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Effectiveness of CFRP-Jackets and R/C-Jackets in Post-Earthquake Retrofitting of Beam- Column Subassemblages

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Summary

The use of a reinforced concrete jacket and a high-strength fiber jacket for the repair and strengthening of reinforced concrete beam-column subassemblages damaged by severe earthquakes is investigated experimentally. In the paper the effectiveness of the two jacket styles is also compared.

Introduction

The feasibility and technical effectiveness of the high-strength fiber jacket system and the reinforced concrete jacket system in a post-earthquake retrofitting case of columns and beam-column joints was investigated in the paper. Thus, two identical reinforced concrete exterior beam-column-slab-transverse beam subassemblages were constructed with non-optional design parameters: flexural strength ratio, joint shear stress, with less column transverse reinforcement than that required by the modern Codes [1] and without joint transverse reinforcement representing the common construction practice of column and beam-column joints of older structures built in the 1960s and 1970s. The subassemblages 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 upgraded specimens were again subjected to the same cyclic lateral load history. The measured response histories of the original and strengthened specimens were subsequently compared and evaluated. The effectiveness of the two jacket styles was also compared.

Description of the Specimens

Two test specimens F_1 and O_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

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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 [2].

Both specimens F_1 and O_2 had less column transverse reinforcement than that required by the new Greek Code for the Design of Reinforced Concrete Structures [1], 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 both specimens F_1 and O_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 and O_2 were 22.00 MPa and 16.20 MPa respectively.

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.

As shown in Fig. 1, 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. 1.

The columns of the strengthened specimen SO_2 satisfied all the requirements of the new Greek Code for the Design of Reinforced Concrete Structures [1] and the b/c joint region of this specimen satisfied all the requirements of the ACI-

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ASCE Committee 352 [2]. The subassemblage SO_2 could therefore be expected to develop flexural hinges in the beams without severe damage concentration in the joint region.

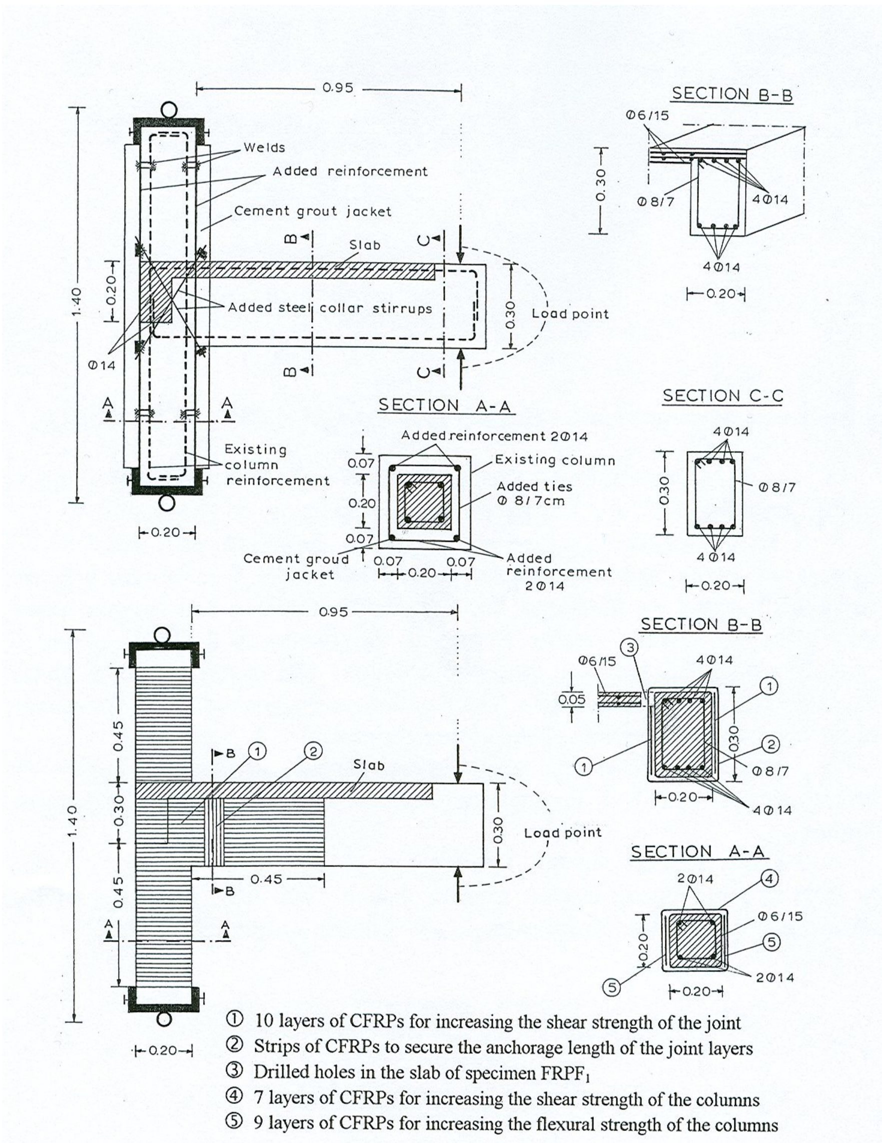

Fig. 1 Jacketing of column and beam-column connection of subassemblages SO_2 and $FRPF_1$ (dimensions in m)

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The concrete compressive strength of the jackets of SO_2 was 40.70 MPa.

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. 1. 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 = 0.165mm (t_f = layer thickness) and ε_{fu} = 1.5%.

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 specimen $CFRPF_1$ had almost the same flexural and/or shear strength with that of specimens SO_2.

The original specimens F_1 and O_2 and the strengthened SO_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 Results

The connections of both subassemblages F_1 and O_2, as expected, exhibited 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 both specimens $FRPF_1$ and SO_2 involved, as expected, the formation of a plastic hinge in the beam near the column juncture. A difference between the failure modes of specimens $FRPF_1$ and SO_2 was that more damage was concentrated in the joint region of $FRPF_1$ compared to that of specimen SO_2.

Plots of applied shear-versus-drift angle for specimens O_2, SO_2 and $FRPF_1$ are shown in Figure 2. 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. 2). 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 3.5 percent respectively. Both specimens showed a considerable loss of strength stiffness and unstable degrading hysteresis beyond drift angle R ratios of 5.0 percent.

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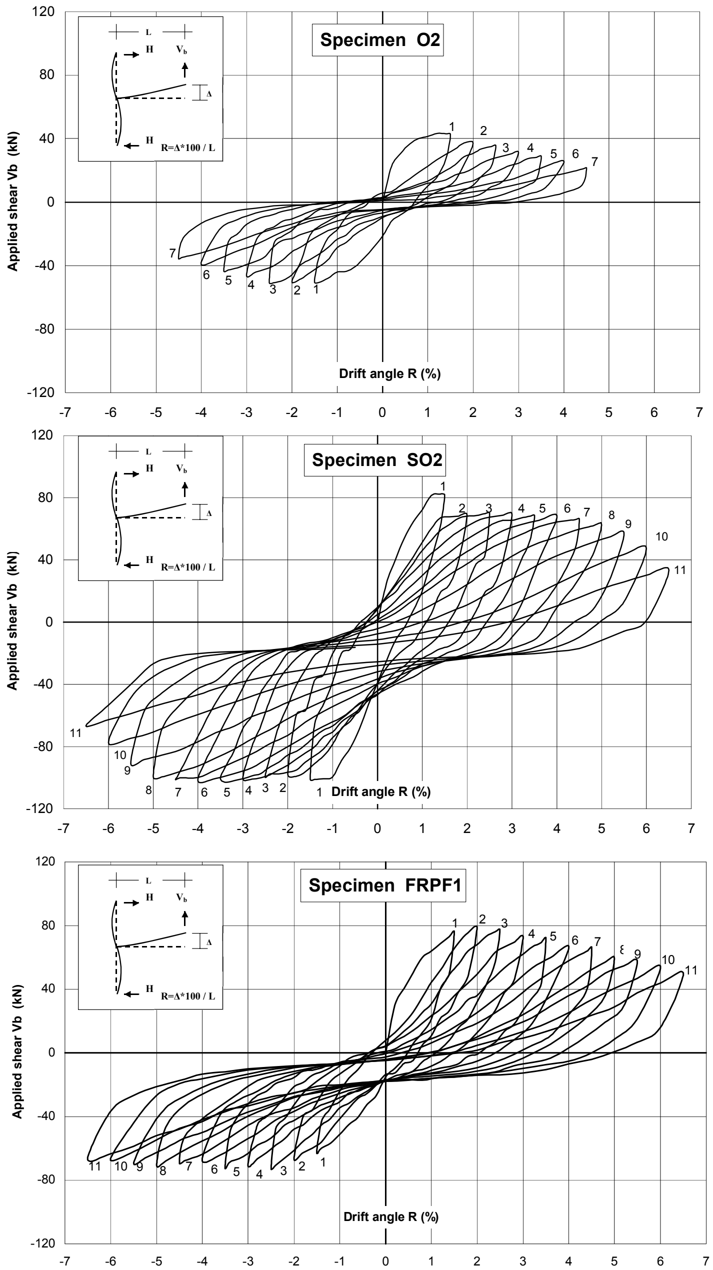

Fig. 2 Applied shear-versus-drift angle for specimens O_2, SO_2 and $FRPF_1$

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From these diagrams, it is clearly seen 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. The increases of strength, stiffness and energy dissipation capacity of subassemblage SO_2 strengthened by a reinforced concrete jacket (as compared with those of the original O_2) were much higher than the increases of strength, stiffness and energy dissipation capacity of subassemblage $FRPF_1$ strengthened by a high-strength fiber jacket (as compared with those of the original F_1).

Conclusions

1. 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, into a more ductile failure mode of strengthened specimens, which developed flexural hinges in their beams. Damage to the strengthened specimens was concentrated both in the beam critical region and in the joint area.
2. It was demonstrated that the reinforced concrete jacket is a more effective way of retrofitting columns and b/c joints than the high-strength fiber jacket..

Acknowledgements

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