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EARTHQUAKE RETROFITTING OF "SOFT-STORY" RC BUILDINGS

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Abstract

Multi-story "old" reinforced concrete (RC) structures with a "soft-story" at ground floor, due to the presence of masonry infills at the upper stories, sustained considerable damage at this level during past earthquakes. Aspects of such masonry infill - RC frame interaction are briefly discussed and a particular retrofitting scheme for the "soft-story" is studied. It consists of RC infills, added within the bays of the ground floor frames and combined with RC jacketing of the surrounding frame, aiming to avert such "soft-story" deficiency. The impact of such a retrofit is studied through the measured response of 1/3 scaled single-story one-bay frames subjected to cyclic seismic type horizontal loads. It is shown that this retrofit results in a considerable beneficial increase in stiffness, strength and plastic energy consumption. The importance of the presence of effective steel ties connecting this RC infill with the surrounding frame is also demonstrated. In order to achieve these desired beneficial effects to such vulnerable buildings additional design objectives are to avoid premature failure of the RC infill panel and/or fracture of the steel ties and to protect the surrounding RC frame from undesired local damage. A numerical methodology, which is validated using the obtained experimental results, is shown to be capable to predict reasonably well these important response mechanisms and can therefore be utilized for design purposes.

Keywords: Upgrading old RC frame structures, soft-story retrofit; RC infills, masonry infills

1 INTODUCTION

The seismic vulnerability of multi-story reinforced concrete (RC) framed structures built according to relatively old seismic code provisions increases significantly when their ground floors bays are left open to function as parking spaces whereas the bays of the upper stories are infilled with unreinforced masonry panels. It was demonstrated by extensive past research that the dynamic and earthquake behavior of such structures, having a relatively flexible ground floor ("soft story") and stiff upper stories, results in increased demands of the ground floor columns and shear walls that are not designed for [1]. This is due to the interaction of the masonry infills with the surrounding R/C frames which contributes to a substantial increase of the story stiffness of these upper stories compared to the story stiffness of the ground floor [1, 2, 3]. This, in turn, leads to structural damage (figure 1, 2), unless the structural R/C elements at the ground floor are properly designed for.

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Figure 1: Damaged columns at the soft story and temporary steel shoring

Figure 2: Damaged columns at the soft story

It is crucial to identify this problem and to introduce appropriate retrofitting countermeasures before such vulnerable buildings are subjected to strong earthquake sequences that may lead to severe structural damage. The basic objectives of a permanent retrofitting scheme in such cases are: To increase the shear and flexural capacity of the ground floor columns; To provide the structural response of this ground floor story with an increased stiffness, shear capacity and possibly ductility. In the relevant provisions of these guidelines (OASP 2012) the designer is provided with a number of distinct choices for a retrofitting technique denoted as “Reinforced Concrete Infill” whereby a reinforced concrete panel is cast in place filling a selected ground floor bay of a RC framed structure [4, 5]. The following choices are studied briefly here:

a. The RC infill panel (RC-IP) is not structurally connected to the surrounding R/C structural elements within (columns or beams). Alternatively, a limited connection between the RC-IP and the upper/lower horizontal frame interface is constructed.

b. The RC infill panel is constructed together with steel ties connecting the RC infill panel with the surrounding RC structural elements strengthened by jacketing within the bay of a frame (columns or beam). The thickness of this RC-IP is usually smaller than the width of the beams and columns that form the infilled bay of the frame.

2 METHODOLOGY

Laboratory tests were carried out in order to investigate strengthening schemes of “soft story” sub-assemblies. For this purpose, the following methodology is used. A 1/3 scaled one-bay single-story specimen is used with all information of its geometry, mechanical characteristics of all the materials and structural detailing defined by testing and depicted at figure 3 and table 1 [6]. This specimen is step-wise transformed to: a) A retrofitted frame with RC jackets b) A retrofitted frame hosting an RC infill not connected to this frame c) An unreinforced concrete infill connected to the same retrofitted frame with steel anchors. d) The same as before but having a lightly reinforced RC infill. All these specimens were subjected to a vertical load in order to simulate an axial force level for the column of the frame whereby at the same time an in-plane horizontal cyclic load was applied at the top bay of the frame of continuously increasing amplitude. This measure response was then used to compare predictions of the response employing a specific numerical analysis process. In particular, the following specimens are studied:

- Bare 1. This is a RC frame specimen without any jacketing in the columns and top beam with the code name “Bare 1” the same with the one depicted in figure 3.
- Unreinforced concrete –Infill Panel 1. This specimen was formed by filling the bay of specimen Bare 1 with an unreinforced concrete panel having a thickness of 50mm with the concrete compressive strength equal to 22MPa. This concrete panel was simply cast within the bay without the use of any ties between this panel and the structural members of the surrounding frame with the code name for this specimen is “UC-IP 1”.
- Bare jacket1. This specimen (figure 4) was formed by removing the fractured unreinforced concrete panel of specimen “UC-IP 1” and by retrofitting the columns and part of the beam of the RC frame near the beam-to-column joints with concrete jackets. The cross-section of the jacket was 180mm by 180mm having at each of its 4 corners longitudinal steel reinforcing bars of 8mm diameter and 570MPa yield stress. These jackets were also reinforced with closed hoop steel stirrups of 5.5mm diameter spaced every 50mm intervals. These jackets were cast with high strength concrete having a compressive stress of 40MPa. The code for this specimen is “Bare jacket 1”.
- Unreinforced Concrete-Infill Panel 2. This specimen (figure 5) was formed by filling the bay of specimen Bare J1 with an unreinforced concrete panel having a thickness of 50mm with the concrete compressive strength equal to 22MPa. This time the unreinforced concrete panel was cast within the bay using 8mm diameter steel ties, with a yield stress equal to 570MPa, spaced at 150mm intervals connecting this panel and the structural members of the surrounding frame as is shown in figure 11. These ties were anchored at the concrete columns and beam, before casting the unreinforced concrete infill, by drilling holes and using a special resin to ensure bonding. The code name for this specimen is “UC-IP 2”.
- Bare jacket2. This specimen (figure 4) was formed by removing the fracture unreinforced concrete panel of specimen “UC-IP 2” and testing it again to record its behaviour before using it again to form the last specimen. The code name for this specimen is “Bare jacket2”.
- Reinforced Concrete-Infill Panel 3. This specimen (figure 5) was formed by filling the bay of specimen Bare Jacket2 using the same steel ties. The reinforcement of the concrete panel was a net of steel reinforcing bars of 4.5mm diameter spaced at 85mm intervals at both horizontal and vertical directions. The code name for this specimen is “RC-IP 3”.

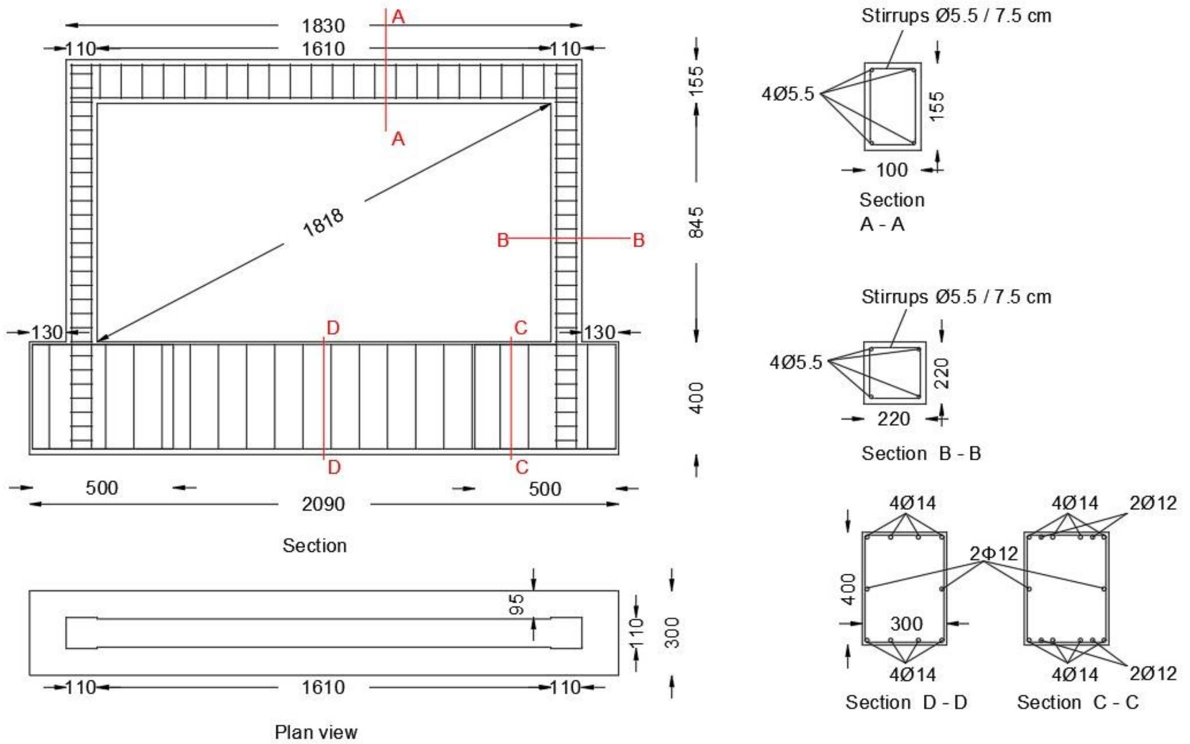

Figure 3 Structural details of the RC frame models hosting UMI tested at Aristotle University

Table 1: Basic mechanical properties of all the building materials

Concrete compressive strength (MPa)	Steel reinforcement yield / ultimum stress (MPa)	Clay brick compressive strength (MPa)	Mortar compressive strength (MPa)	Masonry compressive strength (MPa)	Masonry diagonal tension strength (MPa)	Compressive Strength of mortar applied to the façade (MPa)
23.3	311/425	4.8	1.13	2.5	0.15	17.0

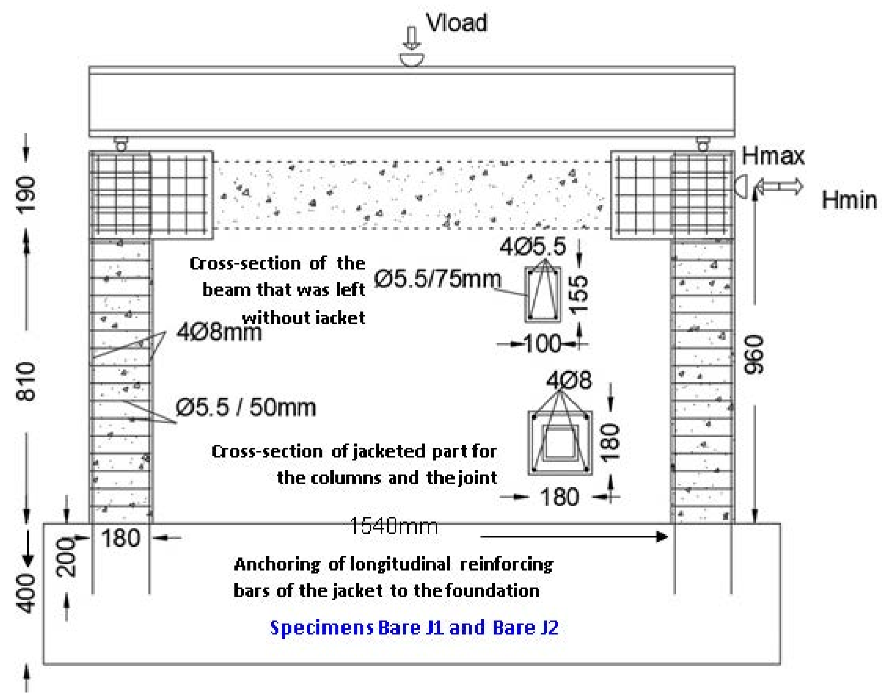

Figure 4: Specimens Bare J1 (Bare Jacket1) and Bare J2 (Bare Jacket2).

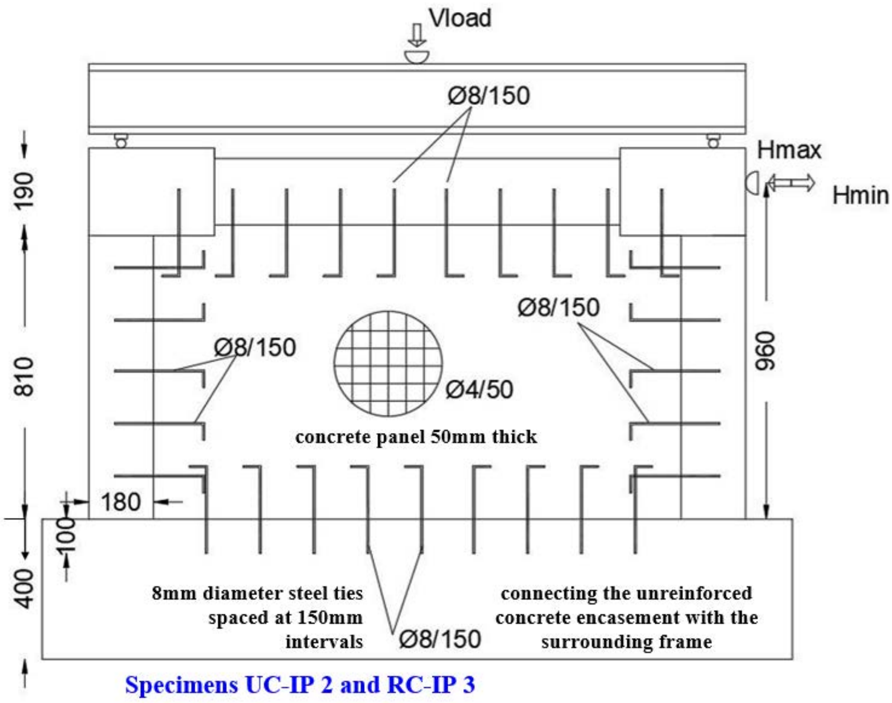

Figure 5: Specimens UC - IP 2 and RC - IP 3.

3 EXPERIMENTAL RESULTS

The obtained cyclic response for the above tested specimens is depicted in figures 6 to 10. All specimens were subjected to imposed horizontal displacement cycles with a frequency of 0.1Hz with continuously increasing amplitude within sequential loading groups, each group having three cycles of constant horizontal displacement amplitude. Prior to applying the imposed horizontal cyclic displacement, a central vertical load was applied at mid-span equally transferred to the top of each column. This vertical load was kept constant throughout each test to an amplitude equal to 45kN per column.

As expected, the construction of the concrete infill panel, with or without metal ties with the surrounding frame considerably increases the initial horizontal stiffness. The jacketing of the frame columns and part of the top beam also increase to a lesser degree this initial horizontal stiffness.

The jacketing of the frame columns and part of the top beam increased almost five times the horizontal load bearing capacity of the initial "Bare" frame. The "Bare" frame both before and after jacketing responded in a ductile manner reaching normalized inter-story drift values in the range of 1.5% without considerable decrease in the maximum horizontal load value. The inclusion of an infill panel increases even further, more than 6 times, the horizontal load bearing capacity of the jacketed frame.

The jacketing of the frame columns and part of the top beam increased more than five times the cumulative plastic energy till the maximum horizontal load was reached when it is compared with the corresponding cumulative plastic energy of the initial "Bare" frame. The inclusion of an infill panel increased even further this cumulative plastic energy. This increase is quite spectacular for specimen named "RC-IP 3" and it is due to the combination of the following contributing mechanisms. The inclusion of the metal ties connecting the infill panel with the surrounding frame alter the interaction between frame and panel in a way that the concentration of compressive stresses at narrow zones near the corners of the frame is avoided and neither the concrete panel nor the frame is crushed prematurely at these regions.

However, because the infill panel is unreinforced it cannot sustain the large forces that develop due to its large stiffness and it fails in a way depicted in figure 13. The capacity of the RC infill panel to large forces is substantially increased by the inclusion of the steel reinforcing net. The developing of cracking within the panel is checked by this reinforcement in a way that it does not lead to a decrease to its bearing capacity even for much larger normalized inter-story drifts values reaching 2.5%. This contributing mechanism mobilizes even further the metal ties connecting the panel with the frame, which represents an additional plastic energy accumulation medium. This combined effect can for the “RC-IP 3” specimen explains the eleven times increase in cumulative plastic energy when compared to the jacketed frame (column 8 of Table 3) whereas the corresponding increase for the “UR-IP 2” specimen is only two-fold. All the above are expected to be valid for prototype structures although the effect of constructing such an infill when applied to a prototype frame bay should be quantified.

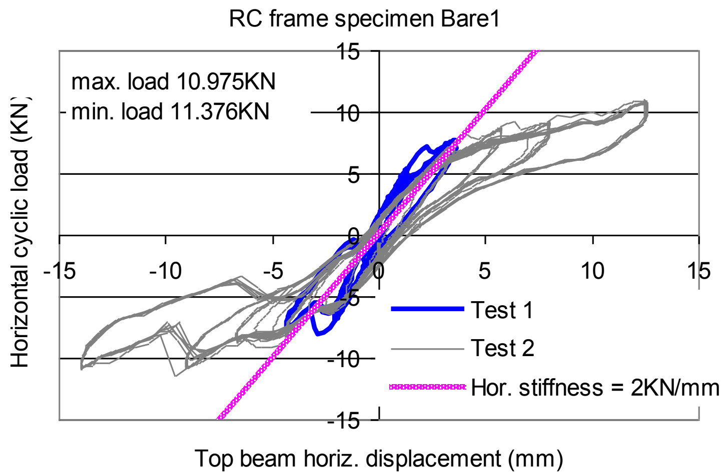

Figure 6 Horizontal displacement versus horizontal load cyclic response of specimen Bare-1

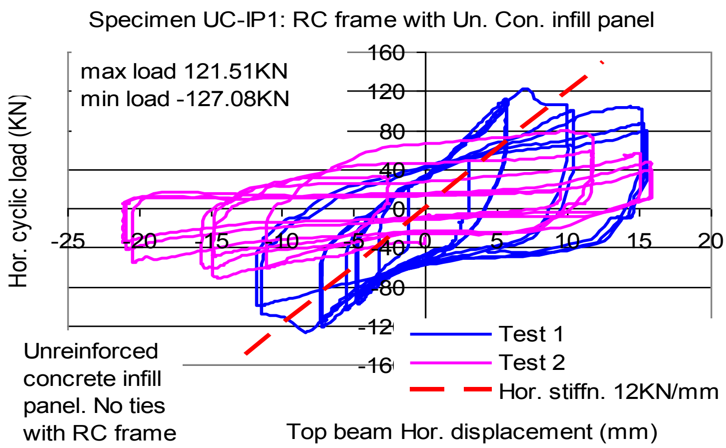

Figure 7 Horizontal displacement versus horizontal load cyclic response of specimen UC-IP 1

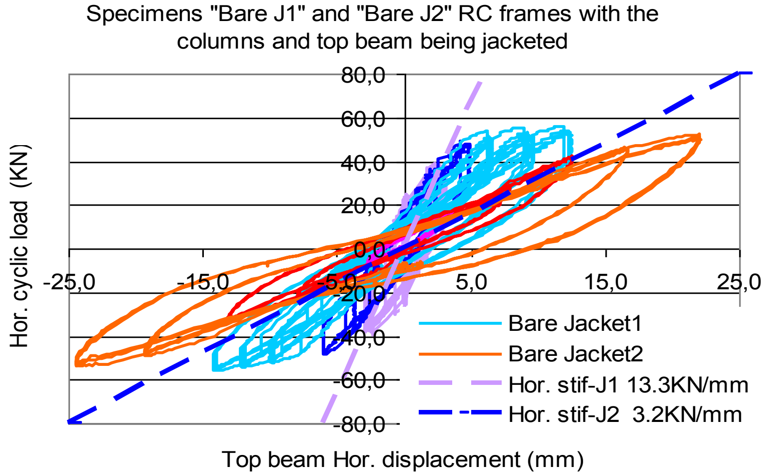

Figure 8 Horizontal displacement versus hor. load for specimens Bare Jacket1 and Bare Jacket2

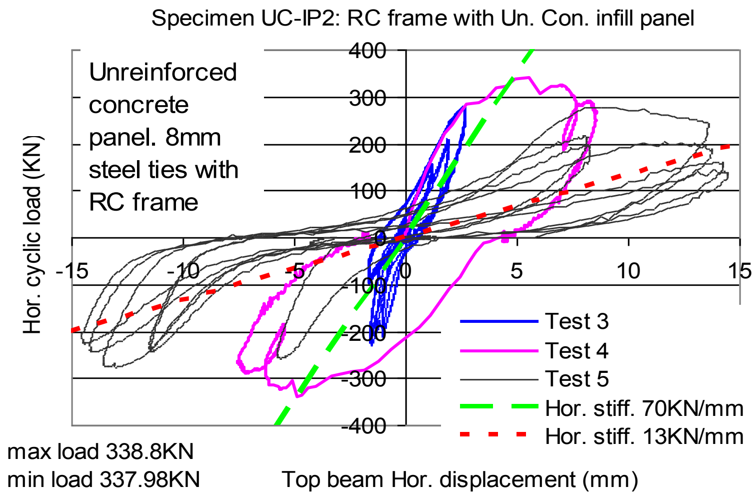

Figure 9 Horizontal displacement versus horizontal load cyclic response of specimen UC-IP 2

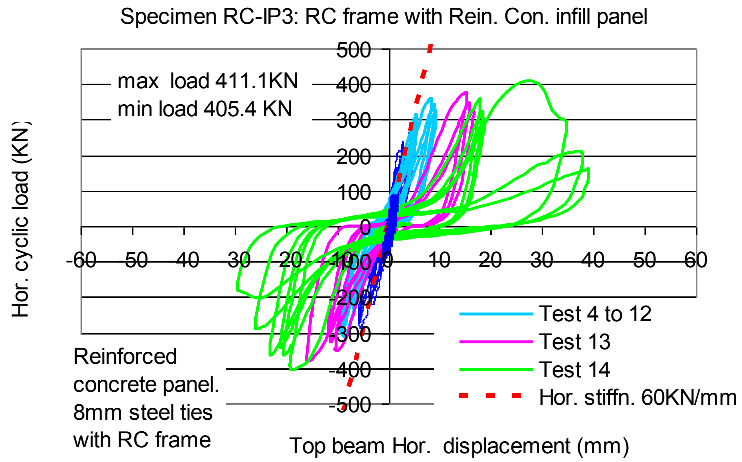

Figure 10 Horizontal displacement versus horizontal load cyclic response of specimen RC-IP 3

4 NUMERICAL PREDICTIONS

In this section summary results of a numerical simulation studying the behaviour of the experimentally studied in section 3 in-filled RC frames. In this numerical simulation the same methodology presented by Manos and Soulis 2012a is followed [1, 6]. The following important mechanisms were included in this simulation:

a) The possibility of either the columns or the beam of the RC frame forming plastic hinges at their ends, figure 11. Towards this end the surrounding frame is simulated with linear frame elements based on the relevant cross-sections having at specific locations plastic hinges with non-linear properties (bending moment against rotation) obtained from the cross-sectional reinforcing details and the mechanical properties of the concrete and the longitudinal reinforcement. The resulting skeleton curve in terms of horizontal load against the corresponding top beam displacement obtained from this numerical simulation is compared with the corresponding experimental cyclic response in figure 12 demonstrating a good degree of approximation. Based on this good agreement the same numerical simulation is followed for the infilled RC frame models presented in what follows.

b) The possibility to simulate numerically the formation of realistic limit states within concrete infill is next presented. For this purpose, diagonal compression tests were conducted at the Laboratory of Experimental Strength of Materials and Structures. Following, numerical models were developed in order to numerically replicate the measured response and calibrate a failure criterion, as depicted in figure 13.

c) The possibility of the concrete infill to be either separated or connected to the surrounding frame at the contact surfaces, depending on the absence or presence of steel ties. This is achieved by placing non-linear link elements that can transfer axial and shear forces between the shell finite elements representing the infill panel and the linear finite elements simulating the surrounding frame. When there are no steel ties these link elements cannot sustain any tension, whereas when steel ties are present these link elements are provided with non-linear properties depending on their diameter (8mm) and yield stress (500Mpa).

The previously described numerical methodology was first applied for specimen UC-IP 1. The numerical response in terms of the envelope curve of horizontal load against the horizontal displacement of the top beam is depicted in figure 14, being compared with the corresponding experimental measurements. The numerical peak value is equal to 350kN and compares reasonably well with the peak measured value of 338kN. The observed sudden drop of bearing capacity for specimen UC-IP 2 is not predicted by this numerical simulation.

Finally, the previously described numerical methodology was applied for specimen RC-IP 3. The numerical response in terms of the envelope curve of horizontal load against the horizontal displacement of the top beam is depicted in figure 15 and compared with the corresponding experimental measurements. The numerical peak value is equal to 450kN and compares reasonably well with the peak measured value of 408kN. The observed sudden drop of bearing capacity for specimen UC-IP 2 is not predicted by this numerical simulation.

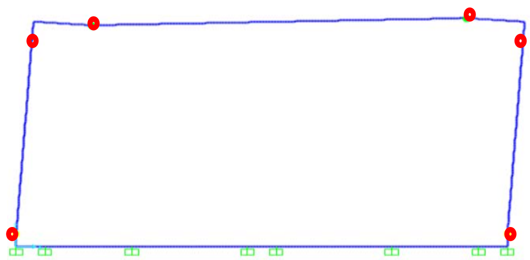

Figure 11 Numerical simulation of the surrounding RC frame with the location of the plastic hinges

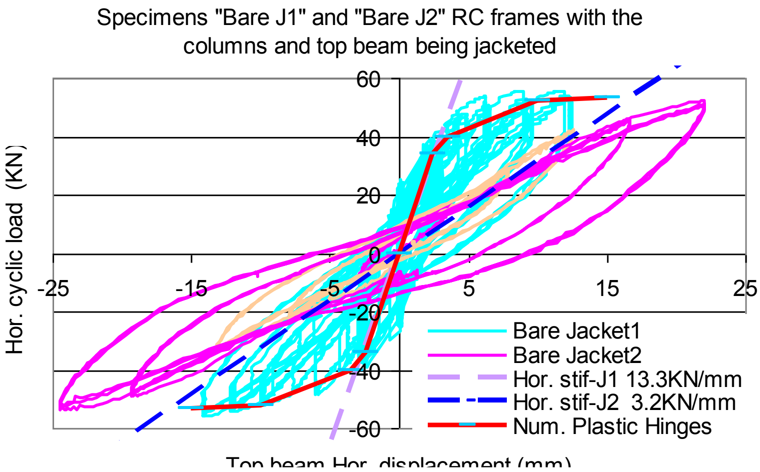

Figure 12: Horizontal ket1, 2

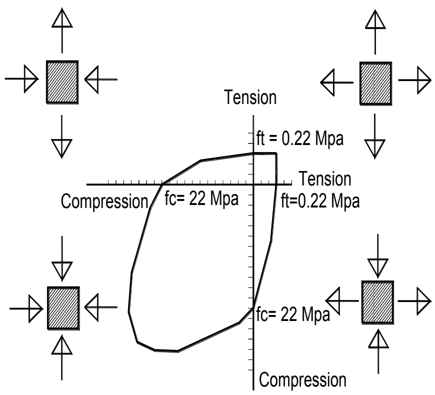

Figure 13 Assumed combined compression – tension failure envelops included in the numerical simulation of the square concrete panel as well as of the infilled panels.

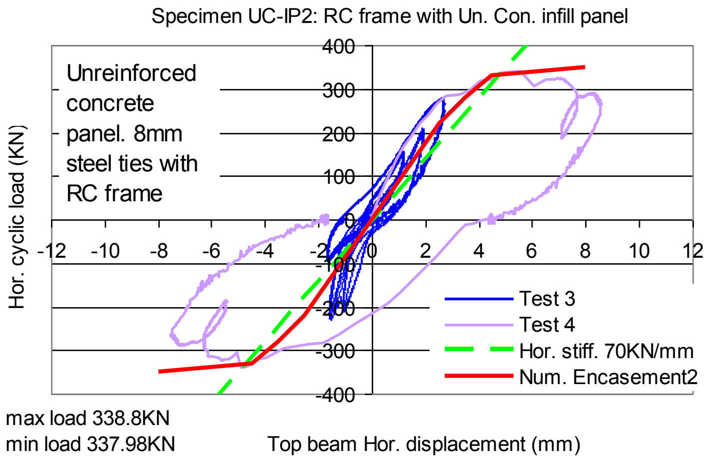

Figure 14: Horizontal displacement versus horizontal load cyclic response of specimen UC-IP 2. Comparison between observed and numerically predicted behaviour

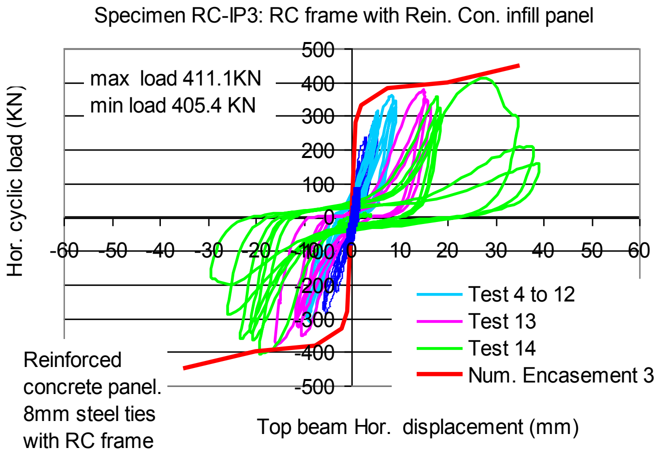

Figure 15: Horizontal displacement versus horizontal load cyclic response of specimen RC-IP 3. Comparison between observed and numerically predicted behaviour

CONCLUSIONS

Relatively old multi-story frame RC structures in earthquake-active regions are in need of seismic retrofitting. The most vulnerable parts are the columns of the ground story that are left without unreinforced masonry infills serving as parking space. A counter measure towards this end is adding RC infill panels to the bays of such ground floor frame bays or similar bays located at higher stories where such a construction of RC infill can be compatible with the basic functions of the building. In these upper stories many bays are hosting unreinforced masonry infills, which many times are prematurely damaged and collapse during strong earthquake ground motions. The current investigation highlighted the benefits of constructing such RC infill panels together with RC jacketing of the columns and beams of the surrounding frames. These are the following.

- The construction of RC infills, connected with metal ties with the surrounding jacketed frame increase considerably the in-plane initial horizontal stiffness and the in-plane horizontal force bearing capacity of the resulting RC frame with RC infill panels when compared with those of the initial "bare" frame.
- The use of metal ties connecting the RC infill panels with the surrounding frame ensures that the transfer of forces does not lead to stress concentration regions that can lead to localized undesired damage to either the RC infill panels of the surrounding frame. Finally, the proper reinforcing of the RC infill panel can prohibit the development of premature damage and sudden loss of bearing capacity.

From the preceding comparison between the observed and the numerically predicted response of the infilled specimens the following conclusions can be drawn.

- The applied numerical simulation could predict quite well the in-plane bearing capacity of the RC frames infilled with concrete panels and the corresponding limit states. Therefore, it could be followed for design purposes.
- The applied numerical simulation is based on limit state assumptions of either the concrete infill panel itself or the steel ties that were reasonably well defined based on specific experimental results.
- The applied numerical simulation could not predict the sudden loss of bearing capacity after the development of the assumed limit states.
- Despite its limitation, the proposed numerical simulation can serve as a useful design tool in quantifying the effect of such infills when applied to prototype frame bays.

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