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A new method for earthquake strengthening of old R/C structures without the use of conventional reinforcement

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Abstract. In this study an 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 high-strength concrete jackets without conventional reinforcement which is usually applied in the construction of conventional reinforced concrete jackets (i.e., longitudinal reinforcement, stirrups, hoops). The proposed in this study innovative steel fiber high-strength or ultra high-strength concrete jackets were proved to be much more effective than the reinforced concrete jackets and the FRP-jackets when used for the earthquake-resistant strengthening of reinforced concrete structural members.

Keywords: steel fiber high-strength concrete; reinforced concrete jackets; 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 (Ilky *et al*. 1998, Karayannis *et al*. 2008, Rodriguez and Santiago 1998, Tsonos 2002, 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.

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

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recognised as alternate strengthening systems to conventional methods such as plate bonding and shotcreting (ACI 440-96, CEB-FIB 2006, Tsonos 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.

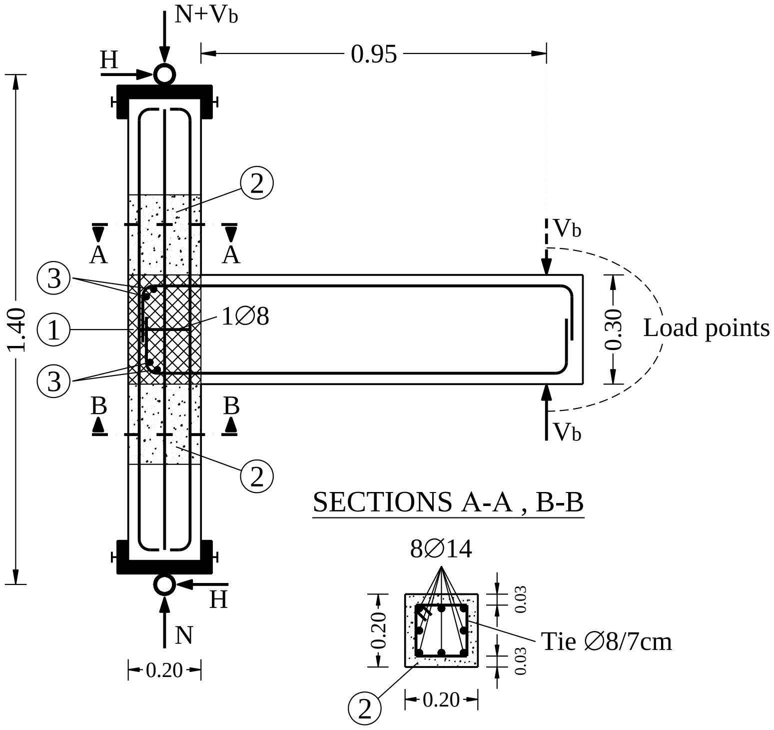
Henager (1977), successfully replaced all the hoops of the joint region and part of the hoops of the critical regions of the adjacent beam and column of an earthquake-resistant beam-column subassemblage, by steel fibers (1.67% fiber volume fraction is used). This replacement involved 50% reduction in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

An important experiment was conducted by Tsonos (2003). 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, Tsonos 2003). The joint shear stress of the specimen was higher than the maximum allowable joint shear stress by the same Committee



(1) removal and replacement of the crushed concrete in the joint region with steel fiber high strength concrete with 0.5% fiber volume fraction

(2) removal and replacement of the damaged concrete cover of part of the columns' critical regions with the same steel fiber high strength concrete

(3) the provision of transverse reinforcement, made of short bars placed and tightly connected under the bends of a group of rebars, was made to ensure the anchorage of the beam bars in the joint region

Fig. 1 Details of repaired specimen RL_3

(τ_{joint}=1.36 $\sqrt{f'_c}$ MPa>$\tau_{permitted}$=1.0 $\sqrt{f'_c}$ MPa). 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 (see Fig. 1), 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.

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 without conventional reinforcement, longitudinal bars or hoops. For this purpose, it was decided to increase the fiber volume fraction from 0.5% to 1%. The experimental results showed that the proposed new type of jacketing by steel fiber high-strength concrete with 1% fiber volume fraction was as effective as the other two types of retrofitting by reinforced shotcrete jacket and by FRP-jacket. A cost reduction of the order of 50% was computed in the application of the new proposed intervention scheme compared to the cost of application of reinforced shotcrete jacket. The compressive and tensile strengths of the non-shrink, non-segregating steel fiber concrete used, were 66MPa and 8MPa respectively.A patent GR 1005657/2007 was awarded to Professor Tsonos by the Greek Industrial Property Organization for the above invention.

In order to increase the effectiveness of the proposed new type of retrofitting it was decided to orient the research to two distinct directions:

a. To increase the steel fiber volume fraction in the non-shrink, non-segregating steel fiber concrete of high-strength used for the construction of the innovative jackets, and

b. To increase the compressive strength of the high-strength fiber reinforced concrete from 70MPa to 120MPa (final aim is to increase the strength above 150MPa) and to increase also the tensile strength of the steel fiber concrete.

The application of steel fiber high-strength concrete jackets without conventional reinforcement for strengthening of poorly detailed members of old RC structures is presented in this study for the first time. However, the use of steel fibers in deficient RC members, mainly beams, has already been studied in the past decades and was found to be beneficial since it increases strength and stiffness, reduces deflections and improves cracking characteristics. Further, it was observed that under some circumstances it can transform failure modes from brittle and dangerous shear failures to more ductile flexural failures (Karayannis 2000a, b). These advantageous observations inspired investigators to study the possibility of fully or partially replacing of steel stirrups with steel fibers, especially in cases where design criteria recommend a high steel ratio that leads to short stirrup spacing (Chalioris and Karayannis 2009, Chalioris and Sfiri 2011).

3. Improvement of the effectiveness of the proposed new method

A large experimental program was organized. Seven 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 40cm×40cm 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. 2 are shown the dimensions and cross-sectional details of these specimens O_3, W_2, M_1, M_2, M_3, M_4 and M_5. 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 approximately $2.10\sqrt{f_c'}$ 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 9.00MPa. 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. 4 is shown the failure mode of the representative specimen O_3 and its hysteresis loops. The failure of O_3 was concentrated mainly in

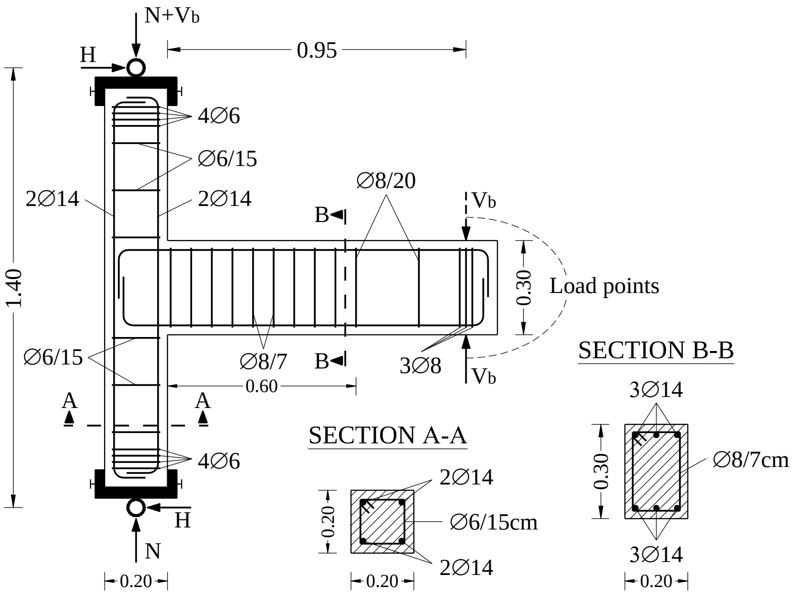


Fig. 2 Dimensions and cross-sectional details of original subassemblages O_3, W_2, M_1, M_2, M_3, M_4 and M_5

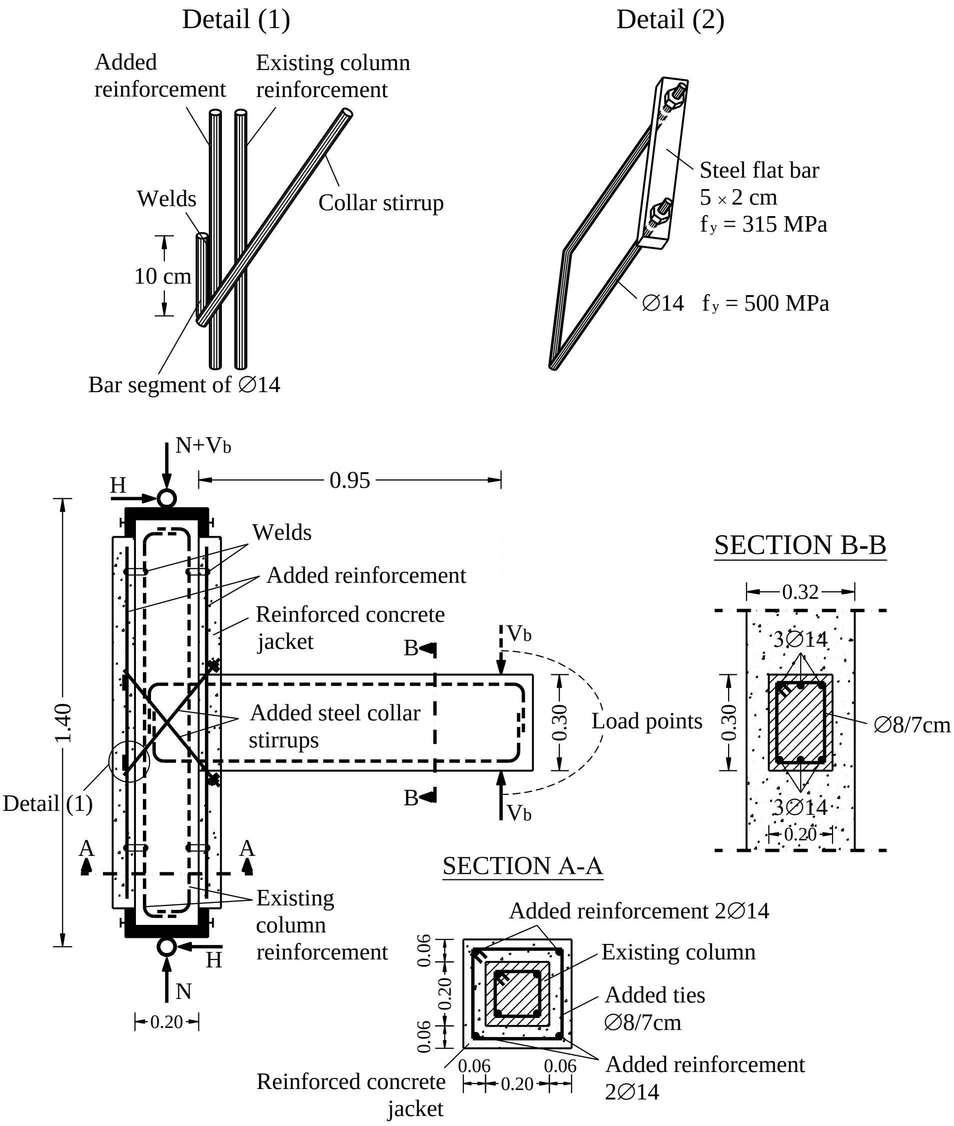
Table 1 Flexural strength ratio M_R, joint shear stress factor γ and joint shear stress τ_{jh} of subassemblages O_3, W_2, M_1, M_2, M_3, M_4, M_5, SO_3, FW_2, $UHSFM_1$, $UHSFM_2$, $UHSFM_3$, $HSFM_4$ and $HSFM_5$

Specimen	M_R [(1)]	γ [(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
M_1	0.98 (1.20)	2.22 (1.00)	6.41
M_2	0.98 (1.20)	2.21 (1.00)	6.41
M_3	0.98 (1.20)	2.18 (1.00)	6.41
M_4	0.96 (1.20)	2.04 (1.00)	6.45
M_5	0.95 (1.20)	2.04 (1.00)	6.45
SO_3	2.66 (1.20)	0.45 (1.00)	1.28
FW_2	1.55 (1.20)	2.04 (1.00)	6.43
$UHSFM_1$	1.13 (1.20)	0.31 (1.00)	0.89
$UHSFM_2$	1.40 (1.20)	0.24 (1.00)	0.70
$UHSFM_3$	1.40 (1.20)	0.25 (1.00)	0.74
$HSFM_4$	1.40 (1.20)	0.24 (1.00)	0.75
$HSFM_5$	1.40 (1.20)	0.24 (1.00)	0.75

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

the joint which lost almost all of the core's concrete since the shear forces acting in the beam-column joints are significantly higher than those acting in their adjacent columns (Paulay and Priestley 1992).

In the following are described in brief the retrofitting works for specimens O_3, W_2, M_1, M_2, M_3, M_4 and M_5.

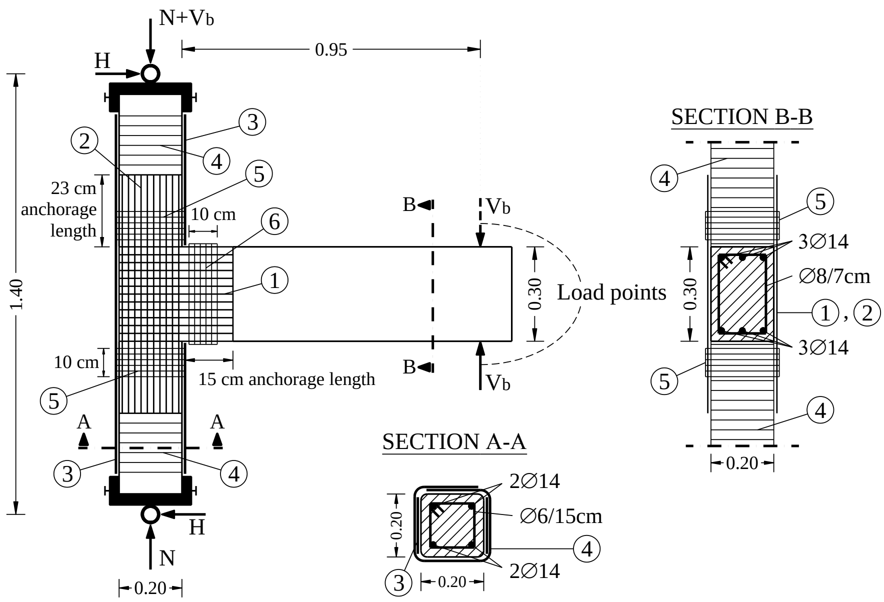


Specimen SO_3

Fig. 3 Jacketing of column and beam-column connection of subassemblages SO_3, FW_2, $UHSFM_1$, $UHSFM_2$, $UHSFM_3$, $HSFM_4$,and $HSFM_5$

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 as SO_3. In Fig. 3 are shown the dimensions and cross-sectional details of the SO_3.

1. 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 design for the retrofit process with carbon fiber-reinforced polymer sheets



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

Specimen FW_2

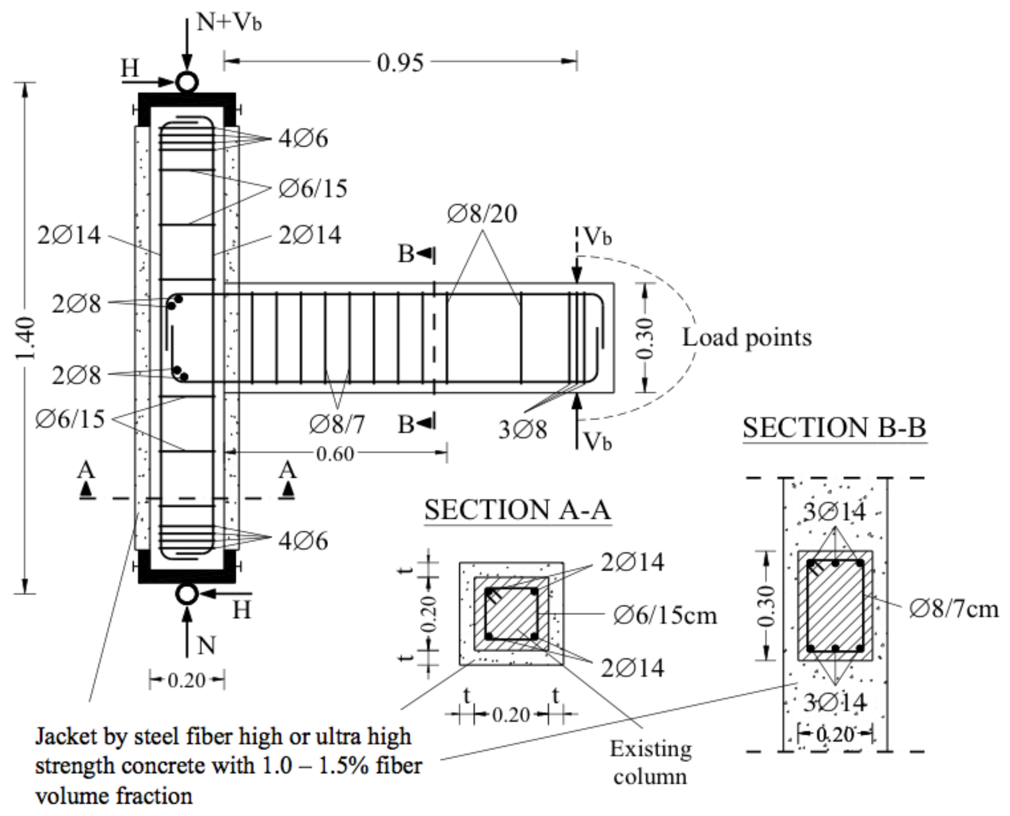
Fig. 3 Continued

Table 2 Details of strengthened subassemblages $UHSFM_1$, $UHSFM_2$, $UHSFM_3$, $HSFM_4$, $HSFM_5$

Specimen	Thickness of the jacket (mm)	Compressive strength of steel fiber concrete (MPa)	Tensile strength of steel fiber concrete (MPa)	Fiber volume fraction (%)
$UHSFM_1$	40	106.33	12.20	1.5
$UHSFM_2$	60	106.33	12.20	1.5
$UHSFM_3$	60	102.30	11.90	1.0
$HSFM_4$	60	65.00	7.80	1.5
$HSFM_5$	60	55.00	6.50	1.5

(CFRPs) was based on E_f=235 GPa, t_f=0.11 mm (t_f=layer thickness) and ε_{fu}=1.5% (ε_{fu}=ultimate FRP strain).The repaired and subsequently strengthened specimen was named FW_2.

2. Subassemblage M_1 was strengthened by jacketing with ultra high-strength steel fiber-



Specimens $UHSFM_1$, $UHSFM_2$, $UHSFM_3$, $HSFM_4$, and $HSFM_5$

Fig. 3 Continued

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 $UHSFM_1$ (see Fig. 3).

3. Subassemblage M_2 was retrofitted by jacketing with UHSFC with 1.5% fiber volume fraction, in the columns and in the joint region. The thickness of the jacket was 6.0 cm (Table 2). The repaired and strengthened specimen was named $UHSFM_2$ (see Fig. 3).

4. Subassemblage M_3 was retrofitted in the same way as specimen M_2, but the fiber volume fraction was 1% (Table 2). Specimen M_3 after the interventions was named $UHSFM_3$ (see Fig. 3).

The compressive strengths of the UHSFC used for the strengthening of $UHSFM_1$, $UHSFM_2$ and $UHSFM_3$ were 106.33 MPa, 106.33 MPa and 102.30 MPa respectively. The tensile strengths were approximately 12.00 MPa (Table 2). The characteristic toughness indexes I_{20} according to ASTM-C1018 for the ultra high-strength steel fiber-reinforced concrete (UHSFC) of specimens $UHSFM_1$, $UHSFM_2$ and $UHSFM_3$were approximately 12.50. The steel fibers used were Dramix ZP30/0.6.

5. Subassemblage M_4 was retrofitted by jacketing with steel fiber reinforced concrete of high-strength with 1.5% fiber volume fraction, in the columns and the joint region (see Fig. 3). The thickness of the jacket was 6.0 cm. The compressive strength of the steel fiber high-strength concrete used was 65 MPa. The tensile strength was 7.80 MPa. (Table 2). The steel fibers used

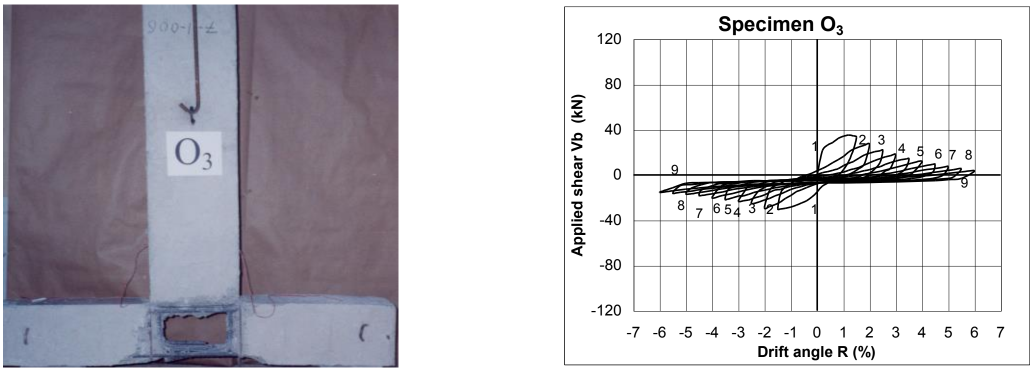


Fig. 4 Plots of applied shear versus drift angle and failure mode of the original subassemblage O_3

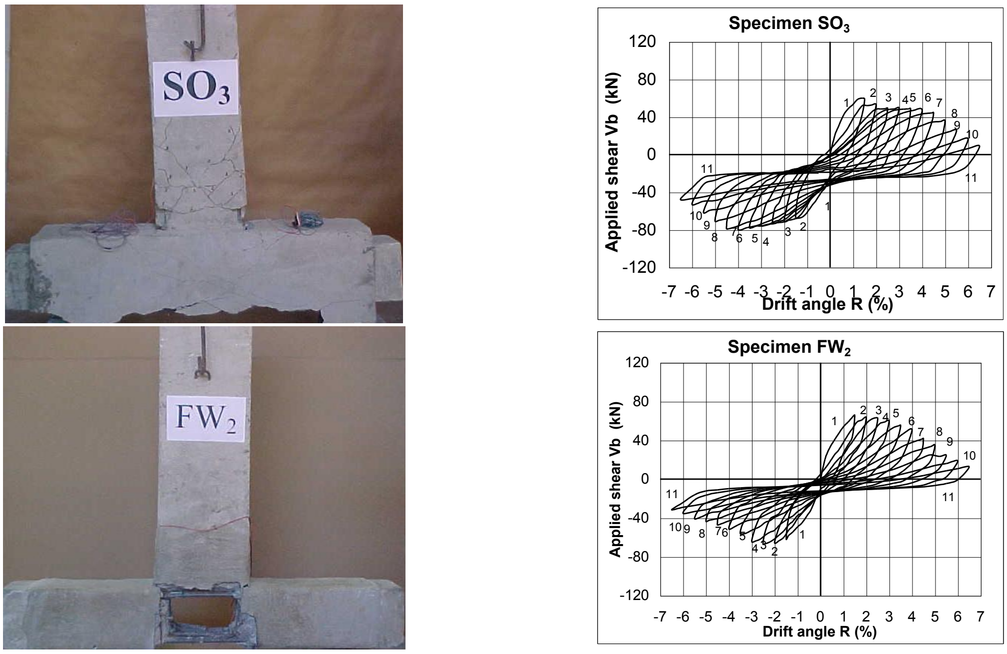


Fig. 5 Plots of applied shear versus drift angle and failure mode of the strengthened subassemblages SO_3, FW_2, $UHSFM_1$, $UHSFM_2$, $UHSFM_3$, $HSFM_4$ and $HSFM_5$

were also Dramix ZP30/0.6. The subassemblage after the interventions was named $HSFM_4$.The characteristic toughness index I_{20} according to ASTM-C1018 for the high-strength steel fiber-reinforced concrete (HSFC) of specimen $HSFM_4$ was 6.5.

6. Subassemblage M_5 was retrofitted by jacketing with steel fiber reinforced concrete of high-strength with 1.5% fiber volume fraction, in the columns and the joint region (see Fig. 3). The thickness of the jacket was 6.0cm. The compressive strength of the steel fiber high-strength concrete used was 55MPa. The tensile strength was 6.50 MPa. (Table 2). The steel fibers used were also Dramix ZP30/0.6. The subassemblage after the interventions was renamed $HSFM_5$. The characteristic toughness index I_{20} according to ASTM-C1018 for the high-strength steel fiber-reinforced concrete (HSFC) of specimen $HSFM_5$ was 5.80.

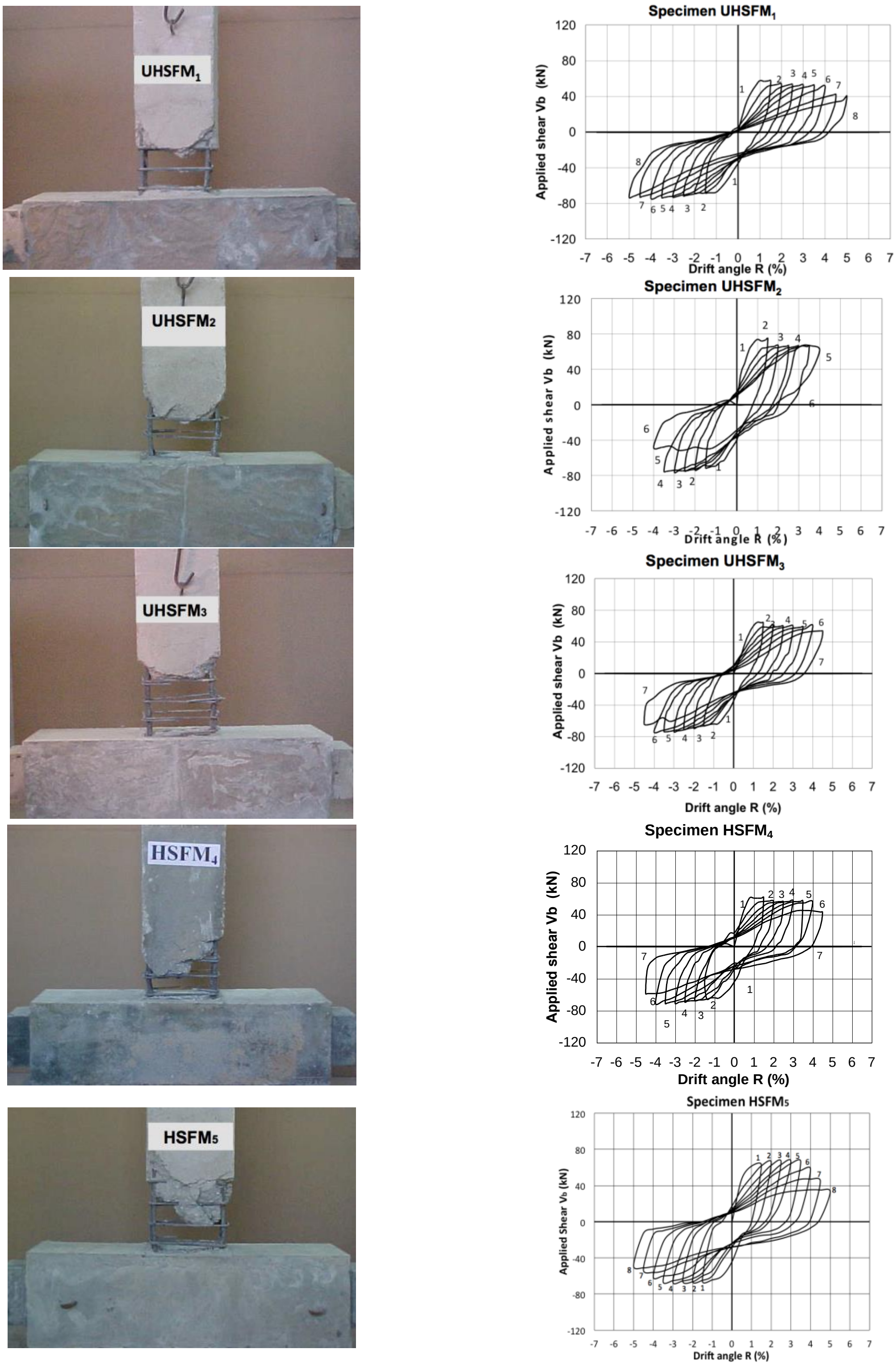


Fig. 5 Continued

All the above strengthened subassemblages SO_3, FW_2, $UHSFM_1$, $UHSFM_2$, $UHSFM_3$, $HSFM_4$, and $HSFM_5$ 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. 4-5 e.g., O_3 - $UHSFM_1$). However, the failure mode of SO_3 and FW_2 was quite different from that of all upgraded specimens by the new proposed jackets $HSFM_i$. 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. 5. On the contrary, the failure mode of all specimens $UHSFM_1$, $UHSFM_2$, $UHSFM_3$, $HSFM_4$, and $HSFM_5$ was the optimum one. Formation of a 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 all these specimens were intact at the conclusion of the tests. This excellent seismic performance of all the $UHSFM_1$, $UHSFM_2$, $UHSFM_3$, $HSFM_4$, and $HSFM_5$subassemblages was demonstrated both in their failure modes and in their hysteresis loops (Fig. 5). The seismic behaviour of all these subassemblages was superior to those of specimens SO_3 and FW_2 retrofitted by reinforced concrete jackets and FRP-jackets. It is worth noting that the seismic performance of subassemblage $HSFM_5$ constructed with lower strength steel fiber jacket compared to that of all the other subassemblages $UHSFM_1$, $UHSFM_2$, $UHSFM_3$, HSFM4and with fiber volume fraction 1.5% was also optimum (see Fig. 5), which clearly demonstrates the high effectiveness of the new proposed technique in strengthening of old reinforced concrete structures.

4. Conclusions

• An innovative method for strengthening of poorly detailed structural members of old buildings is proposed for the first time (patent GR 1005657/2007). This method consists of jacketing the structural members with non-shrink, non-segregating steel fiber concrete of high or ultra high-strength, without the addition of conventional reinforcement in the jackets.

• This 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 innovative method (patent GR 1005657/2007) demonstrated the optimal failure mode, with damage concentration only in the beam region during re-loading with the same loading.

• Calculations showed that the innovative method has significantly lower cost than the other well known methods.

• The future development of the proposed method targets at the use of non-shrink, non-segregating steel fiber concrete of ultra high compressive strength, above 200MPa, and with low steel fiber volume on the order of 0.5%.

Acknowledgements

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