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An innovative solution for strengthening of old R/C structures and for improving the FRP strengthening method

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Abstract. In this study a new innovative method of earthquake-resistant strengthening of reinforced concrete structures is presented for the first time. Strengthening according to this new method consists of the construction of steel fiber ultra-high-strength concrete jackets without conventional reinforcement which is usually applied in the construction of conventional reinforced concrete jackets. An innovative solution is proposed also for the first time that ensures a satisfactory seismic performance of existing reinforced concrete structures, strengthened by using composite materials. The weak point of the use of such materials in repairing and strengthening of old R/C structures is the area of beam-column joints. According to the proposed solution, the joints can be strengthened with a steel fiber ultra-high-strength concrete jacket, while strengthening of columns can be achieved by using CFRPs. The experimental results showed that the performance of the subassemblage strengthened with the proposed mixed solution was much better than that of the subassemblage retrofitted completely with CFRPs.

Keywords: steel fiber ultra high-strength concrete; reinforced concrete jackets; fiber reinforced polymers; beam-column joints; columns; cyclic loads

1. Introduction

Damage incurred by earthquakes over the years has indicated that many reinforced concrete (R/C) buildings, designed and constructed during the 1960s and 1970s, were found to have serious structural deficiencies today. These deficiencies are mainly due to lack of capacity design approach and/or poor detailing of the reinforcement. As a result, lateral strength and ductility of these structures were minimal and hence some of them collapsed (Paulay and Park 1984, Park 2002, Karayannis *et al*. 1998). One of the most popular pre-and post-earthquake retrofitting methods for columns, beam-column joints and walls is the use of reinforced concrete jacketing. In retrofitting building columns, b/c joints and walls with outer R/C jackets, the usual practice consists of first assembling the jacket reinforcement cages, arranging the formwork and then placing the concrete jacket (Karayannis *et al.* 2008, Rodriguez and Santiago 1998, Tsonos 2002, Tsonos 2010, UNDP/UNIDO 1983). Shotcrete can be used in lieu of conventional concrete in the repair works and, in some cases, offers advantages over it, the choice being based on convenience and cost (Tsonos 2010).

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The wrapping of reinforced concrete members (usually columns, b/c joints and walls) 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 through the world in numerous retrofit applications in reinforced concrete buildings. These are recognised as alternate strengthening systems to conventional methods such as plate bonding and shotcreting (FIB (CEB-FIB) 2006, Chalioris 2007, Chalioris 2008, Tsonos 2008, Karayannis and Sirkelis 2008).

The best choice of the appropriate retrofitting method highly depends on the feasibility of the method, on the cost and on the simplicity of the application. Of course, it is well known that the works related to strengthening of buildings have higher difficulties and cost compared to the usual construction works related to the construction of new reinforced concrete buildings.

According to the above conception it would be very interesting to create and introduce in the marketing a new method of retrofitting old reinforced concrete structures, as effective as the other methods of retrofitting but simpler in application and more economical. An earthquake strengthening system with the aforementioned qualifications would be very competitive among the others.

Henager (1977), successfully replaced all the hoops of the joint region and part of the hoops of the critical regions of the adjacent beam and column of an earthquake-resistant beam-column subassemblage, by steel fibers (1.67% fiber volume fraction is used). This replacement involved 50% reduction in building costs.

Fiber Reinforced Concrete or Shotcrete has been successfully applied in many construction applications eliminating or significantly reducing the conventional reinforcement of R/C structures and reducing the construction costs.

The advantages of Fiber Reinforced Concrete has been worldwide recognised (Chalioris and Karayannis 2009, Chalioris and Sfiri 2011, Chalioris 2013a, b), however has not been found yet a reliable way of application of this material in the retrofitting of old reinforced concrete structures, by eliminating or significantly reducing the conventional reinforcement of the R/C jacketings and generally by reducing the cost of retrofitting compared to that involved by the use of other strengthening methods as plate bonding and FRPs. A relatively new process called SIMCON (Slurry Infiltrated Mat Concrete) developed by Hackman *et al*. (1992), seems to be very effective in strengthening applications. SIMCON is made by infiltrating continuous steel fiber-mats, with specially designed cement-based slurry. Nevertheless, SIMCON technique has the same disadvantages as FRPs. Their strengthening layers wrap usually horizontally the columns and the walls increasing their shear strength and ductility, but these layers are terminating in the slabs of the strengthening reinforced concrete buildings. The strengthening layers could not effectively pass through the slabs, thus these layers could not increase the flexural strength of the columns and walls and could not effectively retrofit the beam-column joint regions. The existing experimental results related to the retrofitting of beam-column subassemblages of reinforced concrete structures demonstrated significant damage concentration in the joint regions, although the subassemblages used were of planar-type, without slabs and the retrofitting works related to SIMCON application were easy (Dogan and Krstulovic-Opara 2003).

2. The proposed new innovative strengthening method

An important experiment was conducted by Tsonos (2002). An exterior beam-column subassemblage L_3 poorly detailed in the joint region was subjected to unidirectional reversed cyclic lateral loading. The joint region of this subassemblage was representative of the joint regions of old structures built during the 1960s and 1970s. The subassemblage was reinforced in the joint region by one hoop of diameter 8mm instead of the five hoops of the same diameter required by the ACI-ASCE Committee 352 (ACI 352R-02 2002). The joint shear stress of the specimen was higher than the maximum allowable joint shear stress by the same Committee ($\tau_{joint} = 1.36\sqrt{f'_c} > \tau_{permitted} = 1.0\sqrt{f'_c}$). As expected, this specimen failed in pure and premature joint shear failure from the early stages of the seismic-type loading. The removal and replacement of the damaged concrete in the joint by a non-shrink, non-segregating steel fiber concrete of high-strength with only 0.5% fiber volume fraction and the removal and replacement of the damaged concrete cover of part of the columns' critical regions with the same steel fiber high-strength concrete, resulted in a pure beam failure, when the repaired subassemblage RL_3 was imposed to the same loading as the original control subassemblage L_3.

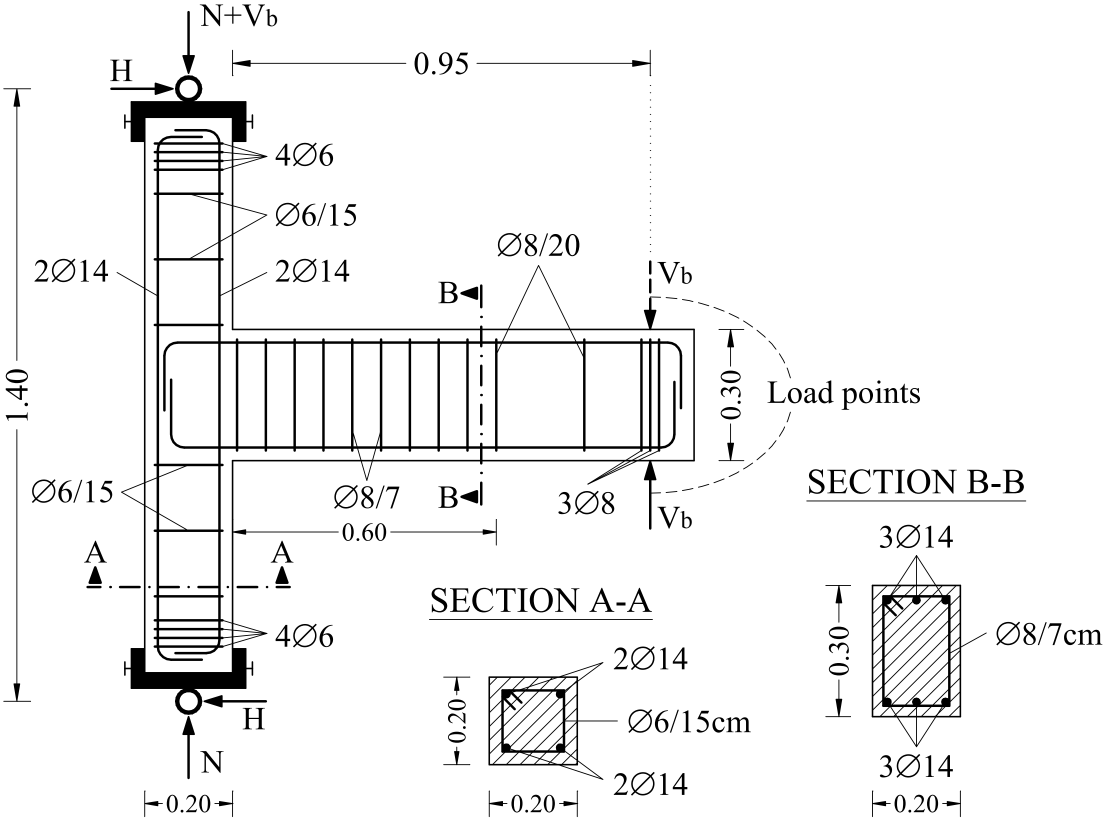


Fig. 1 Dimensions and cross-sectional details of original subassemblages O_3, W_2, W_3, M_1, and M_3

The above experiment led us to the idea of using the same non-shrink, non-segregating steel fiber high-strength concrete for the strengthening of old reinforced concrete buildings, by jacketing not with the use of conventional reinforcement, longitudinal bars or hoops (Tsonos 2006). For this purpose and for best results, it was decided to increase the fiber volume fraction and to increase the compressive and tensile strengths of the steel fiber concrete. The following large experimental program was implemented. Four identical exterior beam-column subassemblages were constructed, using normal weight concrete and deformed reinforcement. The test specimens were 1:2 scale models of the representative 40 cm×40 cm square columns and beam-column joints which are usually found in building constructions within Greece and Europe in general. The columns and b/c joints of these specimens were poorly detailed in order to represent columns and b/c joints of old buildings built in 1960s and 1970s. In Fig. 1 are shown the dimensions and cross-sectional details of these specimens. Their columns had less longitudinal and transverse reinforcement than the modern columns and their joint regions had not joint hoops, the joint shear stress were $2.20\sqrt{f'_c}$ MPa > $1.0\sqrt{f'_c}$ MPa, and the flexural strength ratios of these specimens were lower than 1.0 (Table 1). The concrete compressive strength of these original specimens was approximately 9.00 MPa.

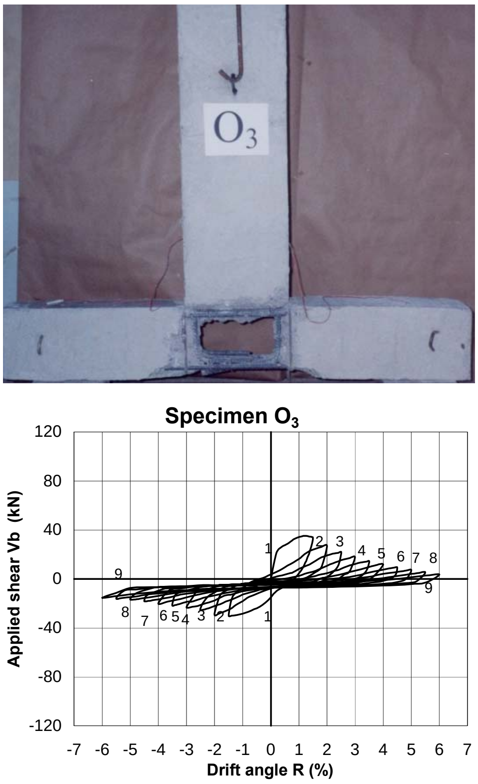


Fig. 2 Plots of applied shear versus drift angle and failure mode of the original subassemblage O_3

Thus, a premature joint shear failure is expected for all these subassemblages during a seismic type loading. All these original specimens were subjected to cyclic lateral load histories so as to provide the equivalent of severe earthquake damage. In Fig. 2 is shown the failure mode of the representative specimen O_3 and its hysteresis loops. The failure of O_3 was concentrated mainly in the joint, which lost almost all of the core's concrete.

In the following are described in brief the retrofitting works for specimens O_3, W_2, M_1, and M_3.

1. Specimen O_3 was retrofitted by reinforced concrete jacket in the columns and beam-column joint region. The compressive strength of the jacket's concrete was 31.70MPa. Deformed bars were used for the construction of the steel cage of the jacket. After the interventions this specimen was designated SO_3. In Fig. 3 is shown the jacketing of column and beam-column connection of subassemblage SO_3.
2. Specimen W_2 was strengthened by a high-strength fiber jacketing in the joint region and in the columns (see Fig. 3). The damaged concrete of the joint region of specimen W_2 was removed and replaced by a premixed, non-shrink, rheoplastic, flowable and non-segregating concrete of high-strength. The repaired and subsequently strengthened specimen was named FW_2. The design for the retrofit process with carbon fiber-reinforced polymer sheets (CFRPs) was based on $E_f = 235$GPa, $t_f = 0.11$mm (t_f = layer thickness) and $\varepsilon_{fu} = 1.5\%$ (ε_{fu} = ultimate FRP strain).
3. Subassemblage M_1 was strengthened by jacketing with ultra high-strength steel fiber-reinforced concrete (UHSFC) with 1.5% fiber volume fraction in the columns and in the joint region. The thickness of the jacket was only 4.0 cm (Table 2). The repaired and subsequently retrofitted specimen was named $HSFM_1$ (see Fig. 3).
4. Subassemblage M_3 was retrofitted by jacketing with UHSFC with 1.0% fiber volume fraction, in the columns and in the joint region. The thickness of the jacket was 6.0cm (Table 2). The repaired and strengthened specimen was named $HSFM_3$ (see Fig. 3).

The compressive strengths of the UHSFC used for the strengthening of $HSFM_1$ and $HSFM_3$ were 106.33MPa and 102.30MPa respectively. The tensile strength of the UHSFC used, was approximately equal to 12MPa (Table 2). The characteristic toughness indexes I_{20} according to ASTM-C1018 for the ultra high-strength steel fiber-reinforced concrete (UHSFC) of specimens $HSFM_1$, and $HSFM_3$ were approximately 12.50. The steel fibers used were Dramix ZP30/0.6.

In Table 1 there are the flexural strength ratio M_R, the joint shear stress factor γ and the joint shear stress τ_{jh} of the strengthened subassemblages SO_3, FW_2, $HSFM_1$ and $HSFM_3$. From this Table it is clearly seen the improvement of these three factors in the strengthened subassemblages compared to those in their corresponding original ones O_3, W_2, M_1 and M_3.

All the above strengthened subassemblages SO_3, FW_2, $HSFM_1$ and $HSFM_3$ were imposed to the same loading as that of their original subassemblages. All strengthened specimens demonstrated increased strength, stiffness and energy dissipation capacity as compared to those of their original specimens (compare hysteresis loops between the original and the upgraded subassemblages in Figs. 2 and 4 e.g., O_3 – $HSFM_1$). However, the failure mode of SO_3 and FW_2 was quite different from that of all upgraded specimens by the new proposed jackets $HSFM_1$ and $HSFM_3$. Thus although, the beams of both SO_3 and FW_2 yielded, the majority of the damage was concentrated in their joint regions, see failure modes of specimens in Fig. 4. On the contrary, the failure mode of both specimens $HSFM_1$ and $HSFM_3$ was the optimum one. Formation of plastic hinge in their beams was observed from the first cycles of loading, while the following cycles resulted in damage concentration only in the critical regions of their beams near their joints. A mixed flexural – shear failure mode was observed in their beams at the end of the tests, which was accompanied by severe buckling of the longitudinal beam reinforcement. The joints and the columns of both these specimens were intact at the conclusion of the tests. This excellent seismic performance of both the $HSFM_1$ and $HSFM_3$ subassemblages was demonstrated both in their failure modes (Fig. 4) and in their hysteresis loops (Fig. 4). The seismic behaviour of both these subassemblages was superior to those of specimens SO_3 and FW_2 retrofitted by reinforced concrete jackets and FRP-jackets.

A patent No 1005657 was awarded to Professor Tsonos (2007) by the Greek Industrial Property Organization for the above invention.

Table 1 Flexural strength ratio M_R, joint shear stress factor γ and joint shear stress τ_{jh} of subassemblages O_3, W_2, W_3, W_4, M_1, M_3, SO_3, FW_2, $HSFM_1$, $HSFM_3$, $FHSFW_3$ and $FHSFW_4$.

Specimen	$M_R^{(1)}$	$\gamma^{(1)}$	τ_{jh}
O_3	0.98 (1.20)	2.25 (1.00)	6.42
W_2	0.95 (1.20)	2.04 (1.00)	6.43
W_3	0.95 (1.20)	2.05 (1.00)	6.43
W_4	0.96 (1.20)	2.15 (1.00)	6.41
M_1	0.98 (1.20)	2.22 (1.00)	6.41
M_3	0.98 (1.20)	2.18 (1.00)	6.41
SO_3	2.66 (1.20)	0.45 (1.00)	1.28
FW_2	1.55 (1.20)	2.04 (1.00)	6.43
$HSFM_1$	1.13 (1.20)	0.31 (1.00)	0.89
$HSFM_3$	1.40 (1.20)	0.25 (1.00)	0.74
$FHSFW_3$	1.55 (1.20)	0.28 (1.00)	0.88
$FHSFW_4$	1.55 (1.20)	0.29 (1.00)	0.91

(1)Numbers outside the parentheses are the provided values, numbers inside the parentheses are the required values by the ACI-ASCE Committee 352-02

Table 2 Details of strengthened subassemblages $HSFM_1$, $HSFM_3$, $FHSFW_3$, $FHSFW_4$.

Specimen	Thickness of the jacket (mm)	Compressive strength of steel fiber concrete (MPa)	Tensile strength of steel fiber concrete (MPa)	Fiber volume fraction (%)
$HSFM_1$	40	106.33	12.20	1.5
$HSFM_3$	60	102.30	11.90	1.0
$FHSFW_3$	50	106.33	12.10	1.5
$FHSFW_4$	50	106.40	11.50	1.0

3. An innovative new solution for improving the FRP strengthening method

An innovative solution is proposed also for the first time. This solution ensures a satisfactory and perhaps perfect seismic performance of existing old reinforced concrete buildings strengthened by using composite materials FRPs. The weak point in using such materials in repairing and strengthening reinforced concrete structures is the area of beam-column joints. Indeed, all the strengthened subassemblages in the beam-column region with composite materials FRPs of the literature demonstrated in the best case a mixed type failure during seismic type loading. Thus, during the first cycles of loading their beams yielded, however during the following cycles a large part of damage of these strengthened subassemblages was concentrated in their joint regions. Of course, this failure mode is highly dangerous for the people who live in old buildings which were retrofitted in post-earthquake or pre-earthquake cases. The representative failure mode of subassemblage FW_2 clearly demonstrates this critical situation, Fig. 4. The whole strengthened beam-column joint region of FW_2 not only failed but also was removed (i.e., leaving a hole in this position) during the last cycles of loading. This exactly is the reason why the Greek Code of the Repair and Strengthening of Reinforced Concrete Buildings (2009) does not allow the use of composite materials for the strengthening of reinforced concrete beam-column joints.

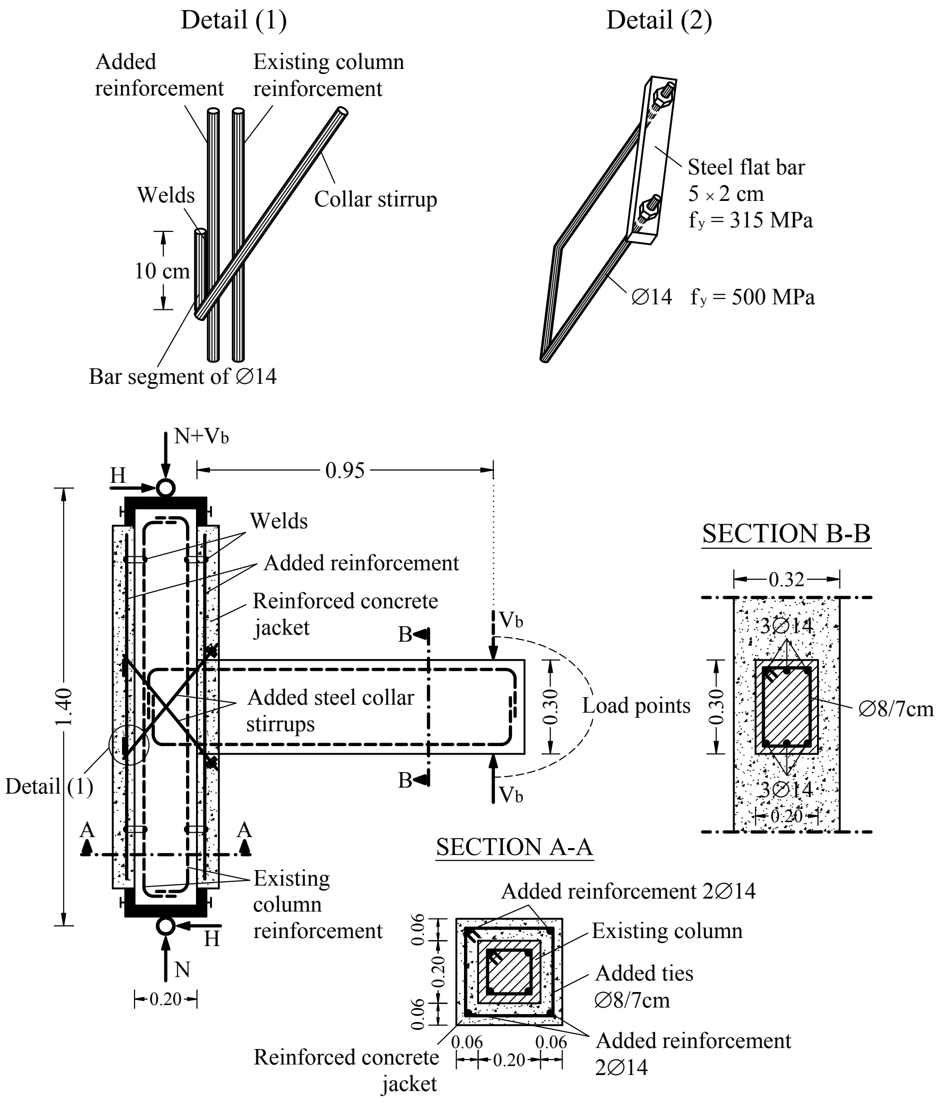
The second innovative solution presented in this study consists of strengthening the joint regions of subassemblages with a local jacket of ultra-high-strength steel fiber concrete with 1.5% fiber volume fraction, while retrofitting the columns can be achieved by using composite materials FRPs. In order to investigate the effectiveness of the proposed solution of mixed type strengthening two new beam-column subassemblages W_3 and W_4 identical with the other four (O_3, W_2, M_1 and M_3, Fig. 1), were constructed and were imposed to seismic type loading as the other original subassemblages. The failure mode of W_3 and W_4 was the same as that of O_3 previously described. The subassemblages were retrofitted by the new mixed type technique shown in Fig. 5. After the interventions these specimens were designated $FHSFW_3$ and $FHSFW_4$. The columns of $FHSFW_3$, $FHSFW_4$ and FW_2 were retrofitted exactly in the same way with composite materials CFRPs. The joint regions of $FHSFW_3$ and $FHSFW_4$ were retrofitted with ultra high-strength steel fiber concrete. The compressive strengths of the UHSFC used for the strengthening of $FHSFW_3$ and $FHSFW_4$ were 106.33MPa and 106.40MPa respectively. The tensile strengths were approximately 12.00MPa. (Table 2). The characteristic toughness indexes I_{20} according to ASTM-C1018 for the ultra high-strength steel fiber-reinforced concrete (UHSFC) of specimens $FHSFW_3$ and $FHSFW_4$ were approximately 12.50. The steel fibers used were Dramix ZP30/0.6.

In Table 1 there are the flexural strength ratio M_R, the joint shear stress factor γ and the joint shear stress τ_{jh} of the strengthened subassemblages $FHSFW_3$ and $FHSFW_4$. From this Table it is clearly seen the improvement of these three factors in the strengthened subassemblages compared to those in their corresponding original ones W_3 and W_4.

The only difference between the retrofitted schemes of the two subassemblages $FHSFW_3$ and $FHSFW_4$ was that the fiber volume fraction of the ultra-high-strength steel fiber concrete of $FHSFW_3$ was 1.5% and the fiber volume fraction of ultra-high-strength steel fiber concrete of the $FHSFW_4$ was 1.0% (Table 2).

Both subassemblages $FHSFW_3$ and $FHSFW_4$ were imposed to the same type loading as that of the original specimens W_3 and W_4. The seismic performance of both $FHSFW_3$ and $FHSFW_4$ was optimal. The damage of both subassemblages was concentrated only in the critical regions of their beams, while their columns and their joint regions were intact at the conclusion of the tests. This optimal performance was also clearly demonstrated in the hysteresis loops of both subassemblages

$FHSFW_3$ and $FHSFW_4$ (Figs. 6 and 7). The hysteresis loops of $FHSFW_3$ and $FHSFW_4$ were much better than the loops of FW_2 (see Figs. 4, 6 and 7). The latter indicate the serious and almost premature and extremely dangerous joint shear failure of the subassemblage FW_2, (see Fig. 4). It is worth mentioning that with the proposed new technique this joint shear failure was avoided. It is worth noting that despite the use of lower fiber volume fraction (1.0%) in the ultra high-strength steel fiber-reinforced concrete jacket of $FHSFW_4$ compared to that of subassemblage $FHSFW_3$ (1.5%) the seismic performance of $FHSFW_4$ was also excellent (see Fig. 7).



(a) Specimen SO_3

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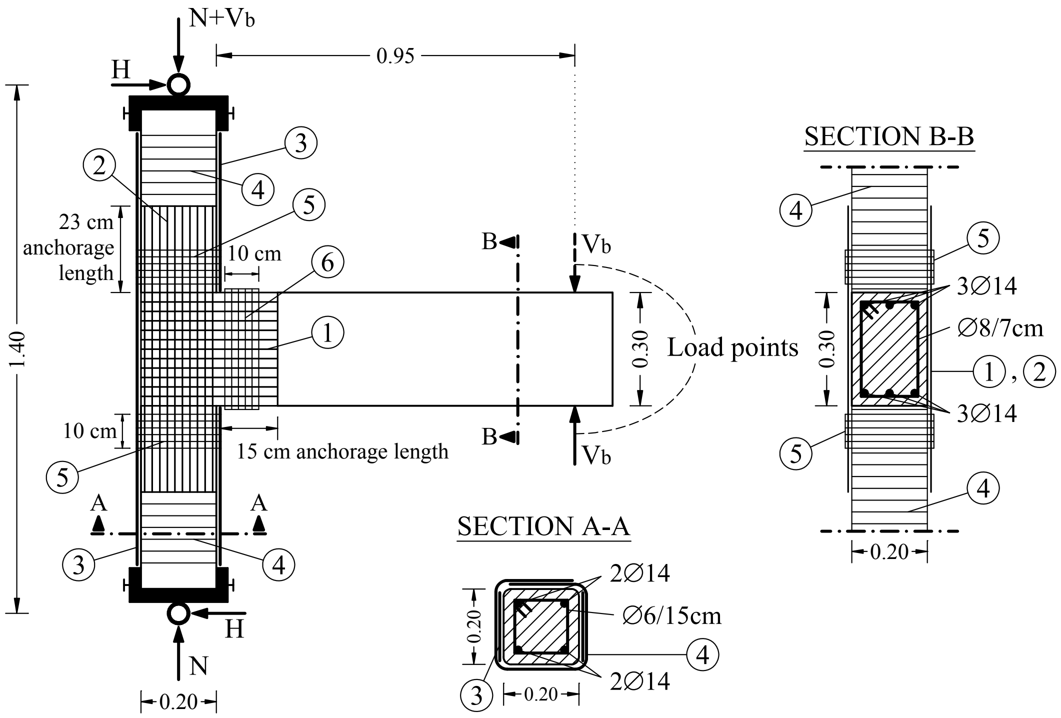

(1) 2 layers of CFRPs for increasing the horizontal shear strength of the joint

(2) 5 layers of CFRPs at the front side and 5 layers at the back side for increasing the vertical shear strength of the joint

(3) 5 layers of CFRPs for increasing the flexural strength of columns

(4) 2 layers of CFRPs for increasing the shear strength of columns

(5) 4 layers of CFRPs, 100mm in width, to prevent premature debonding of column strengthening layers

(6) 4 layers of CFRPs, 100mm in width, to secure the anchorage length of the joint layers

(b) Specimen FW_2

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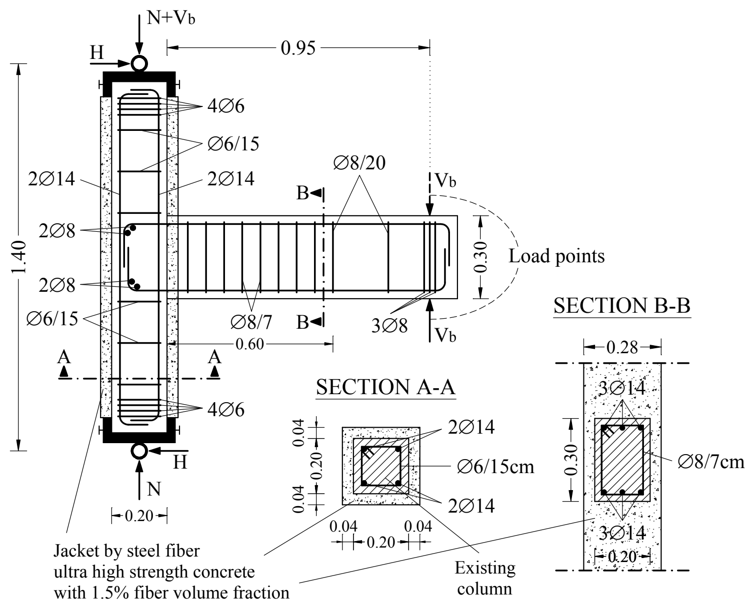
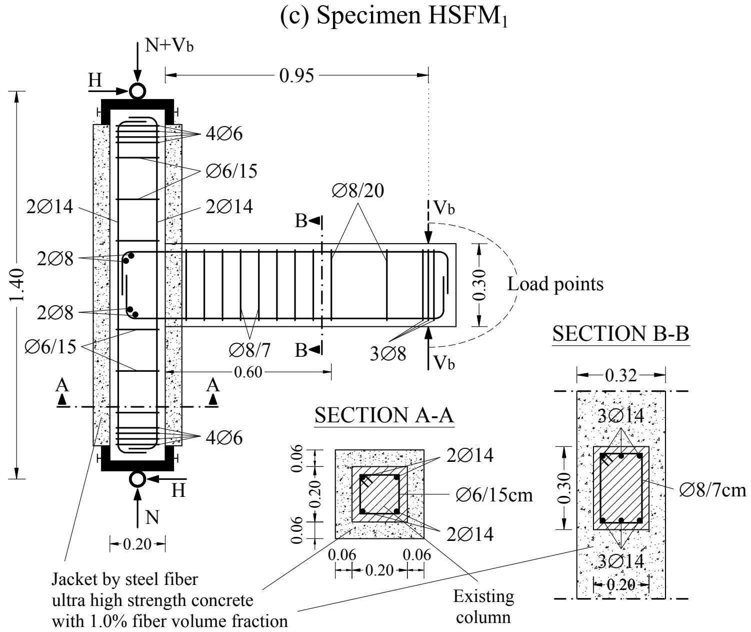
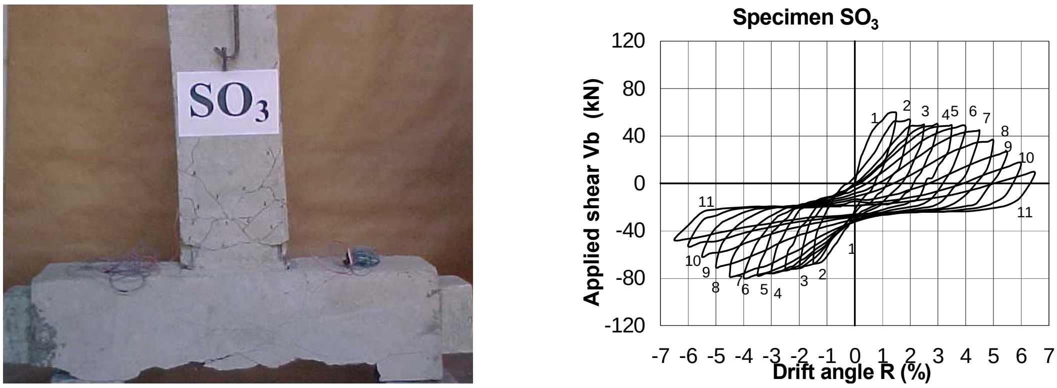


(c) Specimen $HSFM_1$

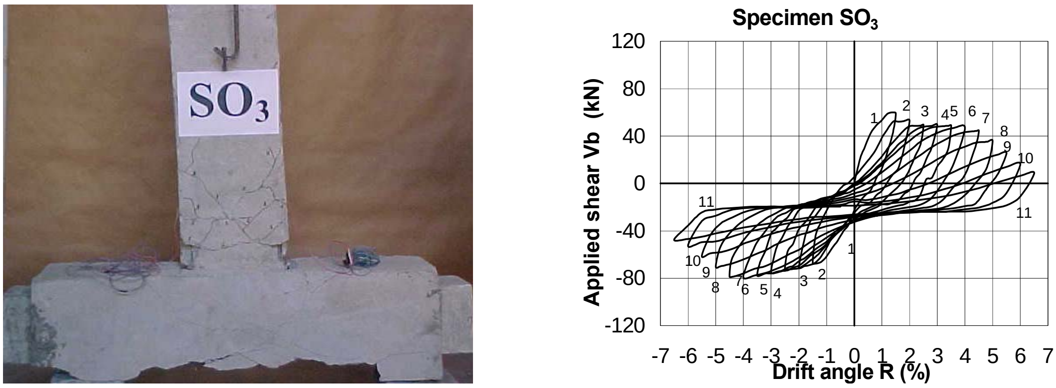


(d) Specimen $HSFM_3$

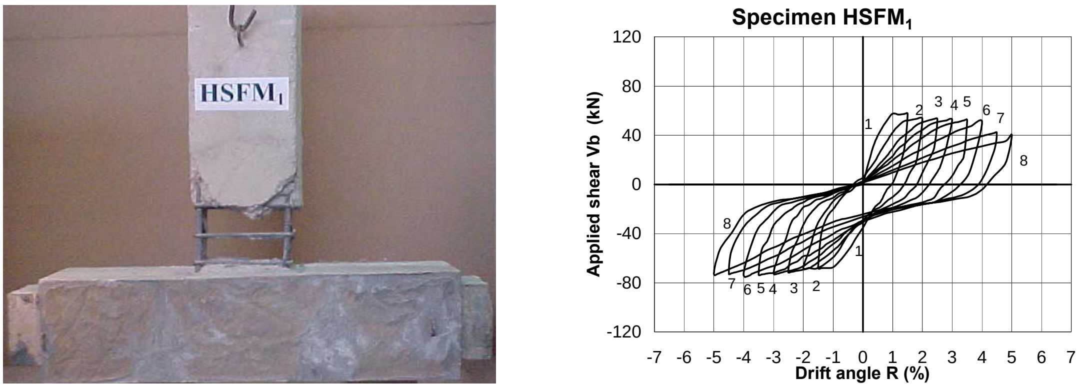
Fig. 3 Jacketing of column and beam-column connection of subassemblages (a) SO_3, (b) FW_2, (c) $HSFM_1$ and (d) $HSFM_3$



(a) Specimen SO_3



(b) Specimen FW_2



(c) Specimen $HSFM_1$

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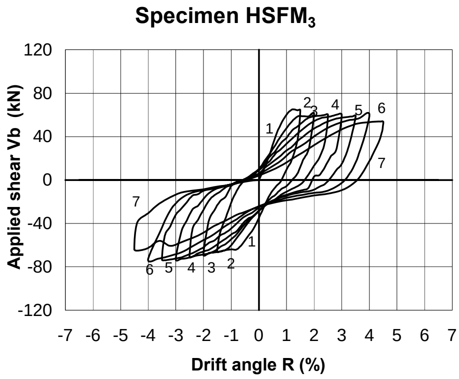
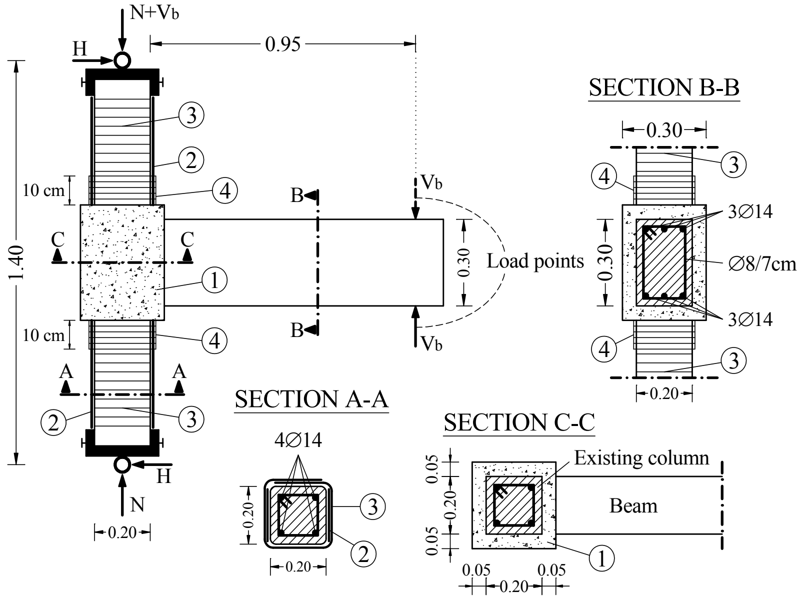


(d) Specimen $HSFM_3$

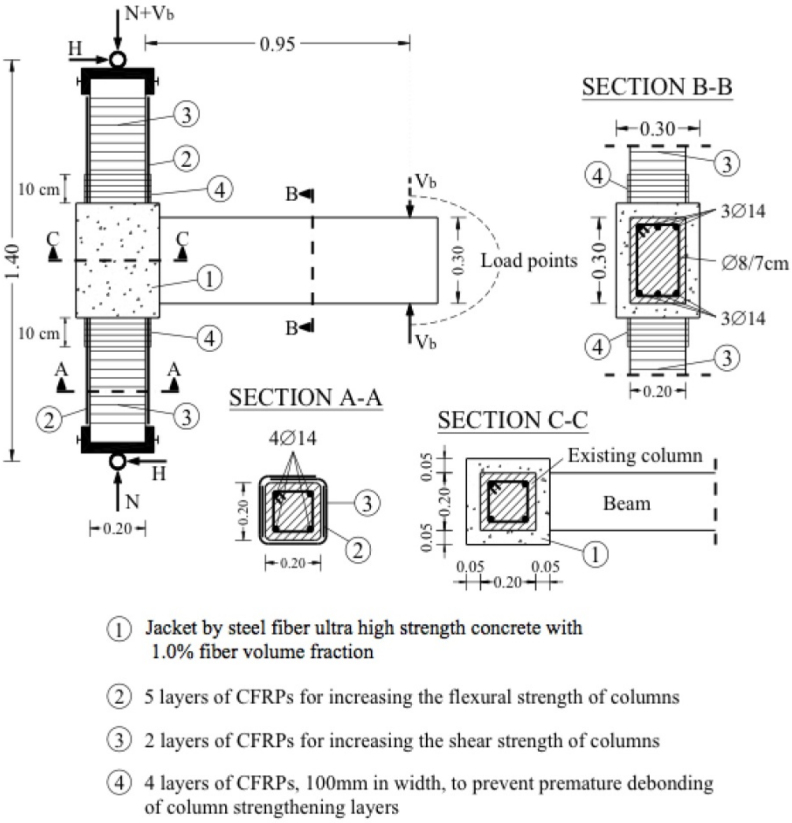
Fig. 4 Plots of applied shear versus drift angle and failure mode of the strengthened subassemblages SO_3, FW_2, $HSFM_1$ and $HSFM_3$



1. Jacket by steel fiber ultra high strength concrete with 1.5% fiber volume fraction
2. 5 layers of CFRPs for increasing the flexural strength of columns
3. 2 layers of CFRPs for increasing the shear strength of columns
4. 4 layers of CFRPs, 100mm in width, to prevent premature debonding of column strengthening layers

(a)

Continued-



(b)

Fig. 5 Strengthening of column and beam-column connection of (a) subassemblage $FHSFW_3$ and (b) subassemblage $FHSFW_4$ by the new mixed type technique

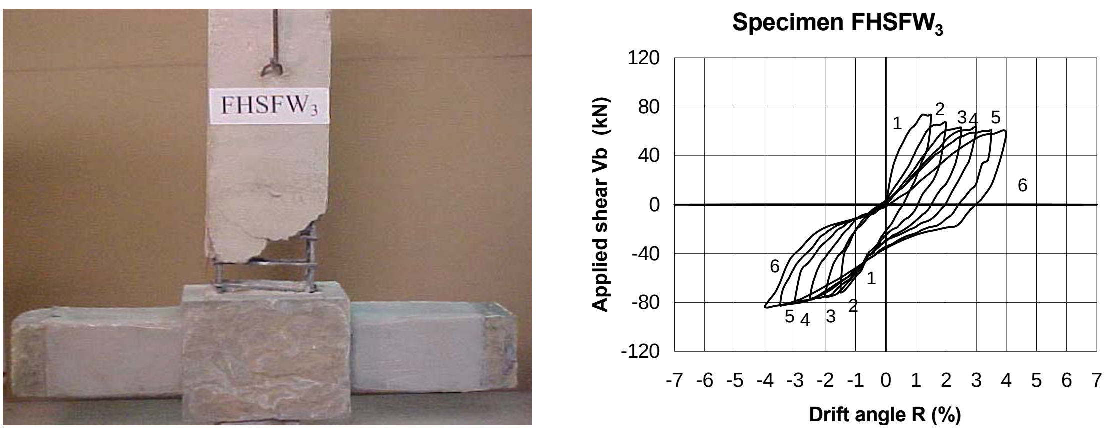


Fig. 6 Plots of applied shear versus drift angle and failure mode of the strengthened subassemblage $FHSFW_3$

Fig. 7 Plots of applied shear versus drift angle and failure mode of the strengthened subassemblage $FHSFW_4$

4. Conclusions

- A new innovative technique for strengthening of poorly detailed structural members of old buildings is proposed for the first time. This method consists of jacketing the structural members with non-shrink, non-segregating steel fiber concrete of ultra high-strength, without the addition of conventional reinforcement in the jackets.
- This new innovative method was found to be much more effective than the conventional reinforced concrete jackets and especially the FRP-jackets.
- Beam-column subassemblages, which had failed in pure joint shear failure during seismic-type loading and upgraded in the columns and beam-column joint region by the new innovative technique (patent No 1005657/2007) demonstrated the optimal failure mode, with damage concentration only in the beam region during re-loading with the same loading
- A second innovative solution is presented in this study also for the first time. This mixed type technique, by using local jacketing with steel fiber ultra-high-strength concrete only in the joint region, while the columns were upgraded by composite materials, eliminated the disadvantages of the application of composite materials FRPs for the strengthening of old building structures, due to the ineffective strengthening of beam-column joints by FRPs.

Acknowledgements

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