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EXPERIMENTAL ASSESSMENT AND RETROFIT OF FULL-SCALE MODELS OF EXISTING RC FRAMES

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ABSTRACT
PSD tests on two full-scale models of existing non-seismic resisting RC frame structures are described. The testing program covered several aspects, namely assessment of seismic performance of existing frames without and with infill panels, retrofitting of the bare frame using Selective Retrofitting techniques, strengthening of the infill panels using shotcrete and retrofitting of the frame using K-bracing with shear-link dissipators. The main results from the tests are summarized and discussed and the conclusions are drawn. The tests on the bare frame have shown how vulnerable are existing structures constructed in the 60's and the beneficial effects of infill panels were confirmed from the tests on the infilled frame. Important improvements, in terms of seismic performance, were achieved by the retrofitting of the frames. However, it was also confirmed that strengthening of the existing infill panels in poorly detailed frames may lead to dangerous 'local' failures, such as the shear out of the external columns.

Keywords: Existing structures, Assessment, Retrofitting, Infilled frames, Seismic tests.

INTRODUCTION
In the framework of the ICONS Topic 2 - Assessment, Strengthening and Repair - research programme [1], [2] two full-scale reinforced concrete frames were tested pseudodynamically at the ELSA laboratory. The frames, representative of the design and construction practice of 40~50 years ago in most European Mediterranean countries, have been tested in order to assess the vulnerability of bare and infilled structures and to investigate various retrofitting solutions/techniques, namely: a selective retrofitting scheme, which provides either strength, or ductility, or stiffness; shotcrete of infill walls; and, k-bracing system with dissipative devices. It is believed that the results from these tests can be considered as reference data for the improvement of the pioneering part 1.4 of EC8 [3] on repair and strengthening.

Design of the test models was performed at LNEC, Lisbon by Carvalho et al. [4] and two similar frames were constructed allowing for a vast testing campaign, which included assessment of bare and infilled frames [9], [10], repair and retrofit. The retrofitting solution for the bare frame was designed by Elnashai and Pinho [5], [6] and is based on a rational

intervention, which balances strength, stiffness and ductility according to the requirements for increased seismic performance. The retrofitting solution with K-bracing and shear-link dissipator was designed by Bouwkamp [11] and comprised the introduction (replacing a wall panel) of ductile steel eccentrically braced assemblies with vertical shear links in an internal bay of the frame. Additionally, an infilled frame with strengthened panels was also tested. Strengthening of the masonry panels was made through a concrete layer with an embedded steel mesh using shotcrete (see Pinto et al. [10]).

The full-scale models were subjected to input motions with increasing intensities up to failure. This paper describes the test structures, testing set-up and loading and presents the main results from the pseudo-dynamic (PSD) tests carried out at the ELSA laboratory.

DESIGN AND CONSTRUCTION OF THE RC FRAMES

Figure 1 shows the general layout of the structure. It is a reinforced concrete 4-storey frame with three bays; two of 5 *m* span and one of 2.5 *m* span. The inter-storey height is 2.7 *m* and a 0.15 *m* thick slab of 2 *m* on each side is cast together with the beams. Equal beams (geometry and reinforcement) were considered at all floors and the columns. All but the wider interior one, have equal geometric characteristics along the height of the structure. The stocky column has a rectangular cross-section with dimensions 0.60 *m* × 0.25 *m* on the first and second storeys and 0.50 *m* × 0.25 *m* on the third and fourth storeys. All beams in the direction of loading are 0.250 *m* wide and 0.50 *m* deep, while transverse beams are 0.20 *mm* wide and 0.50 *mm* deep.

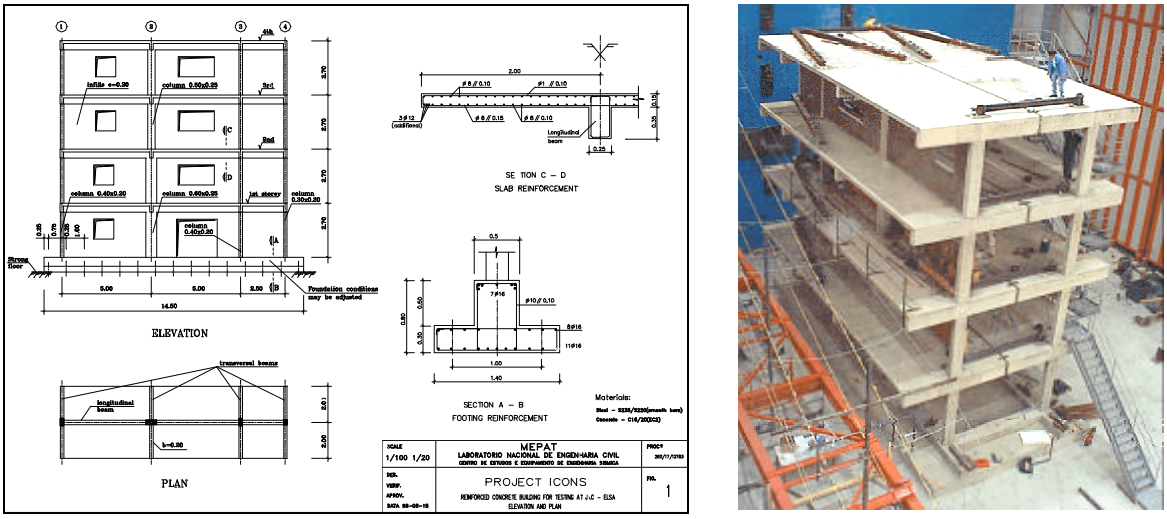

Figure 1 Plan and elevation views for the frame plus infills (left); Models in the ELSA laboratory (right)

The column reinforcement splice and stirrup details (90° bends and ‘overlapping’) should be noted in particular, as they are representative of the lack of confinement common in non-ductile reinforced concrete structures of ~40 years ago. It is also noted that smooth round bars, which were commonly used some years ago, constitute the longitudinal reinforcing steel. The longitudinal reinforcement of all (four) columns has a lap splice (70 *cm*) at the base of the 1st storey and another at the base of the 3rd storey. The materials considered at the design phase [4] were a low strength concrete, class C16/20 (Eurocode 2) and smooth reinforcing steel (round smooth bars) of class FeB22k (Italian standards). Tests on samples of the materials used in the construction (steel reinforcement and concrete) of the structure have been carried out and the following results were obtained: a) for the concrete - mean strength

values for all casting phases, f_{cm}=15 *MPa*, b) for the steel (mean values) - yielding stress, f_{sy}=350 *MPa*, ultimate strength, f_{su}=453 *MPa*, Ultimate deformation, e_{su}=24% ('nominal' values: f_{sy}=250 *MPa*, f_{su}=365 *MPa*). The vertical loads were defined in order to simulate the dead load other than the self-weight of the frame, and considering that parallel frames have a distance of 5.0 *m* (note that the frame model includes a 4.0 *m* wide slab, which requires additional vertical load accounting for such a slab portion missing). Vertical distributed loads on beams and concentrated loads on the column nodes were considered in order to simulate the dead load other than the self-weight of the frame. These correspond to the following vertical loads: Weight of slab: 25 x 0.15 =3.75 kN/m^2, Weight of finishings: 0.75 kN/m^2, Weight of transverse beams: 2.5 *kN/m*, Weight of masonry infills: 1.1 kN/m^2 of wall area (it is considered that these walls exist both over longitudinal and transverse beams), Live load: 1.0 kN/m^2 (quasi-permanent value). A pre-test seismic assessment of the frame was made by Griffith [7].

The input seismic motions were defined in order to be representative of a moderate-high European seismic hazard scenario [8]. Hazard consistent acceleration time series (15 seconds duration) were generated yielding a set of twelve uniform hazard response spectra for increasing return periods. Acceleration time histories, for 475, 975 and 2000 years return periods, were used in the tests (PGA of 218, 288 and 373 cm/s2, respectively).

TESTS ON THE BARE FRAME SPECIMEN (BF)

Two similar frames were constructed out-side of the ELSA laboratory and subsequently transported inside it and positioned in front of the reaction wall (see picture from the mounting/instrumentation phase in Figure 1). One of the frames, the bare frame (BF), was subjected to one PSD earthquake test corresponding to 475 years return period (475-yrp) and subsequently to a second PSD test carried out with a 975-yrp input motion. Results from these tests are given in Figure 2, in terms of storey shear versus storey drift (for two storey levels). It is noted that the 975-yrp test was stopped at 7 seconds of the 15 seconds accelerogram because imminent collapse was attained at the 3rd storey. The significant change in stiffness and strength from the 2nd to the 3rd storey, coupled with the inadequate lap-splicing and shear reinforcement, induced the development of a three-hinges column mechanism in the stocky column, dictating collapse at the 3rd storey. Maximum inter-storey drift profiles are shown in Figure 3 and maximum values of drift and base-shear, for the two PSD tests, is included in Table 1.

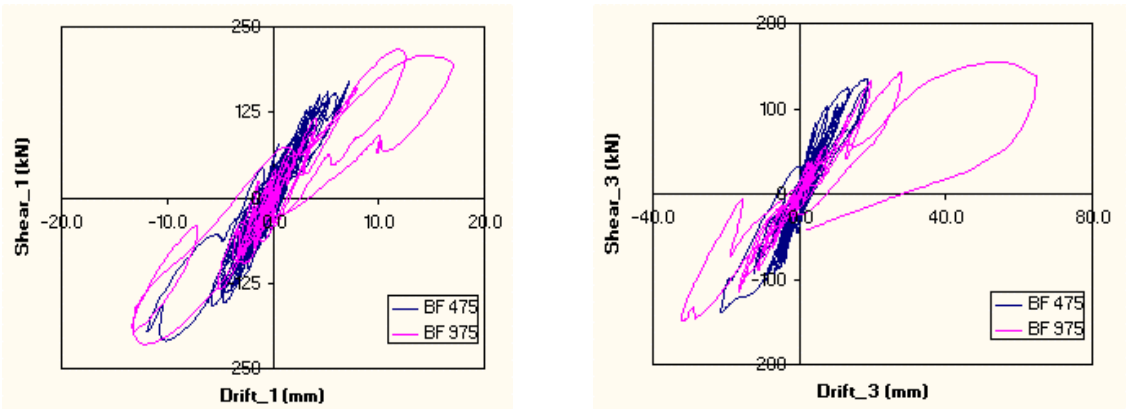

Figure 2 Storey shear-drift diagrams for the Bare Frame tests: BF475 and BF975.

SELECTIVE RETROFITTED SPECIMEN (SR)

Following the two earthquake tests on the bare frame, the damaged parts of the structure were repaired (stocky column of the 3rd storey). The 'spalled' concrete was removed and the cracks were injected with epoxy, the surfaces were cleaned and the selective retrofitting scheme, proposed by the research group at the Imperial College of London [5], [6] was applied.

The selective retrofitting solution involved two types of interventions in the wide internal column (stocky column). A *strength-only* intervention was implemented in the wide column at the 3rd and 4th storeys to reduce the large flexural capacity differential verified at level 3. A *ductility-only* intervention was accomplished at the first three storeys in the wide column, where large inelastic deformation demand is expected. This intervention was achieved by the addition of external confining steel plates at the critical zones (at the base and at the top of the member). Furthermore, to minimize the risk of shear failure, additional plates were also added at mid-height of the columns.

The initial testing program for the selective retrofitting (SR) frame was similar to the bare frame (BF) program. However, in view of the results obtained for the SR975 test, which led to rather small demands and damage, it was decided to perform an additional test with higher intensity. This test was expected to inflict more significant damage on the structure but it was also necessary to guarantee structural integrity for the next strengthening solution foreseen for this specimen (K-bracing with shear-link). A 2000 yrp earthquake was adopted for this high-level test. Maximum values of drift and base-shear for each test are collected in Table 1 and Figure 3 shows the maximum drift profiles for the three earthquake tests.

BARE FRAMES – ORIGINAL (BF) VS SELECTIVE RETROFITTED (SR)

A few results from the BF and SR tests were presented in the previous sections without direct comparison between them. It is important to quantify both the BF and SR demands and ultimate capacities, but it is also useful to highlight the effectiveness of the retrofit provided to the frame.

The tests performed on the bare frame show a concentration of inter-storey drift demand, and consequently damage in the 3rd storey (see Figure 3 (left)). The mechanism developed in the structure was due to its vertical irregularity in terms of stiffness and strength resulting from the change in cross-section size and reinforcement of the strong central column. The selective retrofitting addressed and solved the irregularity problem of the structure. In fact, the maximum storey drift profiles plotted in Figure 3 (right) confirm the effectiveness of the retrofitting. In spite of the fact that no substantial differences exist between the BF and SR drift demands for the 475-yrp, the 3rd storey large drift demand of the bare frame for the 975-yrp test vanishes for the retrofitted frame and the comparable top displacement demands result now from much more uniform storey drift in the retrofitted frame. Furthermore, the retrofitted frame was able to withstand an input motion intensity 1.8 times the nominal one, in terms of PGA, without collapse and with reparable damages, while the bare frame collapsed for an input motion 1.3 times the nominal intensity. Another aspect that should be noted is the uniformity of the inter-storey drift profiles for the different input motion intensities. In fact, as shown in Figure 3 (right) drift demands increase proportionally to the input-motion intensity maintaining the drift pattern. This confirms that the selective retrofitting prevented the storey mechanism. In addition, the drift demands are rather uniformly shared between the three lower storeys.

The results from the SR tests have shown rather improved seismic performance. In fact, the

SR frame was subjected to the same input motions as the BF with limited structural damage and was able to withstand an input motion with intensity 1.8 times the nominal one (corresponding to a return period of 2000 years) maintaining its load carrying capacity with repairable damages. The retrofitting operation addressed and solved the irregularity problem and the confining steel plates definitively increased the limited deformation capacity of the central stocky column. In fact, drift demands were rather uniformly distributed in the first three storeys for the three earthquake tests and reached values much higher than the values of the bare frame tests. Inter-storey drifts of 2.8, 3.0, 1.6 and 0.9% were reached at the first, second, third and fourth storeys respectively without loss of load carrying capacity. It is noted that 2.8% drift in the first storey is twice the ultimate drift identified from the original (non-retrofitted) frame. Therefore, it is concluded that the deformation capacity of the first storey is, at least, the double of the original structure.

There are other aspects that should be highlighted from the test campaigns on the original (BF) and Retrofitted (SR) frames, namely: 1) As expected, the strong-beam weak-column deformation/dissipation mechanism (storey mechanism) is the only one activated for all tests. However, slightly higher demands in the beams were apparent for the retrofitted frame; 2) There is a strong concentration of the inelastic demands at the member ends, leading to equivalent plastic hinge lengths much lower than the empirical values proposed in the literature (calculated plastic hinge lengths are 40% of the empirical values). This a direct consequence of the poor bond characteristics of the smooth round rebars, which leads to extremely high slippage with concentration of the deformation at the member (beam and column) extremities; 3) The values calculated for the slab participation are also much lower than the values proposed in the design codes and also lower than the values estimated from tests on building structures with improved-bond steel (approximately 45% lower). This is also a direct consequence of the poor bond characteristics of the smooth round steel reinforcement also used in the slabs; 4) The test results confirmed that lap-splicing at the base of the columns, particularly in existing structures with smooth round rebars with extremity hooks and poor detailing and amount of shear/confinement reinforcement, develop premature shear cracks at the bar termination zones for inter-storey drifts of approximately 0.4%. These shear cracks dictate dangerous shear failure of the columns for inter-storey drifts in the range of 1.3~1.8%.

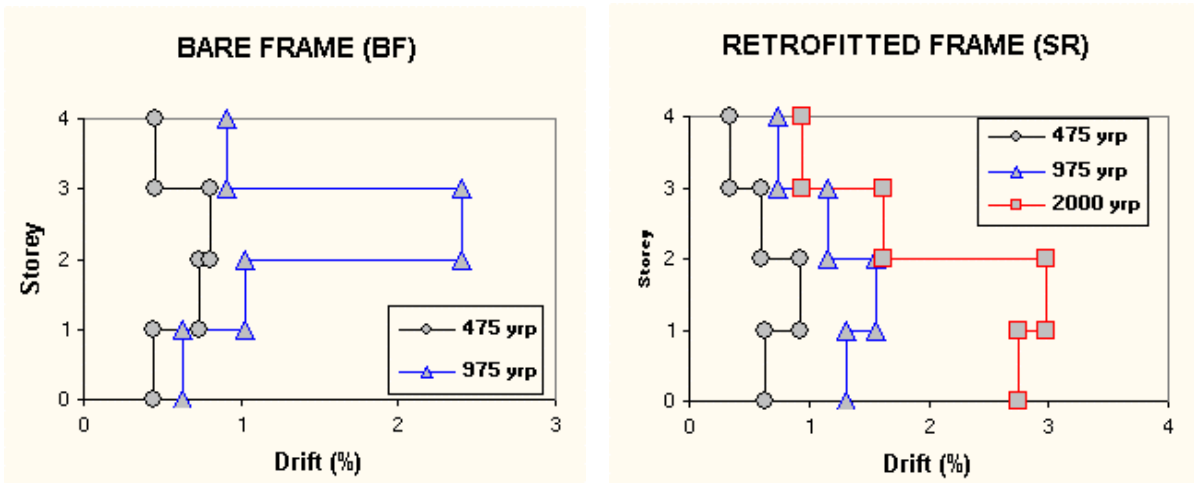

Figure 3 Maximum inter-storey drift profiles - BF (left) and SR (right) tests

MASONRY INFILLED FRAME (IN)

An identical RC frame with masonry infill panels was also constructed and subjected to a series of earthquake tests. The reinforced concrete frame was constructed with the same detailing and materials as for the bare frame. Figure 1 shows the general layout of the structure including infill panels and the type and location of the openings. The 150 *mm* thick infill-walls (non-load bearing) were constructed after the reinforced concrete frame and inside the ELSA laboratory in order to avoid damage of the infills due to even small inter-storey deformation during the structure transport into the laboratory. Representative materials and construction techniques were used, namely: Italian hollow bricks horizontally perforated, with the following dimensions: 0.12 *m* thick, 0.245 *m* base-length and 0.245 *m* height. According to the classification of masonry units presented in Eurocode 6 [12], the hollow blocks are included in Group 3. The infill walls are constructed with the block units bedded on the 0.120 × 0.245 face and the hollows in the horizontal direction (0.12 *m* thick). The mortar used in the joints and plaster was manually prepared. The mortar joints are approximately 1.5 *cm* thick for both vertical and horizontal joints and a 1.5 *cm* thick plaster is applied on both sides of the walls. The same mortar proportioning was used for bed joints and plaster (1:4.5 (hydraulic binder : sand)).

The infilled frame specimen was subjected to three consecutive PSD earthquake tests corresponding to 475, 9755 and 2000 years return periods. During the 2000-yrp PSD test the masonry infills at the 1st storey collapsed and the test was stopped at ~5 seconds. Results from these tests are given in Figure 4 in terms of storey shear-drift and maximum inter-storey drift profiles. For the 475-yrp test, overall, the infilled frame structure behaved very well. Most of the energy dissipation in the brickwork probably occurred along frame/wall interfaces. The 975-yrp earthquake caused a significant amount of damage to the block infill in the bottom storey of the concrete frame with some minor damage to the concrete beam-column joints and several columns at this level. Smaller amounts of damage in similar locations were noted in the 2nd storey. No significant damage was observable in the upper two stories. It was recognised that the infill frame had become, by now, a soft-storey infill frame structure. Nevertheless, it was subjected to the 2000 yrp earthquake signal in order to study how gradually the lateral strength dropped off with increasing drift. The storey shear versus drift hysteresis loops clearly show that the load deflection characteristics approach those of the bare frame as the drifts increase to values in excess of 1% (see Figure 4).

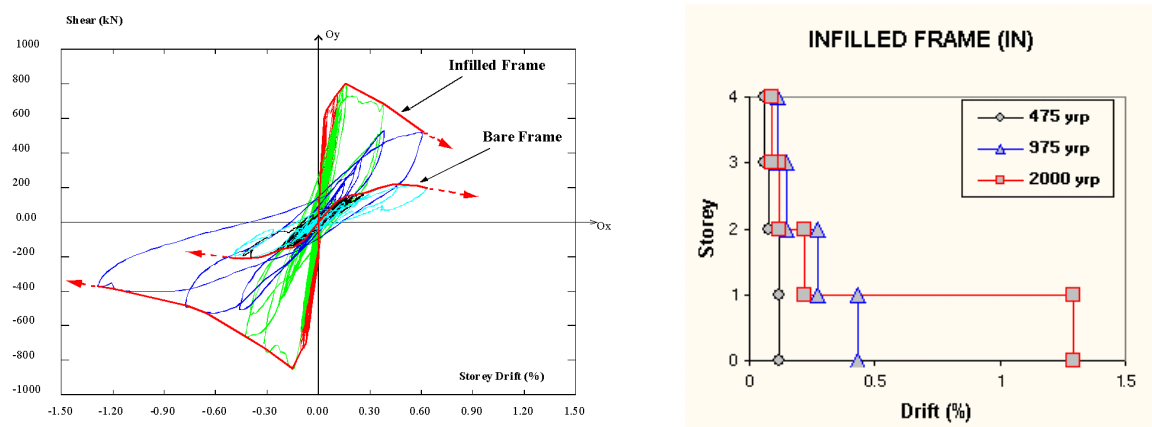

Figure 4 Results from the tests on the infilled frame (IN): a) 1st storey shear-drift diagrams for all tests and envelope curves with comparison to the bare frame (BF) diagrams, b) Maximum drift profiles for the three earthquake tests

SHOTCRETE INFILLED FRAME (SC)

After replacement of the damaged infill panels, shotcrete was applied to part of the walls and new earthquake tests were carried out. On the basis of the analyses of the test results from the previous tests and from visual inspection it was decided to replace only the 1st storey infill walls and to apply the retrofitting (shotcrete) only on the external short-bay, at all 4 storeys. The new infill panels were rebuilt with the same geometry and materials as the original walls. The retrofitting solution applied to the infill walls consists of a concrete layer with an embedded reinforced steel mesh, which are deemed to improve the post-peak behaviour of the walls. The shotcrete applied to the shorter external panels (one side/face only) (Figure 5) at all storey levels consists of a 26 *mm* thick concrete layer, with an embedded welded steel mesh (S500, ribbed, grade 500 *MPa*), with 5 *mm* wire diameter and 10x10 *cm* spacing. No specific connection (e.g.: dowels) was provided between the shotcrete layer and the existing surrounding RC frame. A light connection (clamps) between the shotcrete layer and the masonry walls was provided, in nine points. It is noted that these clamps were not specifically designed for that purpose, but they were used to keep in place the steel reinforcing mesh for the shotcrete works.

The infilled shotcrete frame was subjected to the same earthquakes and the results are given in Figure 5 in terms of maximum drift profiles and storey shear-drift diagrams. It is apparent that equivalent strengths develop for the IN and SC specimens and the deformation capacity of the SC frame is moderately improved. It is also apparent that the 2nd storey drift demands are much higher for the SC frame tests. This is justified by the fact that the infill panels at this storey have not been replaced after the previous tests. Two main aspects should be highlighted from these tests, namely: a) the beneficial effects of the shotcrete on the behaviour of the infill panels, which avoids premature cracking and crushing of the infill walls; b) the shear-off of the external columns in their upper part leading to local collapse (*warning - dangerous effect*), which results from a combination of the shear forces developed in the infill panel and the overturning moment effects (up-lift of the upper beam inhibits transmission of shear forces between the panel and the beam, leading to direct shear-off of the top of the column). This is a point that deserves special comments because it is common practice to apply these strengthening techniques in particular for repair and strengthening of infill structures after earthquakes. Strengthening of infill walls in frame structures should be made with appropriate dowelling to the adjacent beams in order to transfer the shear forces gradually to the surrounding frame.

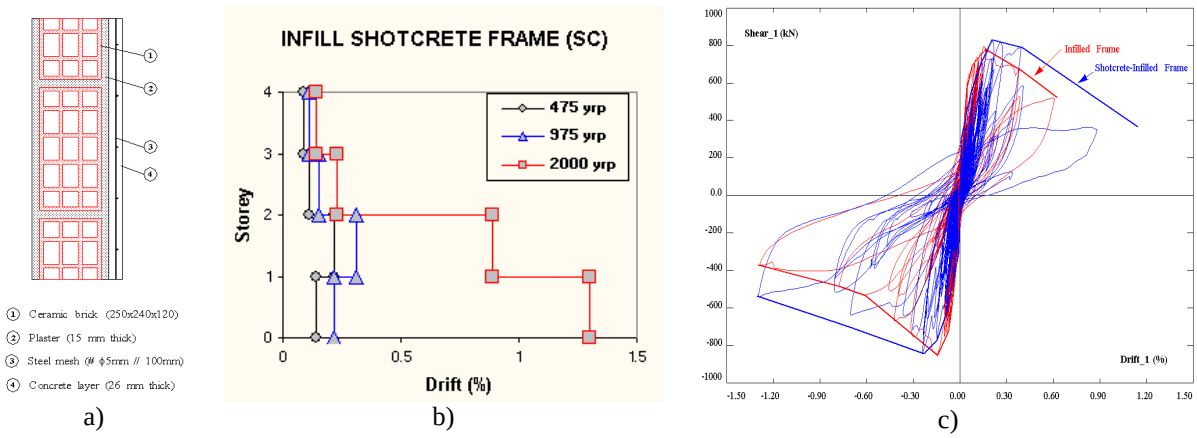

Figure 5 Shotcrete of the infill walls: a) Constructive details (layout); b) Drift profiles; c) 1st storey shear-drift diagrams for the infilled and infilled-shotcrete tests and corresponding envelope curves

TABLE 1
TEST RESULTS: MAXIMUM VALUES OF DRIFT AND BASE-SHEAR FOR ALL TESTS

Test[a)]	**Inter-Storey Drift (%)**				**Global Drift (%)**	**Base Shear (*kN*)**
	Storey 1	Storey 2	Storey 3	Storey 4		
BF 475	0.44	0.74	0.80	0.46	0.56	209.0
BF 975[b)]	0.63	1.03	2.41	0.91	1.08	216.7
SR 475	0.63	0.92	0.60	0.34	0.59	212.2
SR 975	1.31	1.56	1.16	0.74	1.08	261.1
SR 2000 [c)]	2.75	2.98	1.62	0.94	2.03	285.9
IN 475	0.12	0.12	0.08	0.06	0.09	754.0
IN 975	0.43	0.27	0.15	0.11	0.21	846.5
IN 2000	1.29	0.22	0.12	0.09	0.38	543.2
SC 475	0.14	0.22	0.11	0.09	0.14	703.9
SC 975	0.22	0.31	0.15	0.11	0.19	820.1
SC 2000	1.30	0.89	0.23	0.14	0.61	838.6

a) Duration of the input motions is 15 seconds, for the earthquake PSD tests. b) Test performed up to 7.5 seconds because imminent collapse was attained. c) Test performed up to 5.0 seconds in order to reach full collapse of the 1st storey infill panels.

K-BRACING WITH SHEAR-LINK – DSEBS

A ductile steel eccentrically braced system (DSEBS) was used to retrofit one of the storeys of the infilled frame and the structure was subjected to displacement controlled cyclic tests with increasing amplitude [11]. The DSEBS retrofit design concept is illustrated by considering the retrofitting process of the second storey of the masonry-brick infilled test frame shown in Figure 1. The effectiveness of the design assumptions was assessed subsequently by studying the response of the retrofitted second floor frame/wall system under increasing cyclic displacement-controlled loads (holding the lateral displacement of the 2nd floor constant throughout the test and imposing identical displacements at the 3rd and upper floor levels). In order to maintain structurally a basically symmetric layout, it was decided to retrofit the second story frame by replacing the infilled wall of the 5.00 *m* wide middle bay by an eccentrically braced steel frame assembly (see Figure 6). In order to allow a fundamental assessment of the proposed retrofitting procedure and to eliminate secondary effects, the outer 2.50 *m* and 5.00 *m* wide bays were filled completely with hollow-brick masonry of the type used in the other test phases of this project. The DSEBS is formed by an assembly of steel beams, diagonal braces and a centrally located ductile vertical shear link which is designed to replace the infilled masonry in a single bay of a concrete infilled frame. The assemblies are typically placed in one or more vertical arrays over the height of the building. Conceptually, the design aims at developing a retrofit system, which has a total storey-shear resistance similar to the lateral resistance of the original infilled system but with a substantially increased ductile energy absorbing capacity.

The results of tests, in which the retrofitted storey was subjected to cyclic displacement-controlled deflections of increasing magnitude, showed the soundness of the concept. An excellent agreement between the predicted response and experimental results could be observed. Also, the technology used to anchor a steel retrofit assemblage to the surrounding concrete beams and columns of a retrofitted bay, was proven to be most effective. In fact, as

shown in Figure 6, the post-peak behaviour of the frame was substantially improved – the drift reached approximately 1% without important loss of strength and the DSEBS dissipates approximately 45% of the total dissipated energy.

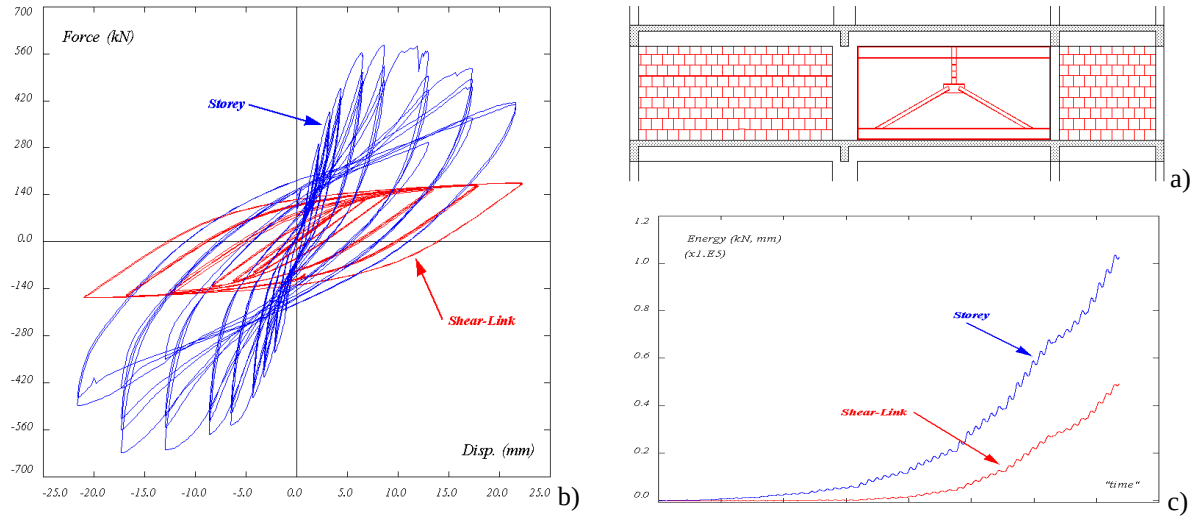

Figure 6 DSEBS system: a) Test assembly b) Storey shear – displacement and shear-link force-displacement results, c) Energy dissipation for retrofitted frame and shear link

FINAL REMARKS

A series of pseudo-dynamic tests on two full-scale models of a 4-storey R/C frame representative of existing structures designed without specific seismic resisting characteristics (common practice of 40~50 years ago) were carried out at the ELSA Laboratory. Five testing campaigns were performed aiming at: 1) vulnerability assessment of a bare frame (BF); 2) assessment of a selective retrofit solution and system (SR); 3) earthquake assessment of an identical frame with infill masonry walls (IN); 4) assessment of the shotcrete retrofitting of the infill panels (SC); 5) assessment of a retrofitting system based on k-bracing with shear-link dissipators (DSEBS).

Analysis of the test results and comparison between the behaviour and earthquake vulnerability and performance of the different structures were briefly reviewed. It is however important to underline a few aspects from these test campaigns, namely: **1)** The high vulnerability of the original bare frame (BF) was confirmed. The structure reached imminent collapse at the 3rd storey (2.4% inter-storey drift) for an input intensity slightly higher than the nominal one (1.3 times, in terms of PGA and corresponding to a 975 years return period input motion); **2)** The SR frame has shown rather improved seismic performance. In fact, it was subjected to the same input motions as the BF with limited structural damage and was able to withstand an input motion with intensity 1.8 times the nominal one (corresponding to a return period of 2000 years) maintaining its load carrying capacity with repairable damages. The retrofitting operation addressed and solved the irregularity problem and the confining steel plates definitively increased the limited deformation capacity of the central stocky column. **3)** The infilled frame showed completely different behaviour compared to the bare frame. Infills protect the RC structure but also prompt storey mechanisms and cause shear-out of the external columns in the joint region; **4)** Shotcrete of infill walls in existing structures improves the behaviour of the walls but can cause premature loss of the vertical load-carrying capacity of the structures by the shear-out of the external columns. Shotcrete can be beneficial only if appropriate doweling to the adjacent beams/girders is provided; **5)** Retrofitting

solutions based on k-bracing and dissipative devices, such as a shear-link can substantially improve storey behaviour and increase energy dissipation capacity. Detailed results from the tests and corresponding analysis can be found elsewhere [9], [10], [11].

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