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EFFECTIVENESS OF CFRP-JACKETS AND RC-JACKETS IN POST-EARTHQUAKE AND PRE-EARTHQUAKE RETROFITTING OF BEAM-COLUMN SUBASSEMBLAGES

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SUMMARY

This paper presents the findings of an experimental study to evaluate methods of retrofit which addresses particular weaknesses that are often found in reinforced concrete structures, especially older structures, namely the lack of the required flexural and shear reinforcement within the columns and the lack of the required shear reinforcement within the joints. Thus, the use of a reinforced concrete jacket and a high-strength fiber jacket for the post-earthquake and pre-earthquake retrofitting case of columns and beam-column joints was investigated experimentally. In the paper the effectiveness of the two jacket styles was also compared.

INTRODUCTION

Damage caused by earthquakes through the years, indicated that some reinforced concrete buildings designed and constructed in the 1960s and 1970s were found to have serious structural deficiencies. These deficiencies are mainly a consequence of a lack of capacity design approach and/or poor detailing of reinforcement. As a result, lateral strength and ductility of these structures were minimal [1]. Wrapping of reinforced concrete members with fiber-reinforced polymer (FRP) sheets including carbon (C), glass (G), or aramid (A) fibers, bonded together in a matrix made of epoxy, vinylester or polyester, has been used extensively throughout the world in numerous retrofit applications in reinforced concrete buildings and are recognized as alternate strengthening systems to conventional methods, such as steel plate bonding and shotcreting [2], [3], [4].

The feasibility and technical effectiveness of the high-strength fiber jacket system and the reinforced concrete jacket system both in a post-earthquake and pre-earthquake retrofitting case of columns and beam-column joints was investigated in the paper. Thus, four identical reinforced concrete exterior beam-column-slab-transverse beam subassemblages (F_1, S_1, O_2 and P_2) were constructed with non-optimal design parameters: flexural strength ratio, joint shear stress, with less column transverse reinforcement than that required by the modern Codes [5] and without joint transverse reinforcement representing the common construction practice of column and beam-column joints of older structures built in the 1960s

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and 1970s. The subassemblages F_1 and O_2 were subjected to cyclic lateral load histories so as to provide the equivalent of severe earthquake damage. The damaged specimens were then strengthened by high-strength fiber jacket and by four-sided reinforced concrete jacket. These jackets were applied in the columns and b/c joint regions of the damaged subassemblages. The subassemblages S_1 and P_2 represent parts of an old frame structure, which was upgraded to resist future strong earthquakes. These two subassemblages were tested only after strengthening by high-strength fiber-jacket and by four-sided reinforced concrete-jacket. These jackets were also applied in the columns and b/c joint regions of the subassemblages S_1 and P_2. The four repaired and strengthened subassemblages were subjected to cyclic lateral load history so as to provide the equivalent of severe earthquake damage.

A direct comparison of the load deflection envelopes of the original and the retrofitted subassemblages was provided in the paper. The effectiveness of the two jacket styles was also compared.

DESCRIPTION OF THE SPECIMENS

Original Test Specimens F_1, S_1, O_2 and P_2

Four identical test specimens F_1, S_1, O_2 and P_2 were constructed using normal weight concrete and deformed reinforcement. Both specimens were typical of existing older structures built in the 1960s and 1970s. ACI-ASCE Committee "Recommendations for Design of Beam-Column Joints in Monolithic Reinforced Concrete Structures (ACI 352R-1985)" specifies the maximum allowable joint shear stresses in the form of $\gamma\sqrt{f_c'}$ MPa, where joint shear stress factor γ is a function of the joint type (i.e., interior, exterior, etc.) and of the severity of the loading, and f_c' is the concrete compressive strength. Lower limits of the flexural strength ratio M_R and joint transverse reinforcement are also confirmed by this Committee. Thus, for the beam-column connections examined in this investigation, the lower limits of M_R and γ are 1.40 and 1.00 respectively [6].

As seen in Fig. 1, all the specimens F_1, S_1, O_2 and P_2 had less column transverse reinforcement than that required by the new Greek Code for the Design of Reinforced Concrete Structures [5], did not have joint transverse reinforcement (often ties in the joint region were simply omitted in the construction process in the past because of the extreme difficulty they created in the placing of reinforcement), whereas the values of flexural strength ratio were less than 1.40, and those of the joint shear stress were greater than $1.0\sqrt{f_c'}$ MPa for all the specimens F_1, S_1, O_2 and P_2. Thus, the beam-column connections of the original specimens can be expected to fail in shear. The dimensions of the test specimens were primarily dictated by the availability of formwork and laboratory testing capacities, resulting in a beam-to-column subassemblage model of approximately 1:2 scale. The concrete compressive strengths of specimens F_1, S_1, O_2 and P_2 were 22.00 MPa, 21.80MPa, 16.20 MPa and 16.00MPa respectively.

Strengthening Technique, Specimens SO_2, SP_2, $FRPF_1$ and $FRPS_1$

Both original specimens F_1 and O_2 had experienced brittle shear failure at the joint region. Strengthening of specimen SO_2 involved encasing the original beam-column joint and the columns of O_2 with a four-sided cement grout jacket reinforced with additional collar stirrups in the joint region and additional ties in the columns.

A premixed, non-shrink, rheoplastic, flowable and non-segregating mortar of high strength with 0.95cm maximum size of aggregate was used for the construction of the cement grout jacket.

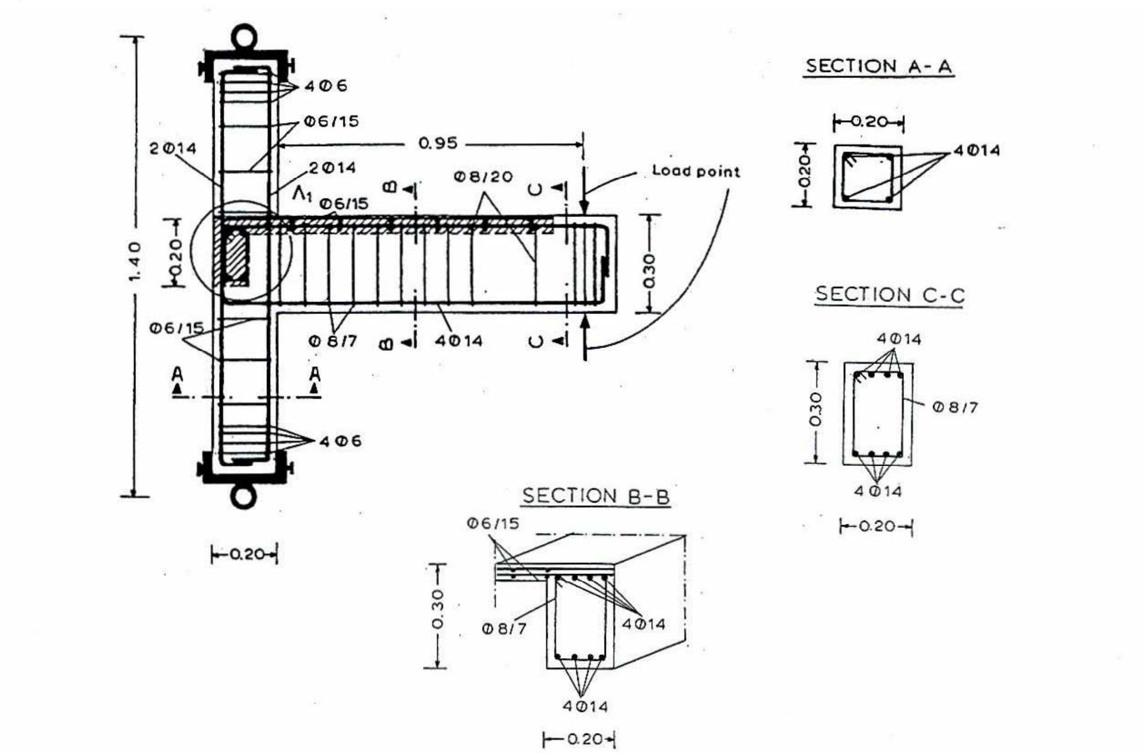


Fig. 1 Dimensions and cross-sectional details of original specimens F_1, S_1, O_2 and P_2 (dimensions in m)

As shown in Fig. 2, specimen SO_2, had a four-sided cement grout jacket, plus ∅14 longitudinal bars at each corner of the column connected by ∅8 supplementary ties at 7 cm. All longitudinal bars in the jackets extended through the joint region of the subassemblages.

Collar stirrups were used in the joint of the strengthened specimen SO_2 to increase its shear strength. These collar stirrups were inclined bars ∅14 bent diagonally across the joint core of SO_2, as shown in Fig. 2.

The columns of the strengthened specimen SO_2 satisfied all the requirements of the new Greek Code for the Design of Reinforced Concrete Structures [5] and the b/c joint region of this specimen satisfied all the requirements of the ACI-ASCE Committee 352 [6]. The subassemblage SO_2 could therefore be expected to develop flexural hinges in the beams without severe damage concentration in the joint region.

The repair measures implemented on specimen F_1 consisted of: (1) removal and replacement of all loose concrete by a premixed, non-shrink, rheoplastic, flowable, and non segregating mortar of high-strength, (2) high-strength fiber jacketing of the joint region and the columns, Fig. 2. The repaired and strengthened specimen was designated $FRPF_1$. Design for the retrofit with carbon fiber-reinforced polymer sheets (CFRPs) was based on $E_f = 230$ GPa, $t_f = 0.165$mm (t_f = layer thickness) and $\varepsilon_{fu} = 1.5\%$.

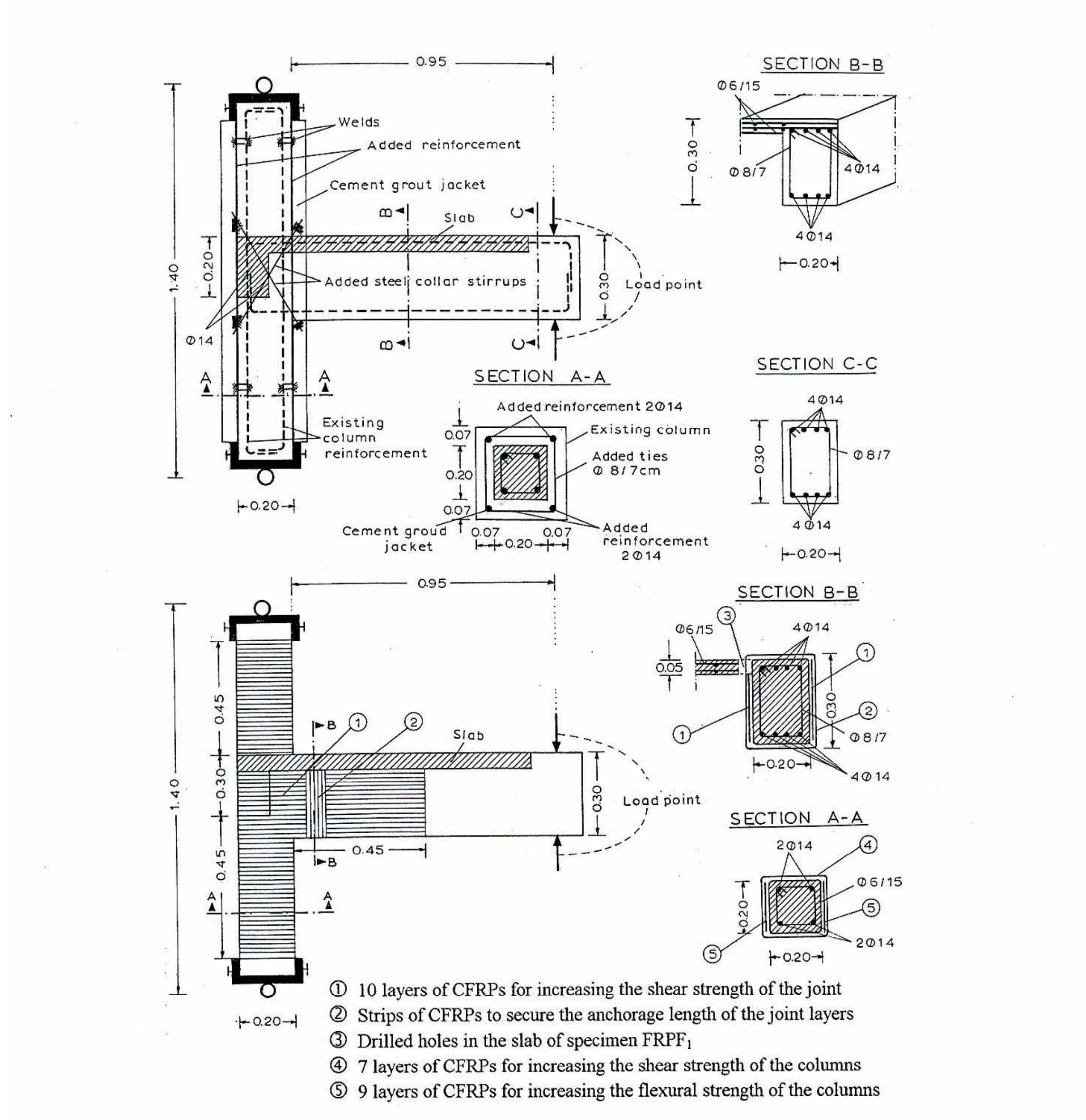


Fig. 2 Jacketing of column and beam-column connection of subassemblages SO_2, SP_2, $FRPF_1$ and $FRPS_1$ (dimensions in m)

In order to compare the effectiveness of the two jacket styles, the corresponding structural members of both the strengthened subassemblages must have the same strength. Thus, each structural member (column, joint) of specimens $CFRPF_1$ and $CFRPS_1$ had almost the same flexural and/or shear strength with that of specimens SO_2 and SP_2 respectively.

The subassemblages S_1 and P_2, represent parts of an old frame structure, which was upgraded to resist future strong earthquakes. Thus, the specimen S_1 was tested after strengthening by high-strength fiber jacketing as specimen $FRPS_1$. The strengthening scheme of this specimen was the same as that of specimen $FRPF_1$ (Fig. 2). The specimen P_2 was also tested after strengthening by reinforced concrete jacketing as specimen SP_2 and the strengthening scheme of it was the same as that of specimen SO_2.

The concrete compressive strength of the jackets of SO_2 and SP_2 were 40.70MPa and 41.00MPa respectively.

The original specimens F_1, S_1, O_2 and P_2 and the strengthened SO_2 and SP_2 were constructed using deformed reinforcement (NOTE: ∅8, ∅14 = bar with diameter 8mm, 14mm). Approximately 10 electrical-resistance strain gages were bonded in the reinforcing bars of each specimen.

TEST SETUP – LOADING SEQUENCE

A testing frame in the Laboratory of Reinforced Concrete Structures at the Aristotle University of Thessaloniki was used to apply cyclic displacements to the beam, while maintaining a constant axial load (150kN) in the column of all the specimens, Fig. 3(a). The specimens F_1, O_2, $FRPF_1$, $FRPS_1$, SO_2 and SP_2 were loaded transversely according to the load history shown in Fig. 3(b).

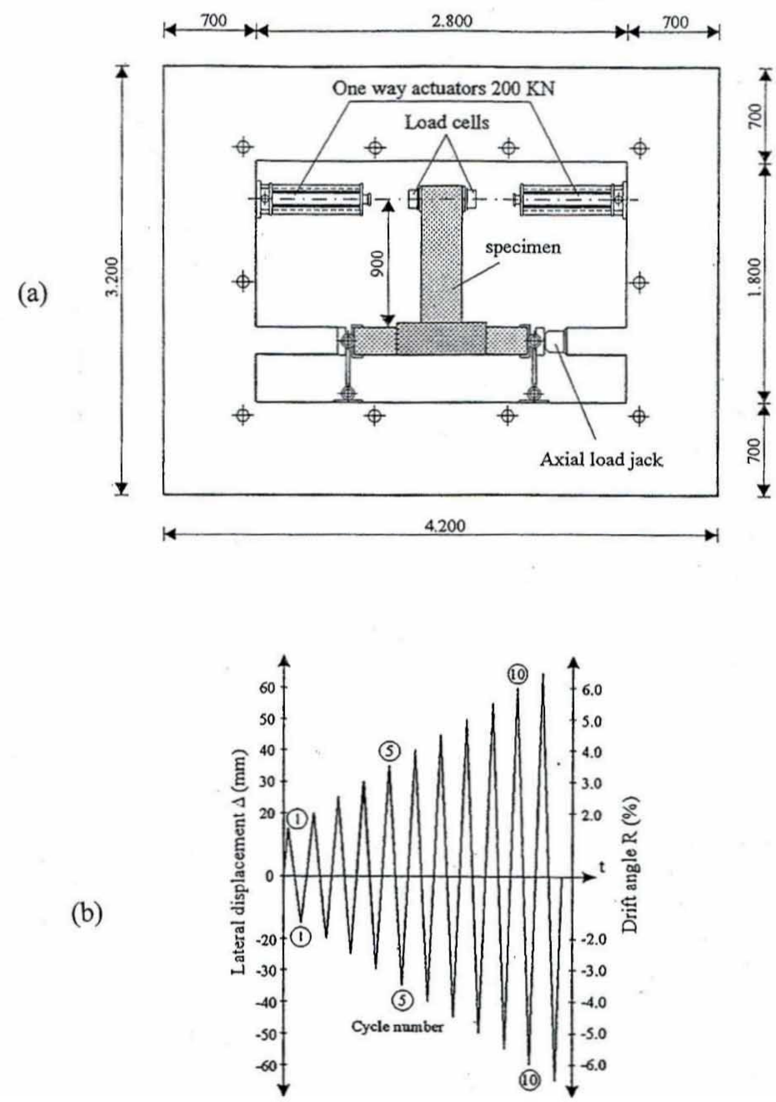


Fig. 3 (a) Test setup (dimensions in mm), (b) Lateral displacement history

TEST RESULTS

The connections of both subassemblages F_1 and O_2 exhibited as expected premature shear failure during the early stages of seismic loading. Damage occurred both in the joint area and in the columns' critical regions. The beams in both specimens F_1 and O_2 remained intact at the conclusion of the tests. Failure mode of specimens $FRPF_1$, $FRPS_1$, SO_2 and SP_2 involved, as expected, the formations of a plastic hinge in the beam near the column juncture. A difference among the failure modes of specimens $FRPF_1$, $FRPS_1$, SO_2 and SP_2 was that more damage was concentrated in the joint region of specimens $FRPF_1$ and $FRPS_1$, as opposed to that of specimens SO_2 and SP_2.

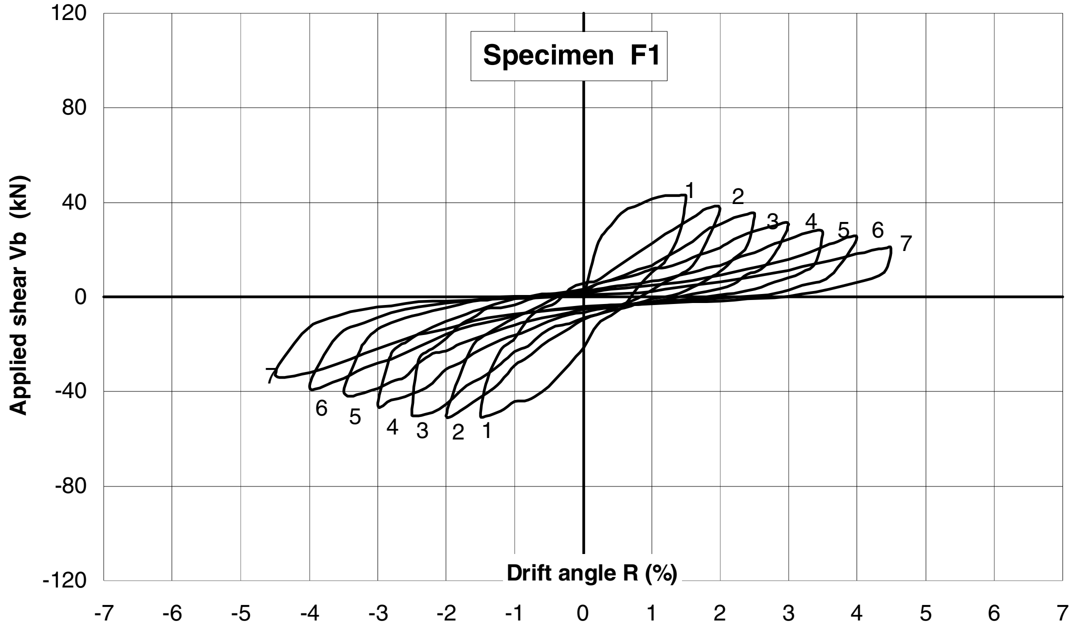
The performance of the test specimens is presented herein and discussed in terms of applied shear-versus-drift angle relations. Drift angle R, which is plotted in figures which follow, is defined as the beam tip displacement Δ divided by the beam half span L and expressed as a percentage (see the inset on Figure 4).

Plots of applied shear-versus-drift angle for all the specimens F_1, O_2, SO_2, $FRPF_1$, SP_2 and $FRPS_1$, are shown in Figure 4. The original subassemblages F_1 and O_2 showed stable hysteretic behavior up to drift angle R ratio of 2.0 percent. They showed a considerable loss of strength, stiffness and unstable degrading hysteresis beyond drift angle R ratio of 2.0 percent (Fig. 4). Strengthened specimens SO_2 and $FRPF_1$ exhibited stable hysteresis up to the 8th cycle of drift angle R, of 5.0 percent and up to the 4th cycle of drift angle R, of 3.0 percent respectively. Both specimens showed a considerable loss of strength, stiffness and unstable degrading hysteresis beyond drift angle R ratios of 5.0 percent (Fig. 4).

A comparison of the performance of strengthened specimens SO_2 and $FRPF_1$ with that of the original specimens O_2 and F_1 respectively indicated that the strengthened specimens achieved significantly increased strength, stiffness and energy dissipating capacities compared to the original specimens, even in the large displacement amplitude cycles of drift angle R ratios between 3 percent and 4.5 percent (Fig. 4).

For comparison of the effectiveness between two jacket styles it is worth studying the comparison of the peak-to-peak stiffness, the energy dissipated and the peak strength observed for every load cycle of the strengthened specimen SO_2 in a post-earthquake case by reinforced concrete jacket with that of the strengthened specimen $FRPF_1$ in a post-earthquake case by high-strength jacket. The comparison of the peak-to-peak stiffness for every load cycle is illustrated in Fig. 5 while the energy dissipated of each specimen SO_2 and $FRPF_1$ is shown in Fig. 6. Figure 7 compares the peak strength observed throughout the tests. The comparison is made by observing the ratio of the peak strengths of SO_2 to that of $FRPF_1$. From these diagrams it is clearly seen that specimen SO_2 achieved a significant increase in strength, stiffness and energy dissipating capacities as compared with those of specimen $FRPF_1$.

However, the seismic performance of specimen $FRPS_1$ strengthened in a pre-earthquake case by a high-strength fiber jacket was almost the same with the performance of specimen SP_2 strengthened in a pre-earthquake case by a reinforced concrete jacket (Fig. 4). Because of the increase in beam-column joint strength of specimen SO_2, due to the use of high-strength repair mortar, the seismic performance of SO_2 strengthened in a post-earthquake case was better than that of specimen SP_2 strengthened in a pre-earthquake case (Fig. 4).



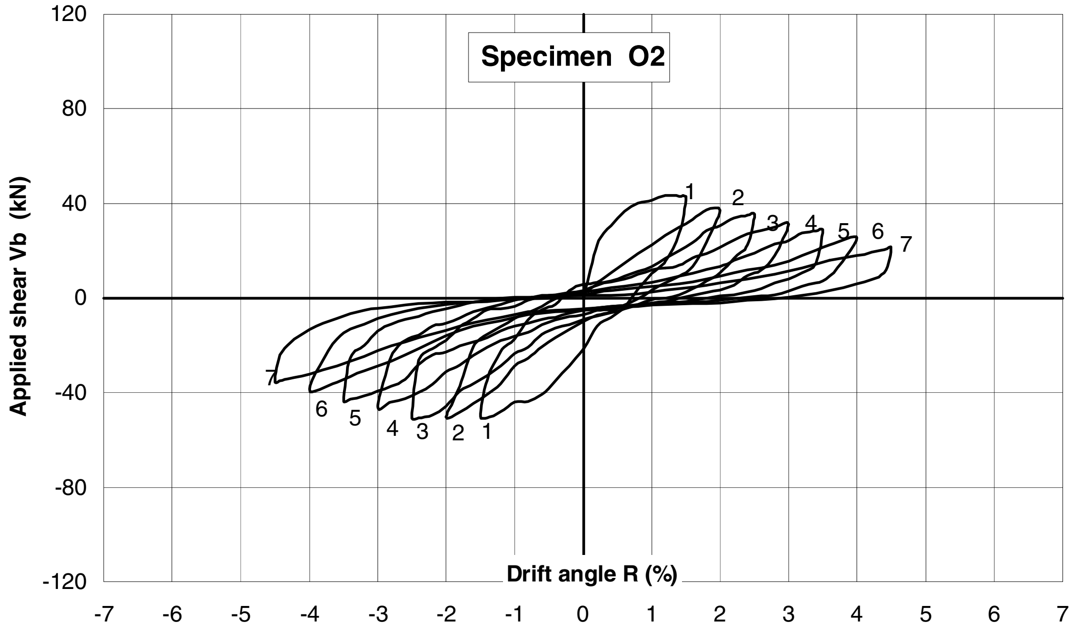
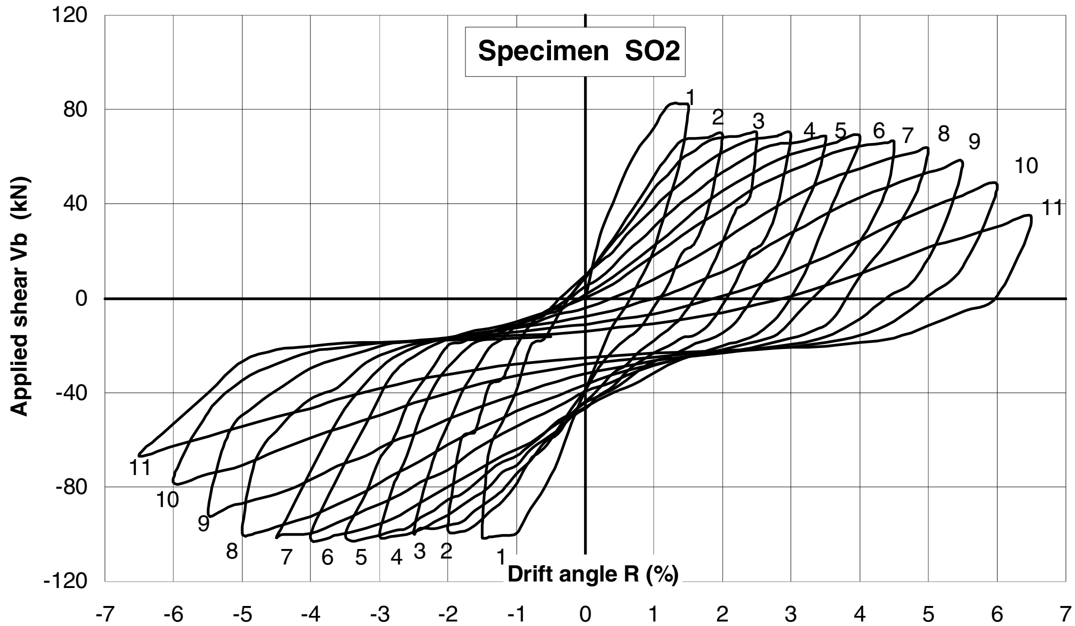


Fig. 4 Plots of applied shear-versus-drift angle for specimens F_1, O_2, SO_2, $FRPF_1$, SP_2 and $FRPS_1$



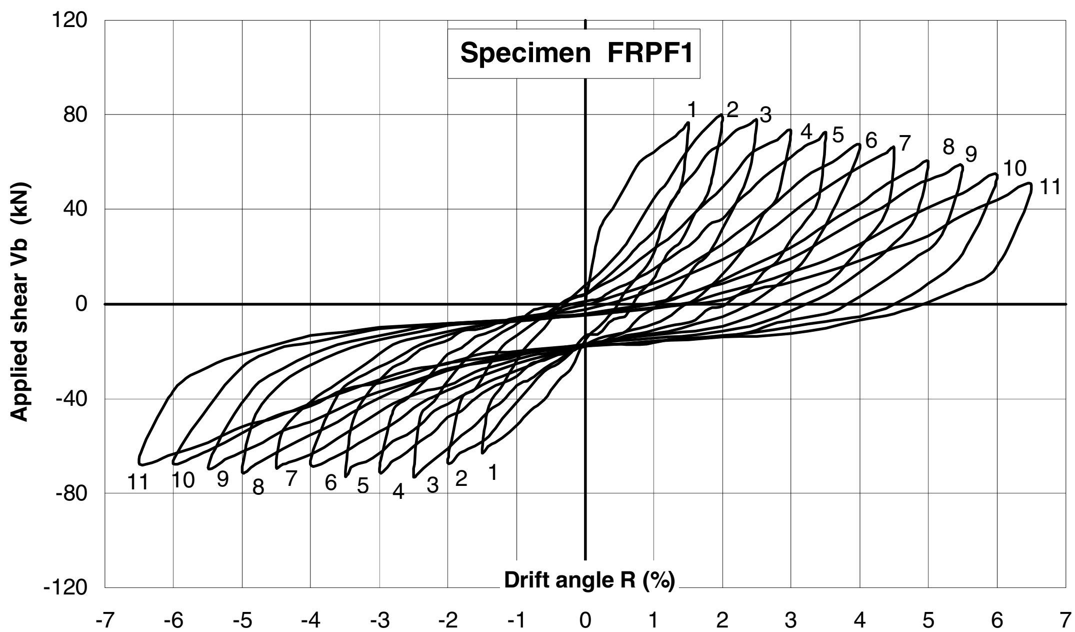
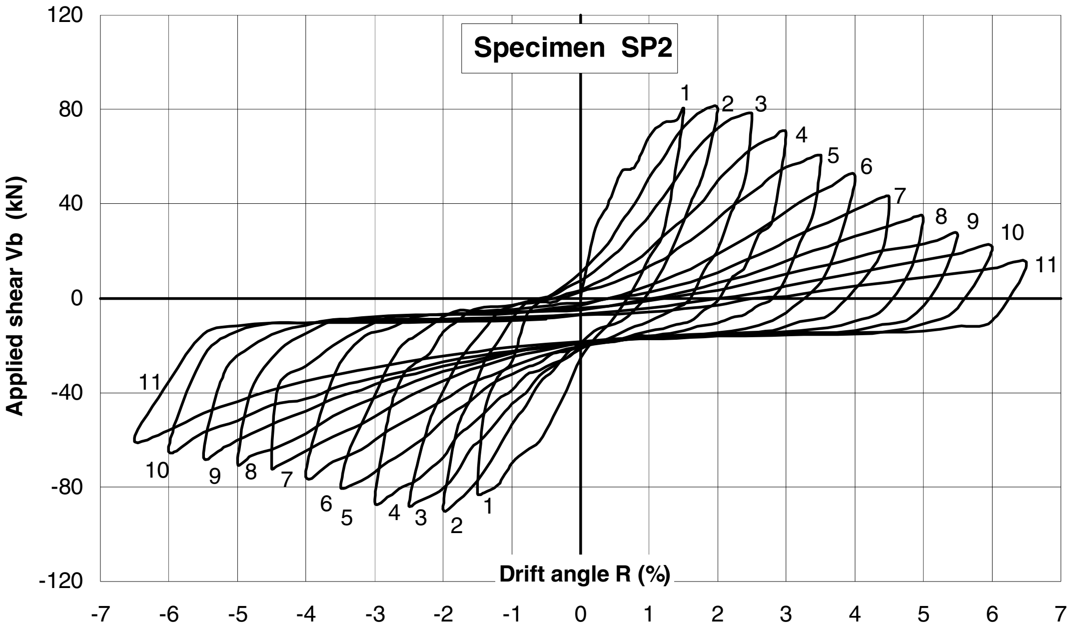
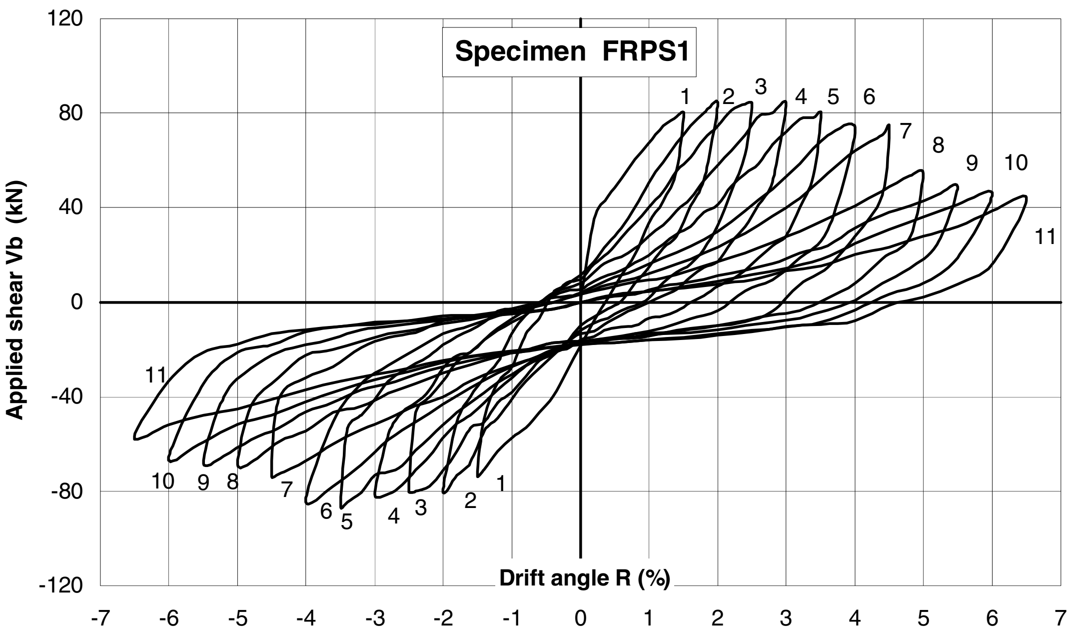


Fig. 4 Continue





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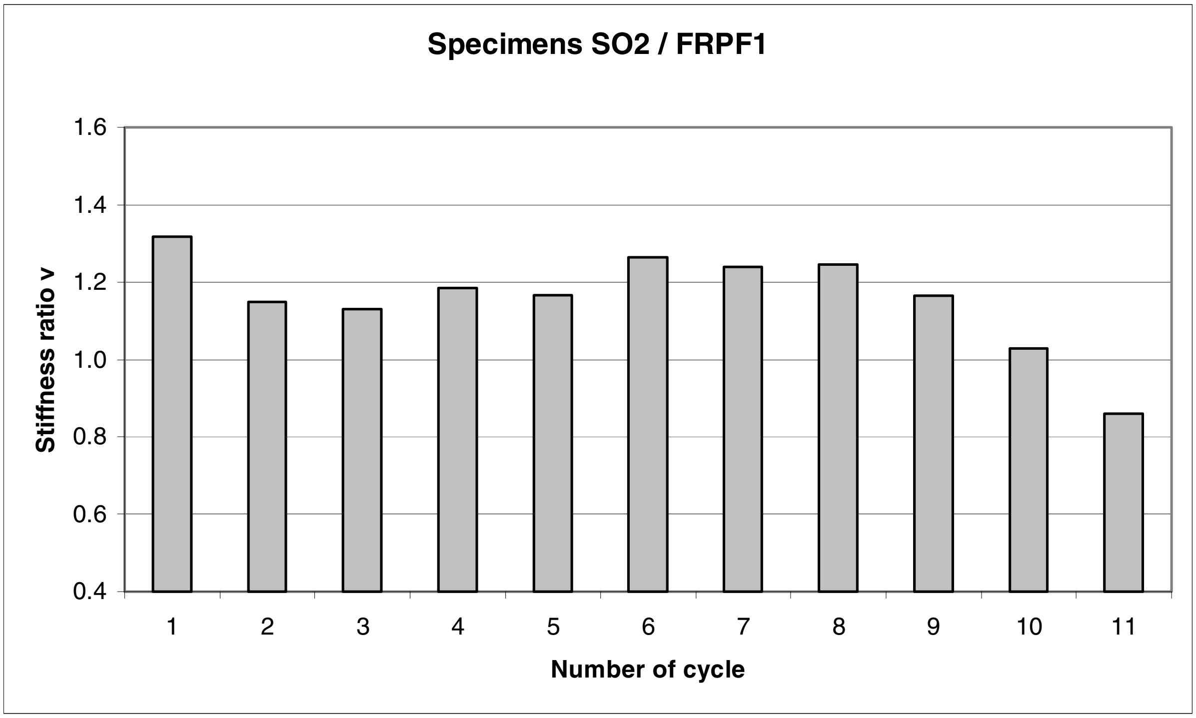


Fig. 5 Stiffness comparison between strengthened specimens SO_2 and $FRPF_1$

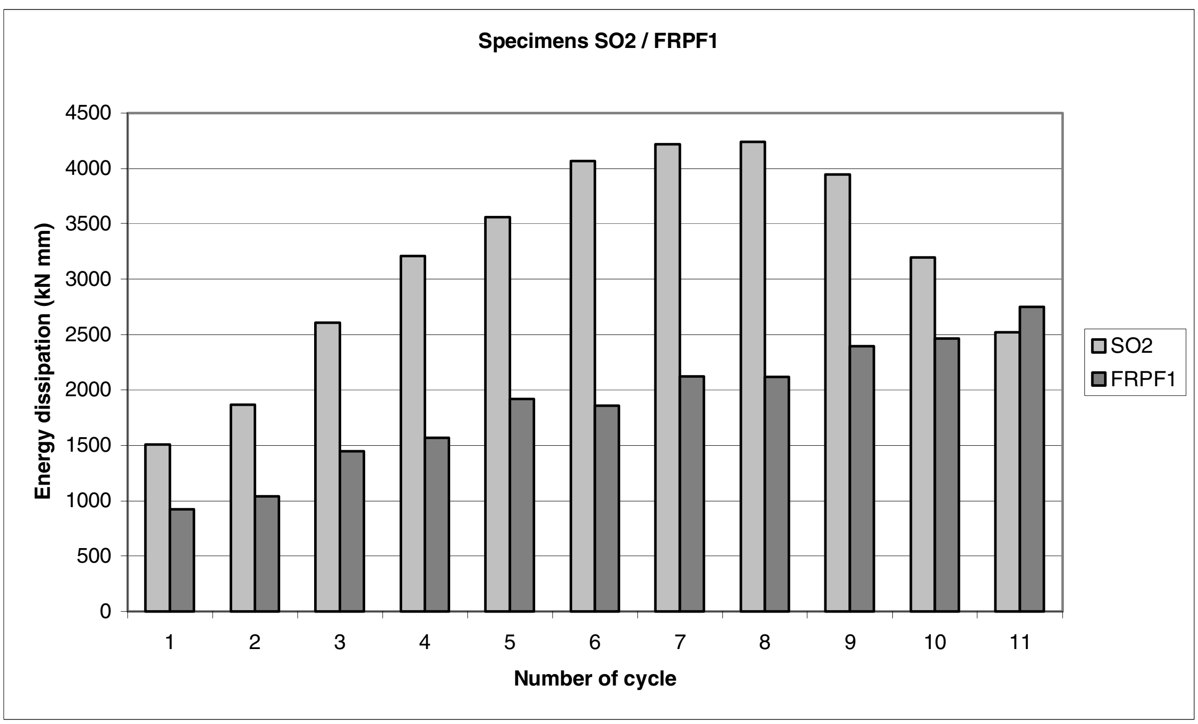
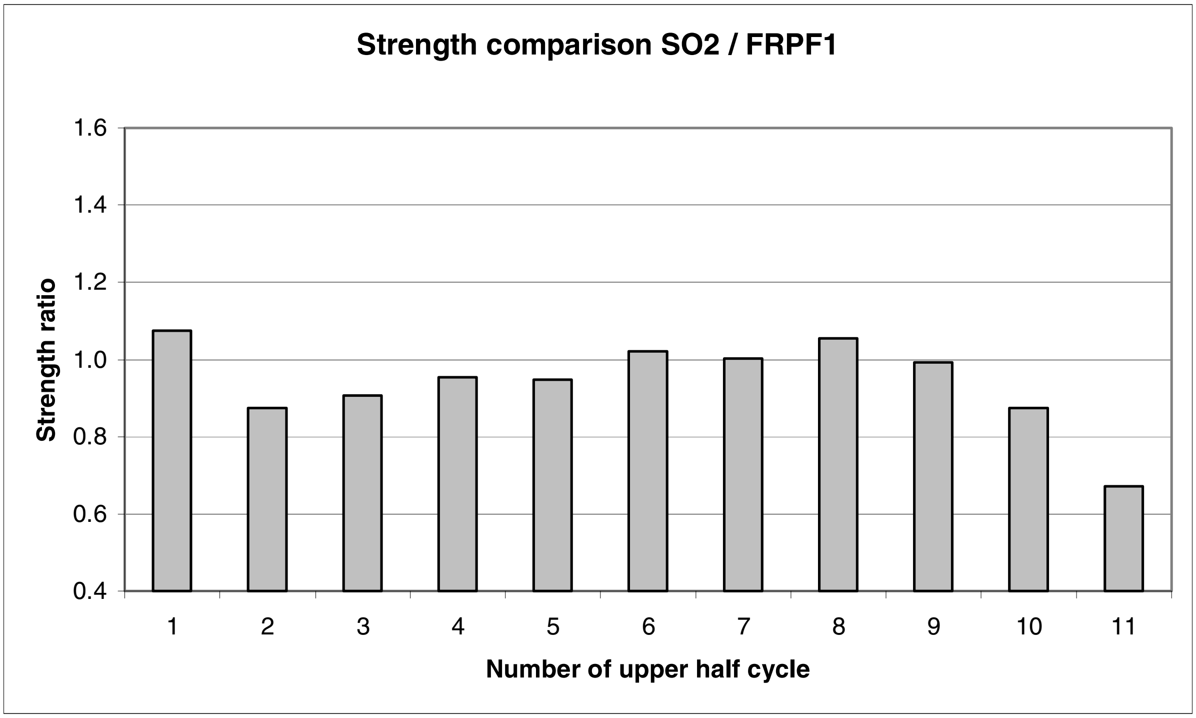


Fig. 6 Energy dissipation comparison between strengthened specimens SO_2 and $FRPF_1$



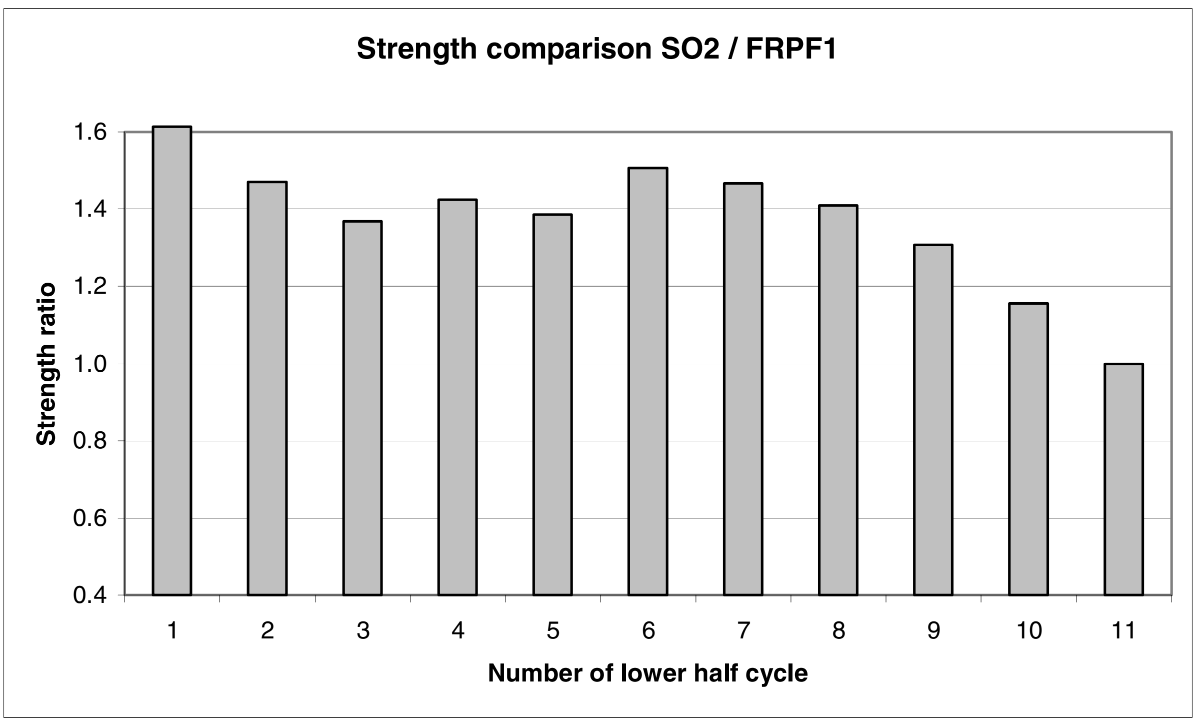


Fig. 7 Strength ratio of specimen SO_2, to specimen $FRPF_1$

CONCLUSIONS

Based on the results described in this paper, the following conclusions can be drawn.

1. Original specimens F_1 and O_2 representing an existing beam-column subassemblage, performed poorly under reversed cyclic lateral deformations. The connections of these subassemblages exhibited premature shear failure during the early stages of seismic loading, and damage to both subassemblages was concentrated in the joint region.
2. The retest of failed beam-column subassemblages repaired and strengthened with fiber carbon/epoxy jacketing or with reinforced concrete jacketing showed that both the employed repair and strengthening techniques were effective in transforming the brittle joint shear failure mode of original specimens (F_1 and O_2), into a more ductile failure mode of strengthened specimens, which developed flexural hinges in their beams. Damage to the strengthened specimens $FRPF_1$ and SO_2 was concentrated both in the beam critical region and in the joint area.
3. The effectiveness of the reinforced concrete jacket system and the high-strength jacket system was demonstrated both in a post-earthquake and pre-earthquake retrofitting case of reinforced concrete columns and b/c joints.
4. It was demonstrated that the reinforced concrete jacket is a more effective way of post-earthquake retrofitting columns and b/c joints than the high-strength fiber jacket.

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REFERENCES

1. Hakuto, S., Park, R. and Tanaka, H. “Seismic Load Tests on Interior and Exterior Beam-Column Joints with Substandard Reinforcing Details”. ACI Structural Journal 2000; 97(1): 11-25.
2. ACI Committee 440. “State of the art Report on Fiber Reinforced Plastic Reinforcement for Concrete Structures (ACI 440R-96)”, American Concrete Institute, Detroit, 1996.
3. Priestley, M.J.N., Seible, F. and Calvi, G.M. “Seismic Design and Retrofit of Bridges”, A Wiley-Interscience Publication, John Wiley & Sons Inc., New York, 1996.
4. Tsonos, A.G. and Stylianides, K. “Seismic Retrofit of Beam-to-Column Joints with high-strength fiber-jackets”. Journal of European Association for Earthquake Engineering 2002; (2): 56-72.
5. Greek Code for the Design of Reinforced Concrete Structures. Ministry of Environment, Land Planning and Public Works, General Secretariat of Public Works, Athens, 2000.
6. ACI-ASCE Committee 352. “Recommendations for Design of Beam-Column Joints in Monolithic Reinforced Concrete Structures (ACI 352R-85)”. ACI Structural Journal 1985; 82(3): 266-83.