captured by the model. In the following time steps, the model predicts a displacement lower than the experimental one as a consequence of a theoretical stiffness higher than the actual one. This can be clearly observed comparing the experimental and global stiffness expressed as the ratio of the base shear and the top displacement showed in Figure 5a. Although a quite good matching can be observed also in the hysteretic response, the numerical model have a post peak degraded stiffness higher than the experimental one in the range 0-1 mm. This results in an underestimation of the top displacement in the post peak after 5 s in the time-history response.



(a) (b)

Figure 5: Comparison of experimental and numerical results in terms of global hysteretic response: 50% of the selected record motion (a); 100% of the selected record motion (b).

With reference to the test at 100% intensity of the L'Aquila 2009 earthquake, a top displacement of about 13.5 mm is achieved. It is significantly larger than the double of the one achieved at 50% due to the stiffness degradation in the infill walls. This can be clearly observed comparing the two responses in Figure 5, where a drop of the experimental secant stiffness of about 1/3 to 1/4 can be observed. The numerical model well captures both the displacement time history and hysteretic response resulting in an good accuracy of the predicted displacement and base shear during the test.

5 STRUCTURAL DAMAGE AND RETROFIT STRATEGY

During the test at 100% of the earthquake intensity a significant shear cracking at top of the ground floor columns was observed. This is related to the infill-to-structure interaction and could be triggering of a shear failure and collapse as observed in the post-earthquake inspections in many exiting RC buildings. In order to assess the reliability of the proposed modelling procedure to assess the shear failure at the top of the columns due to the infill-to-structure interaction, the model proposed by Verderame et al. [6] is used to evaluate the shear strength. According to experimental evidences and other available literature models [27,28], the shear strength degrades as function of the inelastic demand at the top of the column. Despite other degrading models, it allows to account for the actual internal force distribution due to the interaction with the off-diagonal infill strut insisting on the column. The overlapping of the shear strength with the shear demand at the top of the column due to the infill action is depicted in Figure 6. The intersection between capacity and demand allows to recognize the shear failure detected at a top displacement at the ground column of about 4.15 mm corresponding to about 0.2% drift. This result clearly remarks the key role of such failure mode to properly assess the seismic response and the structural safety of existing infilled RC buildings. In order to avoid such a kind of failure, different strengthening solutions can be found in literature [17,29,30]. However, it should be considered that recent financial incentives provided by the

Italian government, consisting in 100% tax deduction of the cost of seismic strengthening interventions, exacerbated the need to have light strengthening intervention that can be applied to the structural system with a minimum level of disruption [31,32].

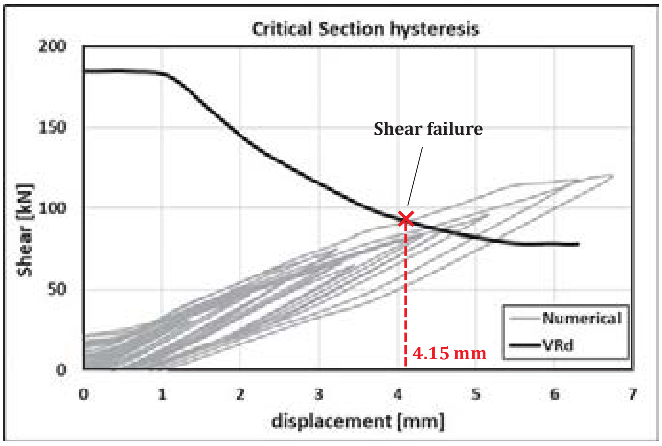


Figure 6: Comparison of the shear demand at the ground floor column top end and shear capacity

This will help the widespread of seismic strengthening interventions to large scale and may fulfill the needs to significantly improve the seismic response with the low disruption to occupants. In this context composite materials may represents a sound solution because they combine the advantage related to the easy installation procedure with their high performances to increase the shear capacity of RC members and beam column joint panels [33]. In light of the observed damage and the structural weaknesses outlined from numerical analyses, the strengthening layout proposed in Figure 7 is initially outlined.

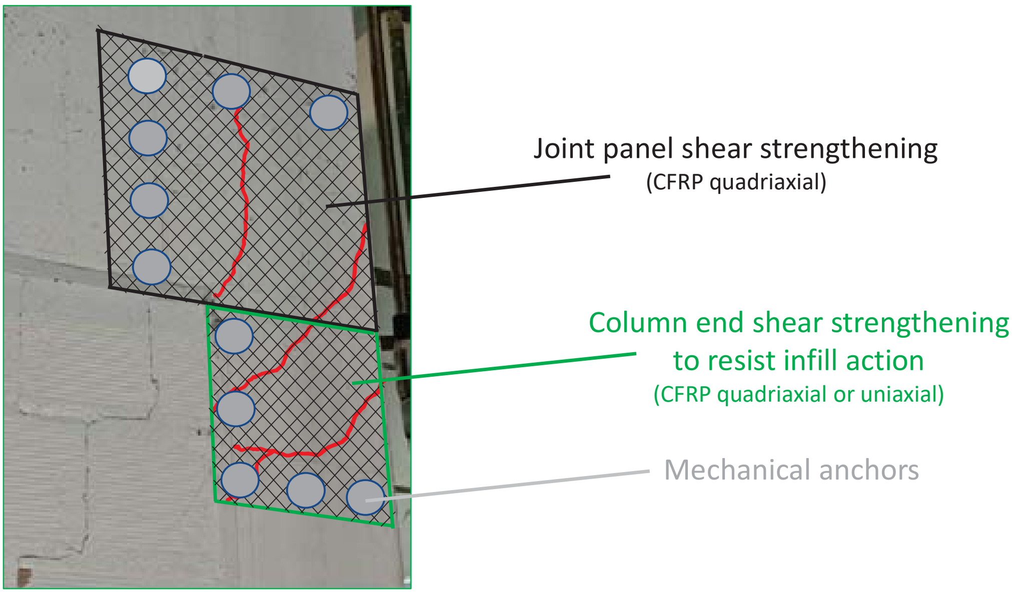


Figure 7: Strengthening system for the RC frame.

It consists of the column end shear strengthening by using Quadriaxial or uniaxial CFRP wrapped around the column on the two sides. This is in line with the strengthening system proposed in the ReLUIS guidelines [29] to resist the infill action. Then quadriaxial fabric is also outlined on the joint panel serving as joint shear strengthening to avoid the joint panel shear cracking, which are commonly observed in poorly detailed subassemblies. Number of layers of quadriaxial fabric on the joint panel can be designed according to the formulation proposed in the fib bulletin 90 [34] properly developed and validated on experimental data [29]. The novelty of this new proposal consists in the use of FRP mechanical anchors to replace the commonly adopted U-wrap on beams and columns useful to avoid end-debonding. This may further reduce the disruption to occupants in real field applications. This is a promising technique [32] that requires further experimental investigation to validate its use at large scale. Ongoing experimental tests will show the advantages and limitations of such strengthening solution.

6 CONCLUSIONS

This research focuses on the design and analysis of an experimental pseudodynamic tests on a full scale infilled RC frame representative of a portion of an existing RC building damaged by the L'Aquila 2009 earthquake. The analytical modelling of the specimen behavior under seismic actions is also presented. In particular, the experimental results in terms of time histories of the top displacement and global hysteretic response are showed and compared with the results of a non-linear FEM model. The following preliminary conclusions can be drawn:

- the experimental tests remarked the role of stiff infills in the lateral response of existing RC frames. They increase the stiffness of the building resulting in a reduction of the displacement demand and in an increase of the seismic forces that will be transmitted to surrounding RC members;
- the proposed non-linear numerical model consisting in a lumped plasticity model for the RC frame and in a three-strut model to reproduce the infill contribution well captured the global displacement and hysteretic response occurred in the experimental tests;
- the proposed numerical model allowed to quantify the actual internal force distribution with particular emphasis on the shear actions transferred from the infill to the RC columns;
- literature shear strength capacity models properly developed to account for the top column shear failure due to infill-to-structure interaction allows to accurately predict the failure mechanism and to provide useful insights for the design of an effective strengthening intervention.

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