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Earthquake-resistant rehabilitation of existing RC structures using high-strength steel fiber-reinforced concrete jackets

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Abstract. The effectiveness of an innovative method for the earthquake-resistant rehabilitation of existing poorly detailed reinforced concrete (RC) structures is experimentally investigated herein. Eight column subassemblages were subjected to earthquake-type loading and their hysteretic behaviour was evaluated. Four of the specimens were identical and representative of columns found in RC structures designed in the 1950s-70s period for gravity load only. These original specimens were subjected to cyclic lateral deformations and developed brittle failure mechanisms. Three of the damaged specimens were subsequently retrofitted with innovative high-strength steel fiber-reinforced concrete (HSSFC) jackets. The main variables examined were the jacket width and the contribution of mesh steel reinforcement in the seismic performance of the enhanced columns. The influence of steel fiber volume fraction was also examined using test results of a previous work of Tsonos *et al*. (2017). The fourth earthquake damaged subassemblage was strengthened with a conventional RC jacket and was subjected to the same lateral displacement history as the other three retrofitted columns. The seismic behaviour of the subassemblages strengthened according to the proposed retrofit scheme was evaluated with respect to that of the original specimens and that of the column strengthened with the conventional RC jacket. Test results clearly demonstrated that the HSSFC jackets effectively prevented the development of shear failure mechanisms, while ensuring a ductile seismic response similar to that of the subassemblage retrofitted with the conventional RC jacket. Ultimately, an indisputable superiority in the overall seismic performance of the strengthened columns was achieved with respect to the original specimens.

Keywords: columns; high-strength steel fiber-reinforced concrete; retrofit; RC jacket

1. Introduction

From historical records of strong and catastrophic earthquakes around the globe it is evident that excessive damage and collapse of RC structures result from significant structural deficiencies, related to the design conception of pre-1970s RC structures. In particular, the most common reasons for the premature loss of the column integrity and axial load carrying capacity were found to be the inadequacy of flexural strength and ductility, the low shear strength and the unreliable column flexural strength due to the deficient length of lap splices, which were located in the potential plastic hinge region (Priestley *et al.* 1996, Kalogeropoulos and Tsonos 2019). Moreover, the use of plain steel bars and widely spaced transverse reinforcement (with ninety-degree hook ends), regardless of the column size and shear force value, results in rapid degradation of the column overall seismic performance. Furthermore, the structural integrity is significantly affected by the detrimental impact of the cyclic inelastic lateral loading, which cause excessive damaging of the columns during strong earthquakes (Saadatmanesh *et al. 1996*). Therefore, it is essential that in earthquake prone areas the structures which are susceptible to developing brittle and premature failure mechanisms and excessive damage should be strengthened, to possess increased flexural and shear strength and adequate ductility during future strong seismic excitations (Chai *et al.* 1991).

The experience from earthquakes of the last sixty years contributed to the re-establishment of international Codes for the design of RC structures, based on the controllable and hierarchically developed damage control philosophy. Therefore, the damage of modern RC structures is limited to predetermined acceptable levels. Nevertheless, most of the existing RC structures were designed and constructed prior to the 1960-70s according to the recommendations of older design codes and thus, their seismic response is seriously jeopardized. Various retrofitting techniques and materials have been developed and used to mitigate the problems of columns found in existing RC structures and improve the seismic performance to satisfy the requirements of modern design codes (Eurocode 2 and 8 2004). The repair and strengthening methods usually take

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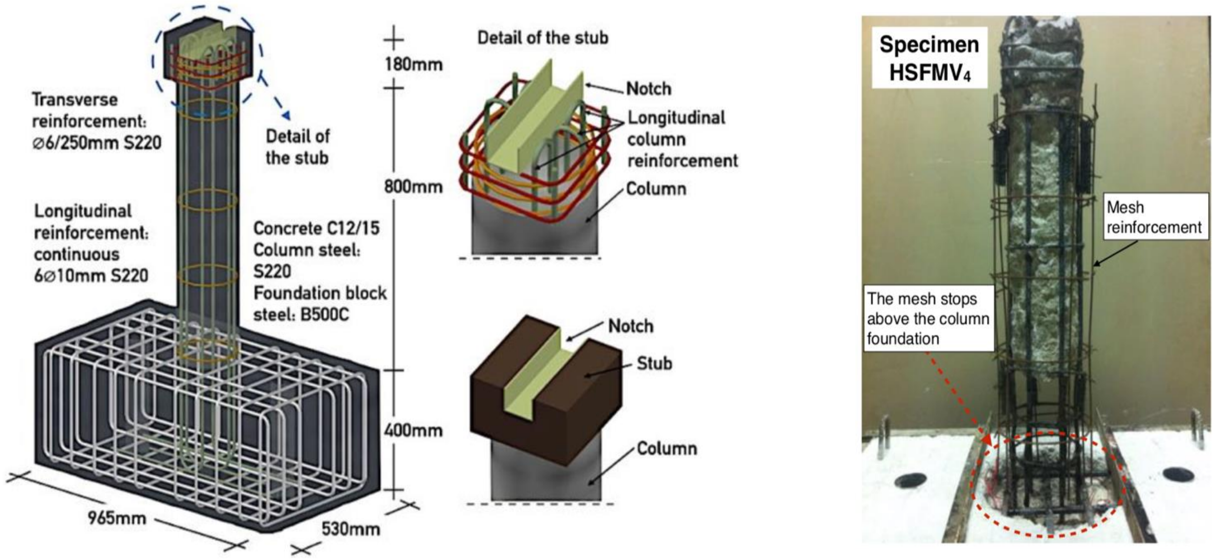
the form of various jackets applied to the structural elements (steel jackets; RC jackets; composite material jackets). Steel jacketing has been extensively used to offer additional confinement to the column and enhance the flexural ductility, shear strength and flexural strength (when affected by deficient lap splices in the critical region). Seismic tests on six large-scale columns (Chai *et al.* 1991) showed that steel jacketing of the columns results in column ductility as high as that available from confined columns designed to recent codes and prevents bond failure of the lap splices of the longitudinal reinforcement in the plastic hinge regions. Aboutaha *et al.* (1999) examined the effectiveness of rectangular solid steel jackets and partial steel jackets in the retrofitting of RC frame columns with inadequate shear resistance. Daudey and Filiatrault (2000) tested five 1:3.65-scale pier models of an existing bridge structure in the Montreal region, incorporating typical pre-1971 reinforcement details. One specimen was used as the original control specimen, while the other four were retrofitted by steel jacketing. The geometry of the jacket, the gap size at the base of the pier and the properties of the fill material between the jacket and the original cross-section were investigated, while a numerical model considering the bond-slip between the concrete and the longitudinal reinforcement was also proposed. El Gawady *et al.* (2010) investigated the seismic behaviour of reinforced concrete columns retrofitted by using carbon fibre reinforced polymer (CFRP) jackets and conventional steel jackets. The bond behavior between steel bars and concrete, when steel wrapping jackets were used as means of external confinement, was studied by Choi *et al.* (2013).

Alternatively to the steel-jacketing scheme, composite material jackets have also been used to improve the lateral performance of RC columns. Saadatmanesh *et al.* (1996) experimentally investigated the effectiveness of fibre reinforced polymers (FRP) composite jackets in either active or passive retrofitting of circular bridge columns. The composite straps were wrapped around the columns in the region of lap splices and/or the potential plastic hinge zone. Both active and passive retrofitting schemes were found to be effective in increasing the column earthquake resistance and preventing bond failure and buckling of the longitudinal reinforcement. The repair of earthquake-damaged columns with FRP wraps was also studied by Saadatmanesh *et al.* (1997). The repaired specimens showed higher flexural strength and displacement ductility, more stable hysteresis loops and lower rate of stiffness deterioration than the original columns. Pavese *et al.* (2004) used FRP strengthening solutions to improve the cyclic response of hollow RC bridge piers with structural deficiencies such as low shear strength, limited ductility and inadequate lap splices. Pampanin *et al.* (2007) experimentally and analytically investigated the efficiency of CFRP laminates in the retrofitting of poorly detailed existing RC buildings designed only for gravity loads (with beam bars anchored with endhooks in the joint, deficient lap splices with hooked anchorages and lack of joint transverse reinforcement), typical of the construction practice in the Mediterranean countries in the 1950s-1970s period. Two exterior knee joints, two exterior T-joints and two interior joint subassemblages were subjected to quasi-static cyclic loading, as well as a three-storey three-bay frame system, while a simplified analytical procedure was presented to evaluate the sequence of events using a M-N performance domain. The exterior CFRP strengthened subassemblages showed desirable ductile and dissipating hysteresis behaviour with the formation of the plastic hinges in the beam, while the interior joints exhibited an acceptable and controlled minor cracking in the joint panel zone. A partial retrofitting strategy using CFRP laminates was adopted in the case of the three-storey three-bay frame structure, which proved to be very satisfactory in improving the lateral behaviour and preventing brittle failure of the exterior joints and the formation of soft-storey mechanism. The use of CFRP sheets for the shear retrofit of circular and rectangular hollow bridge piers was also experimentally investigated by Yeh and Mo (2005). Karayannis and Sirkelis (2008) examined the behaviour of exterior beam-to-column joints strengthened with a combination of epoxy resin injections and carbon-fiber-reinforced plastics sheets. The enhanced specimens showed a significantly improved lateral performance with respect to the original specimens. A new category of FRP products, super laminates, was recently used for the production of seamless shells around existing columns (Ehsani 2010). The jacket of super laminates has continuous fibers in both hoop and longitudinal axis of the column, while the annular space can be filled with expansive grout or resin and, if desired, can be pressurized for improved confinement of the column.

A significant improvement of the overall seismic behaviour of the columns, including flexural and shear strength, ductility, stiffness and energy dissipation capacity, can be provided by RC jacketing. Although, footing retrofit measures should be undertaken in this case to ensure that plastic hinging develops in the column. In the experimental study of Julio and Branco (2008) the influence of interface treatment on the seismic performance of columns enhanced with reinforced concrete jackets was investigated. Tsonos (2008) also investigated experimentally the performance of the reinforced concrete jacket system and of the high-strength fiber-reinforced concrete jacket system in the cases of post-earthquake and pre-earthquake retrofitting of columns and beam-to-column joints. Both repair and strengthening techniques found to be effective. In particular, the RC jacket system found to be more effective in a post-earthquake retrofitting of columns and of beam-to-column joints than the high-strength fiber jacket system, while the two systems were equally effective in the case of pre-earthquake retrofitting. Kalogeropoulos and Tsonos (2014) experimentally investigated the effectiveness of RC jacketing of 1:1.5-scale columns with poor reinforcement detailing, typical of pre-1960 RC structures. The original columns had plain steel reinforcement, widely spaced transverse reinforcement and short lap splices located in the plastic hinge region. Test results clearly demonstrated that the retrofitted columns with welded lap splices and a RC jacket showed a substantially improved lateral behaviour with respect to the performance of the original specimens, similar to that of a retrofitted column with continuous longitudinal reinforcement without lap splices. Karayannis *et al*. (2008) used thin RC jackets to locally retrofit earthquake damaged beam-to-column joints. The



(a) Reinforcement details of the original specimens - Detail of the columns' stub (b) Specimen $HSFMV_4$

Fig. 1 Reinforcement details (a) of the original subassemblages and (b) of the strengthened subassemblage $HSFV_4$

strengthened specimens were retested and the experimental results indicated the success of the jackets in restoring and improving the lateral performance of the joint specimens. Kalogeropoulos *et al.* (2016) experimentally and analytically investigated the efficiency of pre-earthquake and post-earthquake reinforced concrete jacketing of beam-to-column joints, found in existing pre-1970 RC structures. They concluded that if the anchorage of the beam longitudinal reinforcement in the joint region is improved, then the retrofitted subassemblages perform very satisfactorily under a large number of cycles of inelastic lateral deformations. On the other hand, the retrofitted beam-to-column joint subassemblages show poor seismic response, similar to that of an original specimen, when the inadequacy of the beam reinforcement anchorage in the joint region is underestimated and no additional means are undertaken to improve bond between concrete and the steel bars.

A cost-effective earthquake strengthening system, equally effective as the other retrofit methods but simpler in application, with less time and labor demands, would have a competitive advantage over the others. Henager (1977) successfully replaced all the hoops in the beam-to-column joint region and part of the hoops in the critical regions of the adjacent beam and column of an earthquake-resistant beam-column subassemblage, with steel fibers (1.67% fiber volume fraction is used). This replacement resulted in 50% reduction in building costs. Another process developed by Hackman *et al*. (1992) called SIMCON (Slurry Infiltrated Mat Concrete) 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. Tsonos (2014) achieved a significant reduction of the number of ties in the beam-column joint region (5⌀8 to 1⌀8) using 0.5 percent by volume of steel fibers mixed in a non-shrinking high-strength concrete repair mix for the post-earthquake repair of exterior beam-column joints by the removal and replacement method. Noteworthy, throughout the duration of the earthquake-type loading cracking of the joint was prevented, while the damage and failure of the subassemblage were concentrated solely in the beam. The same type of HSSFC jacket was also used for the strengthening of the columns and the beam-column joint region of another specimen. In this case the percentage of steel fibers was slightly increased to 1.0 percent per volume to achieve greater efficiency. By evaluating the seismic behaviour of the aforementioned strengthened specimens it was concluded that the strengthening type applied was at least equally efficient as the conventional RC jacketing, while it was more efficient than the FRP jacketing. A new innovative technique was proposed for the first time by Tsonos (2007 patent No 1005657/2007, 2014) and uses non-shrink, non-segregating steel fiber concrete of ultra-high strength, without the addition of conventional reinforcement in the jackets for the strengthening of poorly detailed structural members of old buildings. The strengthened subassemblages performed very satisfactorily during the seismic loading, much more effectively than the specimens strengthened with conventional RC jackets and especially with FRP jackets. In particular, beam-to-column joint specimens that had failed in pure shear in the joint during the earthquake-type loading showed an optimal seismic behaviour when strengthened by the proposed innovative jackets and subjected to the same loading sequence to failure. The damage was concentrated in the beam only, while the joint region (which is one of the most critical regions of the structures) remained intact.

The present study presents an extensive experimental work carried out to investigate the effectiveness of an innovative retrofit method for the post-earthquake strengthening of seismically damaged existing old-type RC structures using high-strength steel fiber-reinforced concrete.

Table 1 Experimental program - original and retrofitted column subassemblages

	Original column specimens					Retrofitted column specimens				
Specimen	V_1*	V_2	V_3	V_4	V_5	$HSFV_1$*	$HSFV_2$	$HSFV_3$	$HSFMV_4$	CV_5
Strengthening scheme	-	-	-	-	-	High-strength steel fiber-reinforced concrete (HSSFC) jacket				RC jacket
Reinforcement of the original specimens and of the jackets	Continuous longitudinal plain bars: 6∅10mm S220 Transverse plain reinforcement: ∅6/250mm S220					No conventional steel reinforcement			Mesh T-131 ∅5mm/150mm	Longitudinal: 6∅10mm B500C Transverse: ∅8/80mm B500C spiral
Column diameter (mm)	200					300	320	300	300	300
Jacket width	-	-	-	-	-	50	60	50	50	50
(%) Steel fiber volume fraction	-	-	-	-	-	1.5	1.0	1.0	1.0	-
Concrete compressive strength (MPa)	11.89	12.94	11.27	13.34	13.11	63.50	58.45	63.39	66.22	28.78
Steel yield stress (MPa)	374 (longitudinal)/ 263.50 (transverse)					No conventional reinforcement				518

*V_1 and $HSFV_1$ were tested by Tsonos *et al.* (2017)

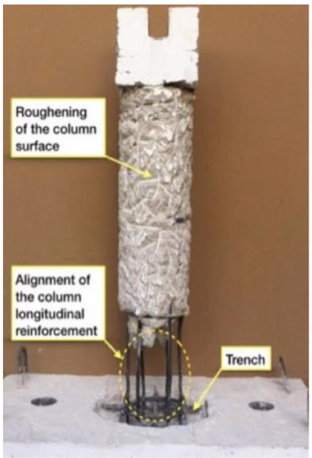
2. Experimental program

During strong seismic excitations of the last sixty years worldwide, numerous existing pre-1970s RC structures, showed poor overall hysteresis behavior and exhibited brittle failures which eventually resulted in catastrophic collapses. Moreover, it was clearly demonstrated that the design seismic forces may well be exceeded during strong earthquakes, with particularly detrimental impact in the column seismic performance, due to the unexpected excessive damage. Therefore, the seismic response, the control of damage and the collapse prevention of RC structures are related to both flexural/shear strength and ductility demands of the columns.

Along these lines it was considered of a particular interest to attempt to propose an innovative retrofit scheme for the post-earthquake strengthening of seismically damaged circular columns of existing RC structures. This scheme is cost-effective and easy to apply, while ensures the satisfactory ductile behaviour of the strengthened structure. The proposed strengthening system includes the use of high-strength steel fiber-reinforced concrete (HSSFC) for the jacketing of the column, while no conventional steel reinforcement is used. The main variables examined in this study were the width of the HSSFC jacket and the contribution of mesh steel reinforcement (which was not anchored in the column foundation block) in the seismic behaviour of columns strengthened with HSSFC jacket (see Table 1). Moreover, the experimental results from a previous work of Tsonos *et al.* (2017) of two seismically tested subassemblages, V_1 and $HSFV_1$, were also used to examine the influence of the steel fiber volume fraction in the seismic performance of the enhanced specimens. Details of specimens V_1 and $HSFV_1$ are presented in Table 1. Furthermore, the efficiency of the proposed strengthening scheme was evaluated with respect to the hysteretic response of a column specimen strengthened with a conventional RC jacket.

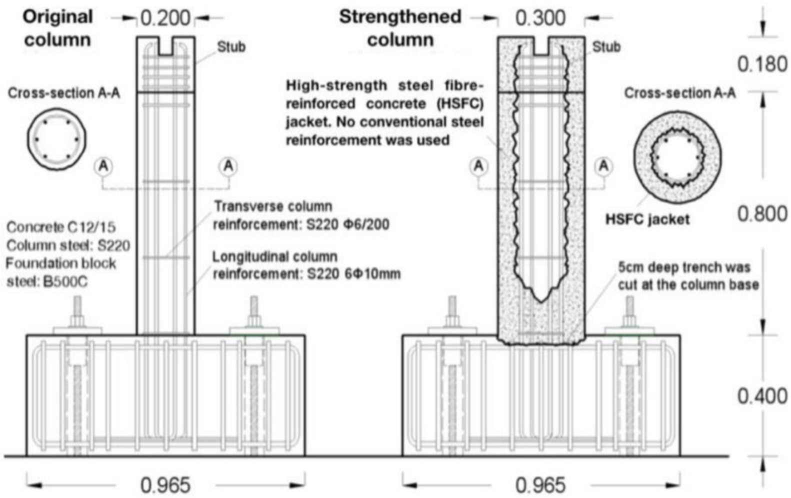
An extensive experimental program was conducted for eight cantilever column specimens with circular cross-section of approximately 1:1.5-scale. Four identical column subassemblages, V_2, V_3 V_4, and V_5, representative of structural members found in pre-1970 RC structures were designed and constructed (see Fig. 1(a)), with plain steel reinforcement (S220), transverse reinforcement with no hooks spaced at 250mm and normal weight concrete with low compression strength C12/15, measured by using 150x300mm cylinder compression tests (see Table 1). The column longitudinal reinforcing bars were continuous and well anchored in the strong foundation block of the specimens. Reinforcement details of the original column specimens are presented in Table 1. The columns were subjected to a large number of inelastic cyclic lateral displacements under constant axial loading of 150kN to simulate the equivalent of strong earthquake motions.

The longitudinal steel reinforcement of columns found in pre-1970s RC structures was usually lap-spliced inside the potential plastic hinge region. Moreover, the lap splices were designed for gravity loads only (under compression). Thus, the seismic performance of these columns was dominated by premature bond-slip failure and showed rapid and severe deterioration of strength and stiffness from the first cycles of the seismic loading sequence, due to the excessive slipping and pullout of the bars (Kalogeropoulos and Tsonos 2014, 2019). However, the original subassemblages examined in this study were intentionally designed to have continuous longitudinal steel reinforcement, in order to highlight the poor hysteresis behaviour of the existing columns, even when lap splices of the column longitudinal reinforcement were not located in the potential plastic hinge region. In particular, the absence of lap splices provided favourable load transferring conditions and restriction of bar slipping. Thus, the original specimens showed an optimal hysteresis (regarding columns found in pre-1970s RC structures), while post-yield strain values of the longitudinal steel bars were observed. Nevertheless, the subassemblages developed a brittle failure mechanism and eventually collapsed. Thus, it was clearly demonstrated experimentally that in the case of columns found in existing pre-1970s RC structures, strengthening interventions are necessary to prevent excessive damage and ensure the ductile seismic behaviour during future earthquakes.



(a) Roughening of the column surface and alignment of the buckled bars

(b) A 5cm deep trench was cut around each column in the foundation



(c) Construction of the HPSFC jackets

Fig. 2 (a), (b) Post-earthquake strengthening interventions applied to the column specimens, (c) Construction of the innovative HSSFC jacket

The original specimens were subjected to earthquake-type loading until failure. Subsequently, the seismically damaged columns were repaired, strengthened and re-tested according to the same lateral displacement history. Two different values of the HSSFC jacket width (50mm and 60mm) were examined, while the steel fiber volume fraction equaled to 1.00 percent (or 1.50 percent in the case of subassemblage $HSFV_1$). Four subassemblages were strengthened by HSSFC jacketing, one of which had additional mesh reinforcement. The fifth column specimen was enhanced with a conventional RC jacket designed according to the provisions of EC2 and EC8 (2004). The experimental program, material properties and reinforcement details of the original and strengthened specimens are shown in Table 1. The cyclic lateral performance of the original and the strengthened subassemblages was compared to evaluate the effectiveness of the retrofitting schemes applied to the columns.

3. Strengthening interventions applied to the damaged column subassemblages

The original column subassemblages, V_2, V_3 V_4, and V_5, were subjected to a large number of reversed incremental amplitudes of inelastic lateral displacement. All specimens exhibited brittle shear failure and developed excessive damage in the plastic hinge region at the lower part of the column. Eventually, the original columns collapsed due to the loss of axial load carrying capacity. Thereafter, repair and strengthening interventions were implemented to the damaged columns and the retrofitted specimens were re-tested according to the same lateral displacement history as the original ones.

The strengthening process included the following steps:

1. Using a jackhammer, the surface in the circumference of the columns was roughened, while a 5cm deep trench was cut around each column in the foundation block (see Figs. 2(a)-(b). The spalled concrete at the lower part of the original columns was subsequently removed.

2. The buckled longitudinal steel bars were realigned to their original position (see Figs. 2(a)-(b)).

3. Afterwards, the original subassemblages V_2 and V_3 were strengthened by pouring flowable, rheoplastic, non-shrink and non-segregating steel fiber-reinforced concrete

of high-strength into the formwork of the HSSFC jackets. The latter did not include additional conventional steel reinforcement (see Fig. 2(c)). In Table 1 the fiber volume fraction ratio and the width of the jacket applied to each column are presented. The steel fibers had tensile strength, $R_{m,nom}$, equal to 1270 N/mm, Young's modulus of ±210000 N/mm^2, length (l) of 30 mm, diameter (d) equal to 0.62mm and aspect ratio (l/d) equal to 45. The fourth original column, V_4, was strengthened with HSSFC jacket that had additional mesh steel reinforcement of Ø5/150 mm bars (see Fig. 1(b)). However, this mesh reinforcement was not anchored in the foundation block of the specimen, but only attached to the reinforcement bars of the original column. The compressive strength of the jackets is presented in Table 1. The post-earthquake strengthened columns were designated $HSFV_2$, $HSFV_3$ and $HSFMV_4$, respectively.

4. The retrofit process of the damaged original specimen V_5 included the construction of a RC jacket, designed according to the recommendations of modern design codes, EC2 and EC8 (2004). The post-earthquake strengthened column was designated CV_5. Six holes of diameter equal to 12mm were drilled in the foundation block of the specimen (see Figs. 3(a)-(b)). The holes were subsequently cleaned with air pressure. Thereupon, six longitudinal B500C steel bars of 10mm diameter each were inserted in the holes and anchored in the foundation block of the subassemblages. The anchorage of the bars was achieved by inserting epoxy resin into the holes with a syringe. The control of the concrete compression strut adequacy was made according to equation Eq. (2). In Table 2 the Eurocode provisions for the shear reinforcement are presented. The transverse reinforcement demand was finally calculated according to the expression Eq. (3), where it is recommended that $V_{Rd,s} \geq V_{sd}$. In Eqs. (1)-(3) $A_{s,ex}$ is the cross-section area of the existing column; $A_{s,j}$ is the cross-section area of the RC jacket; $f_{ck,ex}$ is the concrete characteristic strength of the existing column; $f_{ck,j}$ is the concrete characteristic strength of the RC jacket; f'_{ck} is the equivalent concrete characteristic strength of the retrofitted column; $V_{Rd,max}$ is the strength of the concrete compression struts; $V_{Rd,s}$ is the shear force resisted by the hoops; A_{sw} is the cross-section area of the transverse reinforcement; s is the spacing of the stirrups; f_{ywd} is the yield strength of the transverse reinforcement; b_w is the cross-sectional depth of the column; $z = 0.9 \cdot d$; a_{cw} is a coefficient taking account of the state of the stress in the compression strut; v_1 is the strength reduction factor for concrete cracked in shear with recommended value of $1 - f_{ck}(MPa)/250$ and θ is the angle between the concrete compression strut and the member axis perpendicular to the shear force ($\theta = 22°$). Ultimately, the transverse reinforcement of the RC jacket consisted of an 8 mm diameter B500C spiral with 80mm pitch along the critical column height, while C25/30 concrete was used for the construction of the RC jacket. The strengthened specimen was designated CV_5. The retrofitting interventions applied to CV_5 are shown in Fig. 3. Connection between the reinforcing bars of the RC jacket and the bars of the existing column was achieved using s-shaped (⌐◡) segments welded to the reinforcement. The welding was made according to Code for Steel Reinforcement Technology of Concrete (C.S.R.T.C.) (2008).

$$(A_{s,ex} \cdot f_{ck,ex} + A_{s,j} \cdot f_{ck,j})/(A_{s,ex} + A_{s,j}) = f'_{ck} \qquad (1)$$

$$V_{Rd,max} = a_{cw} \cdot b_w \cdot z \cdot v_1 \cdot f_{cd}/(\cot\theta + \tan\theta) \qquad (2)$$

$$V_{Rd,s} = (A_{sw}/s) \cdot z \cdot f_{ywd} \cdot \cot\theta \geq V_{sd} \qquad (3)$$

4. Reaction frame and loading sequence

The original and the retrofitted subassemblages were

(a) Six ∅12mm holes were drilled in the foundation

(b) The longitudinal bars of the RC jacket were anchored in the foundation block. The holes were filled with epoxy resin

(c) S-shaped (⌐◡) segments were used to connect the longitudinal reinforcement of the RC jacket with the bars of the existing column

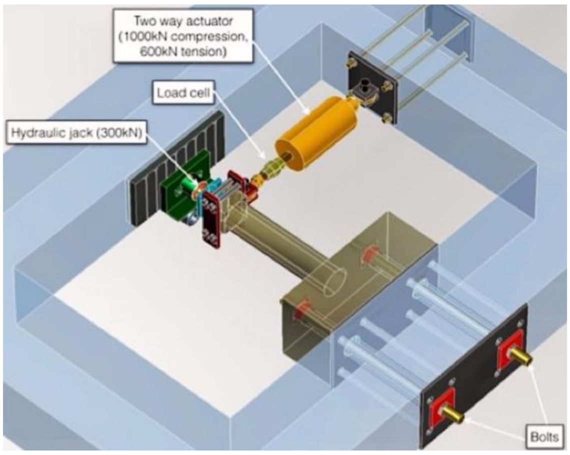
(d) Longitudinal and transverse (spiral) reinforcement of the RC jacket

Fig. 3 Strengthening interventions applied to column subassemblage CV_5

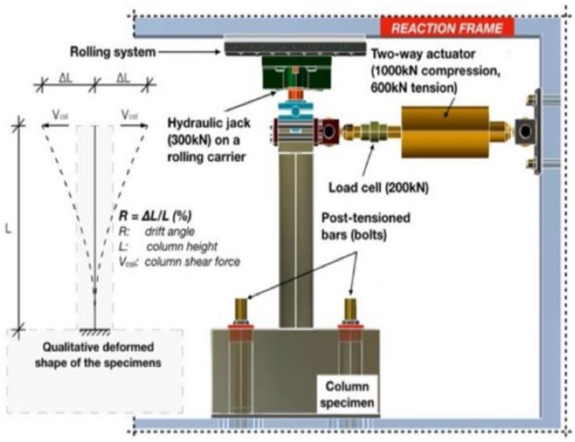
Table 2 Design of the RC jacket in shear (Eurocode 2 and 8)

V_{sd} [a] (kN)	$V_{RD,max}$ (kN)	$V_{Rd,s} \geq V_{sd}$ (kN)	Transverse reinforcement of the jacket
41.74	211.63>V_{sd}	For $V_{Rd,s} \geq V_{sd}$ s [b]≤450 mm 132.07>V_{sd}	∅8/80 mm spiral

[a] V_{sd}: Shear force corresponding to the moment resistance of the retrofitted column.
[b] s: Spacing of the spiral.



(a) Aerial view of the test setup



(b) Details of the reaction frame and the instrumentation used

Fig. 4 Test setup and qualitative deformed shape of the specimens

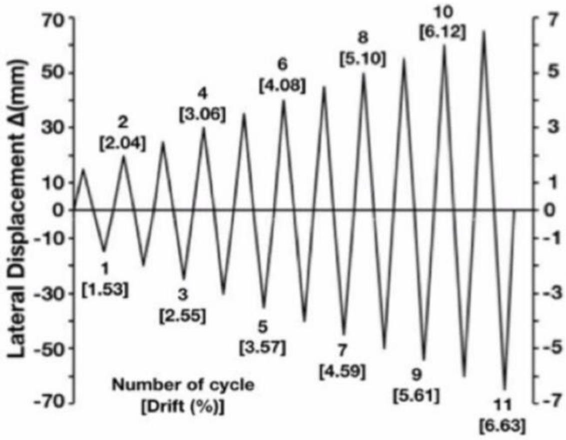


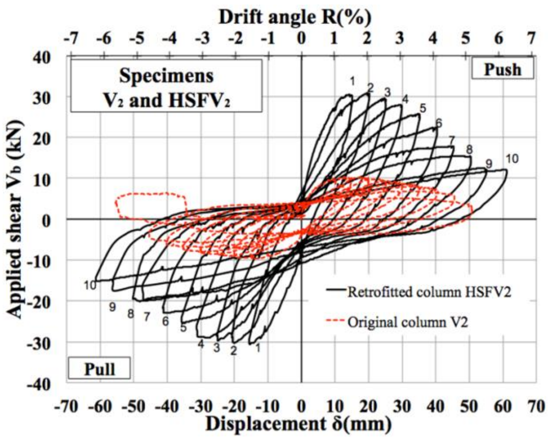
Fig. 5 Lateral displacement history

subjected to earthquake-type loading to simulate the equivalent effect of strong earthquakes. The columns were loaded transversely, under constant axial loading of 150kN. The seismic tests were conducted in the test setup shown in Fig. 4, which is located in the Laboratory of Reinforced Concrete and Masonry Structures of the Aristotle University of Thessaloniki. The structures were fixed to the test frame with post-tensioned bars (see Fig 4(a)), thus the horizontal and vertical displacement and the rotation of the foundation block of each column were restrained. A hydraulic jack, placed on top of the stub of each column perpendicular to the lateral loading direction, was used to impose the axial load to the specimens and controlled to keep constant during the tests. The lateral loading was applied with a two-way actuator by slowly displacing the column free end of the specimens. The shear resistance of the columns was measured by a load-cell, while a calibrated linear variable differential transducer was used to control the load point displacement. Electrical resistant strain gages were installed to the columns' longitudinal and transverse reinforcement to measure the steel strain values during the seismic loading and ascertain the yielding of reinforcement. The exact location of each strain gage is shown in Fig. 13.

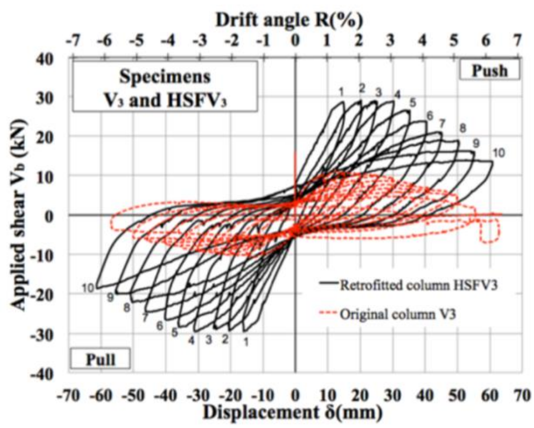
All specimens were loaded transversely following the displacement-controlled schedule shown in Fig. 5 (the original columns were subjected to fewer cycles of loading due to excessive damage and collapse). In Fig. 5 the correspondence of the top displacement amplitudes to the drift angles is depicted. The seismic loading sequence was established to capture critical issues of the element capacity, for instance the ultimate limit state of the column. Given that the inelastic cyclic deformations cause cumulative damage and that the behaviour of subassemblages is mainly demonstrated by the envelope curves, a loading sequence with constantly increasing lateral displacement per step and with one cycle per amplitude of displacement was adopted, without considerable influence in the seismic performance of the subassemblages. An original specimen was used to determine the steps of loading and was at first loaded to its yield displacement. This was measured from the plot of applied shear-versus-displacement of the specimen for the point when a significant decrease in stiffness occurred and was also verified by the yielding of the longitudinal column reinforcement. The loading was continued in the same direction (push cycles) to 1.5 times the yield displacement and the subassemblage was subsequently loaded in the other direction (pull cycles) to the same lateral displacement. After the first cycle of loading, the maximum displacement of each subsequent cycle was increased incrementally by 0.5 times the yield displacement (Hakuto *et al.* 2000, Ehsani and Wight 1985, Durani and Wight 1987). The original columns, V_2, V_3 V_4, and V_5 were subjected to eight, nine, eight and nine cycles of seismic loading respectively and performed similarly. In particular, they showed deterioration of the hysteretic response with the incremental displacement amplitudes of the lateral loading and loss of axial load carrying capacity. The retrofitted specimens, $HSFV_2$, $HSFV_3$, $HSFMV_4$ and CV_5 remained intact after ten cycles of lateral displacement.

5. Experimental results-Interpretation of the hysteretic performance of the specimens

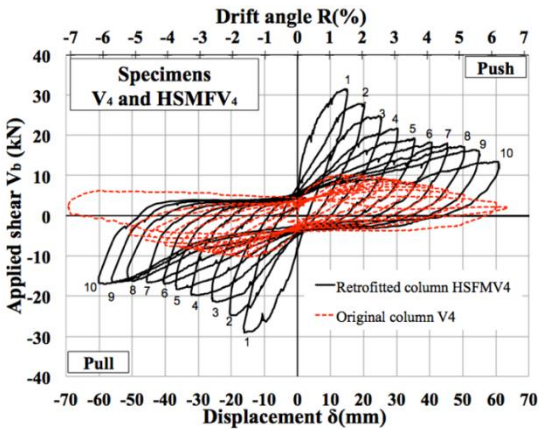
In the following, the overall seismic performance of the original and the strengthened column specimens is compared by means of lateral strength, peak-to-peak



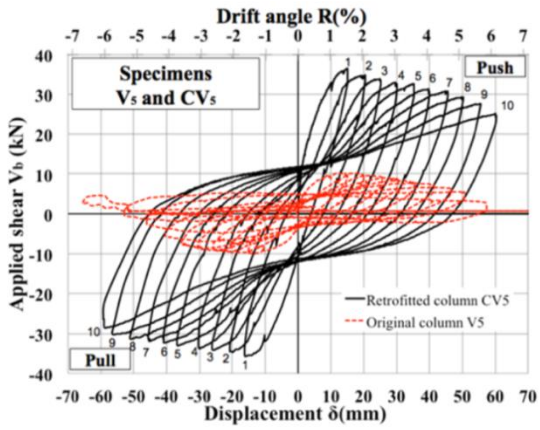
(a) Hysteresis loops of specimens V_2 and $HSFV_2$



(b) Hysteresis loops of specimens V_3 and $HSFV_3$



(c) Hysteresis loops of specimens V_4 and $HSFV_4$



(d) Hysteresis loops of specimens V_5 and CV_5

Fig. 6 (a-d) Plots of applied shear-versus-displacement of the original and the strengthened subassemblages

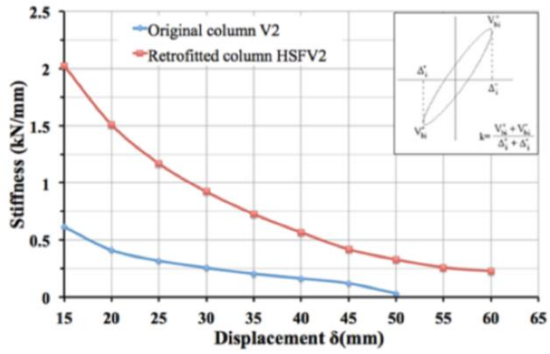
stiffness and energy dissipation capacity. Thus, the efficiency and suitability of the innovative retrofit scheme in improving the hysteretic behavior of the strengthened subassemblages are evaluated. Moreover, the influence of the jacket width and the contribution of the mesh reinforcement in the column cyclic lateral response were investigated. Furthermore, in order to investigate the influence of the steel fiber volume fraction in the column seismic performance, the experimental results of Tsonos *et al*. (2017) are also discussed and compared herein. The seismic behaviour of the subassemblages is clearly reflected in the hysteresis loops, illustrated in Fig. 6.

A progressive deterioration of the lateral strength of the original specimens was observed throughout the consecutive cycles of the earthquake-type loading. From the first cycle hairline flexural and shear cracks were formed in the plastic hinge region at the lower part of the column (see Fig. 9(a)). The increase in lateral drift angle, R, caused significant gradual dilation of these cracks. The cracking propagation is illustrated in (Fig. 9(a)-(b)). Meanwhile, the cyclic inelastic lateral deformations caused cumulative damage and rupture of the concrete in the compression zones. Therefore, the excessive disintegration of the core concrete in the plastic hinge region resulted in loss of both the aggregate interlock shear transfer mechanism and the dowel action of reinforcement. Ultimately, given that the original columns were poorly confined with hoops spaced at 250mm, the specimens exhibited brittle shear failure and loss of axial load carrying capacity. The original columns collapsed under gravity loads and buckling of the longitudinal reinforcement occurred (see Figs. 10 (a), (c), (e), (g)). The longitudinal steel bars of the original columns were continuous and well anchored in the foundation block of the specimens. Thus, the columns developed their flexural nominal moment capacity, while significant post-yield strain values of the longitudinal steel bars were measured using electrical resistant strain gages (see Fig. 12).

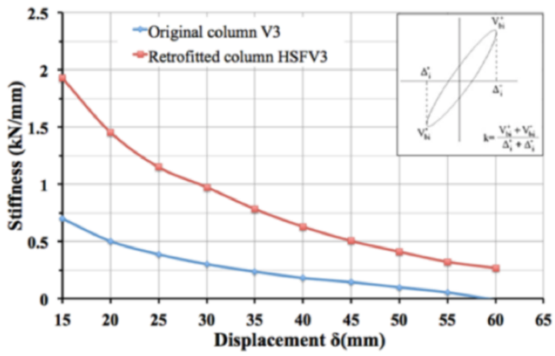
The post-earthquake strengthened subassemblages, $HSFV_1$(Tsonos *et al.* 2017), $HSFV_2$, $HSFV_3$ and $HSFMV_4$ exhibited superior overall seismic performance with respect to the corresponding original specimens, V_1(Tsonos *et al*. 2017), V_2, V_3 and V_4. In particular, the HSSFC jacket ensured the ductile behavior of the strengthened columns, while also effectively and substantially increased the lateral strength of the enhanced specimens. Moreover, the HSSFC jacketing provided adequate confinement to the plastic hinge region, thus, brittle shear failure of the columns, as well as buckling of the longitudinal steel bars were prevented. The failure mode of $HSFV_2$, $HSFV_3$ and $HSFMV_4$ clearly demonstrates the satisfactory ductile cyclic performance of the columns (see Fig. 6). At the end of the seismic tests the strengthened columns remained intact, while the damage was concentrated solely in the foundation block of the subassemblages (see Figs. 6, 10 (b), (d), (f), (h) and 11).

As aforementioned, the original columns developed their flexural nominal moment capacity. Nevertheless, the lateral strength of V_1, V_2, V_3, V_4 and V_5 was a mere portion of that of the corresponding post-earthquake strengthened subassemblages (see Fig. 6). For instance, during the first cycle of loading the lateral strength values (push/pull half cycles) of specimens $HSFV_1$, $HSFV_2$, $HSFV_3$, $HSFMV_4$ and CV_5 were 2.89/2.62, 3.25/3.37, 2.67/2.85, 3.06/2.87 and 3.61/3.67 times the values of the corresponding original columns. For drift angle, R, equal to 4.59 percent the strengthened subassemblages showed 354.60 (push half cycles), 317.93 (pull half cycles), 264.71 (pull half cycles), 173.52 (push half cycles) and 1041.20 (pull half cycles) percent increase in lateral strength with respect to specimens V_1, V_2, V_3, V_4 and V_5, respectively. For lateral drift angle, R, equal to 5.10 percent (specimen $HSFV_1$) and 6.12 percent (specimens $HSFV_2$, $HSFV_3$, $HSFMV_4$ and CV_5) the strengthened columns $HSFV_1$, $HSFV_2$, $HSFV_3$, $HSFMV_4$ and CV_5 maintained 61.55/64.71, 39.94/49.77, 47.78/62.78, 43.20/57.00, 68,73/74.77 percent (push/pull half cycles) of their initial strength values in the first cycle of the earthquake-type loading. Therefore, the exploitation of material properties and of the inherent strength of the HSSFC jacket resulted in a significant improvement in lateral strength of the retrofitted specimens with respect to the original ones. It is worth noting that the latter was achieved without using additional conventional steel reinforcement. A slight decrease in lateral strength of specimens $HSFV_1$, $HSFV_2$, $HSFV_3$ and $HSFMV_4$ was observed during the incremental displacement amplitudes of the seismic loading sequence. However, this results from the gradual rupture of the low-strength concrete (C12/15) of the foundation block in the interface with the HSSFC jacket at the circumference of the columns (see Figs. 10 and Fig. 11). Besides, the columns remained intact until the end of the seismic tests (see Fig. 10). Specimens $HSFV_1$, $HSFV_2$, $HSFV_3$ and $HSFMV_4$ showed similar values of lateral strength (see Fig. 6). The slightly lower strength values of $HSFMV_4$ resulted from the premature rupture of the mortar which was used to fill the gap in the column circumference between the jacket and the foundation block.

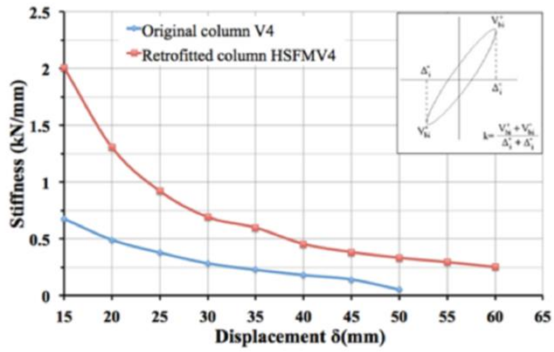
The strengthened subassemblage CV_5 showed a ductile behavior. This is clearly reflected in the spindle-shaped hysteresis loops illustrated in Fig. 6(d). The overall seismic performance of the column was substantially improved with respect to that of the original specimen V_5, due to the additional longitudinal (six ⌀10 mm B500C bars) and transverse reinforcement (⌀8/80 mm B500C spiral) of the RC jacket. Nevertheless, the lateral strength and peak-to-peak stiffness values of columns $HSFV_1$, $HSFV_2$, $HSFV_3$ and $HSFMV_4$ (which were strengthened with a HSSFC jacket) and the corresponding values of CV_5 (which was strengthened with a conventional RC jacket) are comparable. During the first cycle of the earthquake-type loading the strength ratio values $CV_5/HSFV_1$, $CV_5/HSFV_2$, $CV_5/HSFV_3$ equaled to 1.19/1.29, 1.20/1.16 and 1.27/1.21 (push/pull half cycles), respectively, while for drift angle, R, equal to 5.61 percent the ratio values equaled to 1.45/1.69, 2.18/1.88 and 1.72/1.55, respectively.



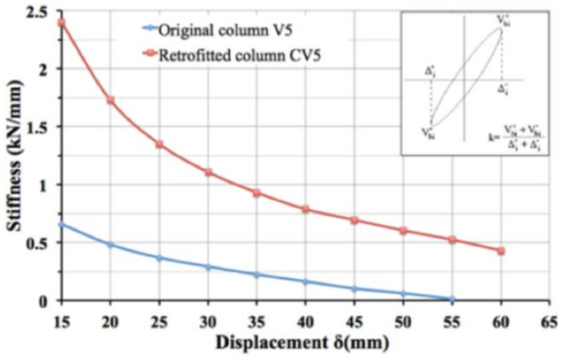
(a) Specimens V_2 and $HSFV_2$



(b) Specimens V_3 and $HSFV_3$



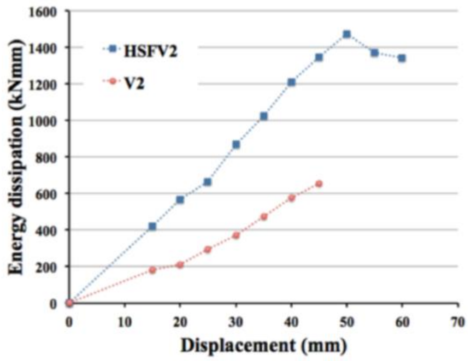
(c) Specimens V_4 and $HSFMV_4$



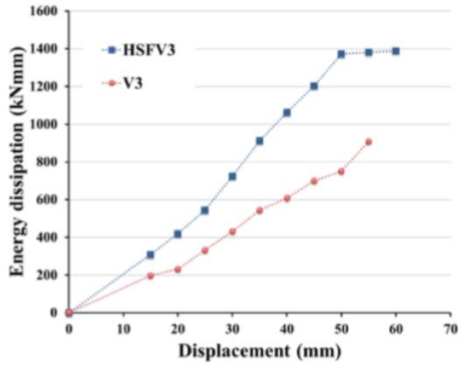
(d) Specimens V_5 and CV_5

Fig. 7 Peak-to-peak stiffness of the column specimens

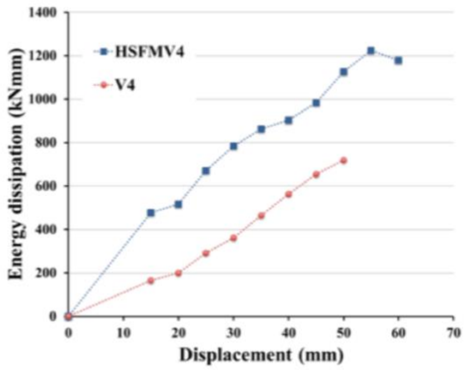
The hysteresis behaviour of all retrofitted subassemblages is characterized by spindle-shaped loops. The latter reflects the substantial improvement achieved in



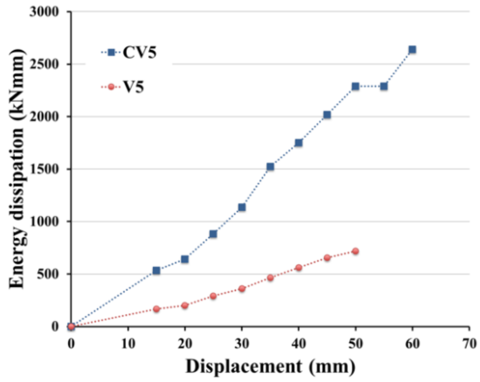
(a) Specimens V_2 and $HSFV_2$



(c) Specimens V_3 and $HSFV_3$



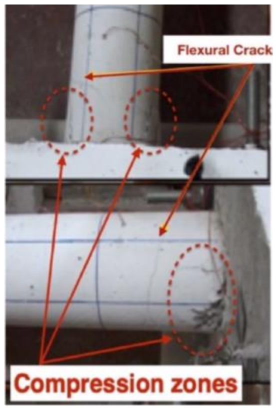
(c) Specimens V_4 and $HSFMV_4$



(d) Specimens V_5 and CV_5

Fig. 8 Energy dissipation capacity of the specimens

the energy dissipation capacity with respect to that of the corresponding original columns. During the first cycle subassemblages $HSFV_1$, $HSFV_2$, $HSFV_3$, $HSFMV_4$ and CV_5 dissipated 2.13, 2.29, 1.56, 2.87 and 3.21 times the

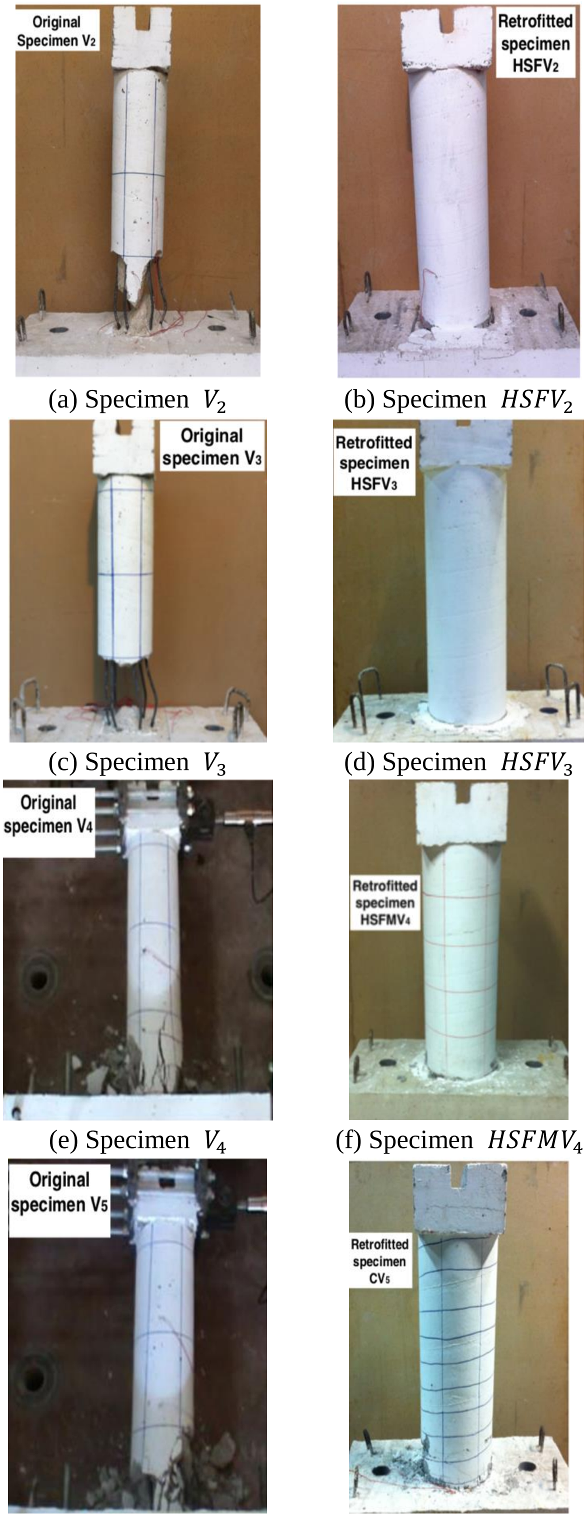


(a) Formation of hairline flexural and shear cracks and gradual rupture of the concrete in the compression zones

(b) The dilation of shear cracks results in gradual disintegration of the core concrete

Fig. 9 Cracking propagation - original column specimens

energy which was dissipated by the columns V_1, V_2, V_3, V_4 και V_5, respectively (see Fig. 8). Moreover, for lateral displacement equal to 45mm the values of energy dissipated by the strengthened columns showed 60.16, 106.28, 72.28, 50.19 and 208.23 percent increase with respect to the corresponding values of the original columns. At the end of the seismic loading sequence, the strengthened subassemblages achieved 195.48, 220.44, 348.78, 146.82 and 393.90 percent increase in energy dissipation values with respect to the first cycle of loading. Due to the additional reinforcement of the RC jacket, specimen CV_5 showed a substantial improvement in the energy dissipation capacity with respect to the original column V_5, while also dissipated higher values of energy compared to the columns that were retrofitted with the innovative HSSFC jackets. Thus, during the first cycle CV_5 dissipated 10.81, 21.55 and 42.11 percent increased values of energy compared to $HSFV_1$, $HSFV_2$, $HSFV_3$, respectively. After seven cycles of loading these values equaled 43.36, 33.22, 40.39 percent. However, the proposed innovative retrofit scheme was indisputably satisfactory in remarkably improving the energy dissipation capacity, peak-to-peak stiffness and lateral strength of $HSFV_1$, $HSFV_2$, $HSFV_3$ and $HSFMV_4$ with respect to the original specimens V_1, V_2, V_3 and V_4, without the use of conventional steel reinforcement and with significantly less labour demand with respect to the RC jacket. Moreover, the effectiveness of the innovative retrofit scheme proposed herein is further substantiated by the overall ductile hysteresis behaviour and the failure mode of the enhanced columns (see Fig. 10).



(a) Specimen V_2 (b) Specimen $HSFV_2$

(c) Specimen V_3 (d) Specimen $HSFV_3$

(e) Specimen V_4 (f) Specimen $HSFMV_4$

(g) Specimen V_5 (h) Specimen CV_5

Fig. 10 Failure mode of the original and the strengthened column subassemblages

The increase in column dimensions resulted in a significant increase of the peak-to-peak stiffness of the post-earthquake strengthened column subassemblages, with respect to that of the original ones. During the first cycle of the seismic loading $HSFV_1$, HSFV_2, HSFV_3, HSFMV_4 and CV_5 showed 2.75, 3.31, 2.77, 2.94 and 3.64 times higher

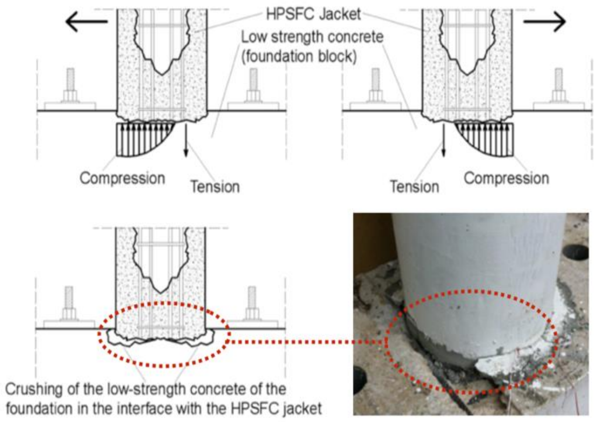


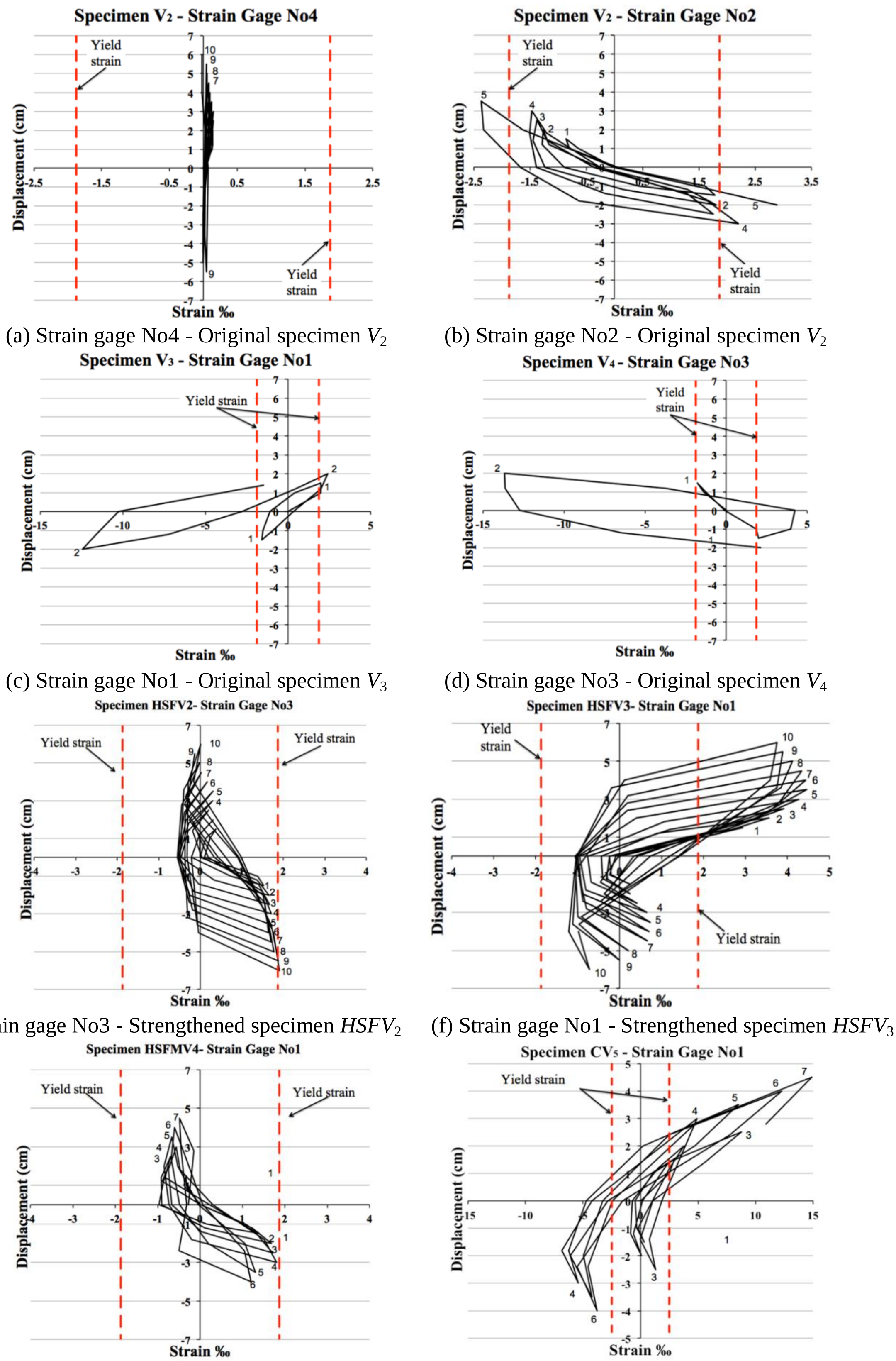
Fig. 11 The damage was concentrated solely to the foundation block of the strengthened subassemblages

peak-to-peak stiffness values than the corresponding original columns (see Fig. 7). Furthermore, the enhanced specimens maintained 21.47, 16.30, 14.02, 16.60 and 21.79 percent of their initial stiffness value (during the first cycle) for the same lateral displacement for which the original subassemblages collapsed, due to the loss of axial load carrying capacity. The peak-to-peak stiffness values of the retrofitted columns were similar for both the subassemblages strengthened with the HSSFC jackets and the specimen enhanced with the conventional RC jacket.

The original subassemblages developed brittle failure mechanisms and exhibited excessive seismic damage. Eventually, the columns collapsed due to the loss of axial load carrying capacity. Nevertheless, the HSSFC jacket ensured the desirable ductile behaviour of the retrofitted specimens and prevented damage during the seismic loading. The experimental results showed that the HSSFC jackets with either 50 mm or 60 mm width and steel fiber volume fraction equal to either 1.00 or 1.50 percent, found to be equally effective in restoring and substantially improving the overall hysteresis behaviour of the retrofitted columns, when subjected to a large number of inelastic lateral deformations. The contribution of mesh reinforcement to the effectiveness of the HSSFC jacket was found to be minor. Moreover, the lateral performance of subassemblages $HSFV_1$, HSFV_2, HSFV_3 and HSFMV_4 was very close to that of CV_5, which was strengthened with the conventional RC jacket. Ultimately, the proposed innovative retrofit scheme proved to be a reliable and easy to apply solution for the earthquake-resistant rehabilitation of existing earthquake-damaged (or undamaged) RC structures, that ensures the ductile behaviour of the columns with less time and labor demands.

6. Steel micro-strain monitoring

Electrical resistant strain gages were attached to the longitudinal and transverse reinforcing bars of the original subassemblages to allow monitoring of the variation of steel reinforcement strain values throughout the seismic tests. After the failure of the original specimens these strain gages were removed, the longitudinal steel bars of the columns were realigned and new strain gages were attached to the ongitudinal bars prior to the construction of the HSSFC or RC jacket. The exact location of each strain gage is presented in Fig. 13, while in Fig. 12 the plots of the load



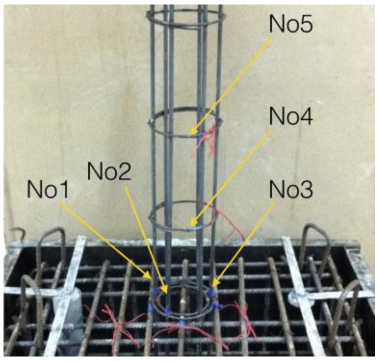
(a) Strain gage No4 - Original specimen V_2 (b) Strain gage No2 - Original specimen V_2

(c) Strain gage No1 - Original specimen V_3 (d) Strain gage No3 - Original specimen V_4

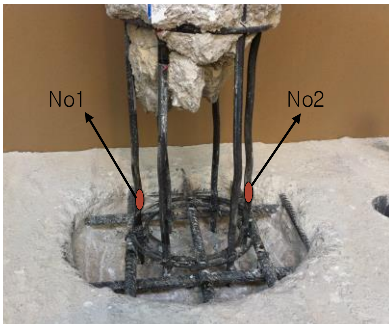
(e) Strain gage No3 - Strengthened specimen $HSFV_2$ (f) Strain gage No1 - Strengthened specimen $HSFV_3$

(g) Strain gage No1 - Strengthened specimen $HSFMV_4$ (h) Strain gage No1 - Strengthened specimen CV_5

Fig. 12 Plots of displacement-versus-strain of the longitudinal and transverse reinforcement of the original and the strengthened column subassemblages



(a) Original columns



(b) Strengthened columns

Fig. 13 Location of the strain gages attached to the column reinforcement

point displacement-versus-strain of reinforcement are illustrated. As a result, critical information for the seismic performance of the original and strengthened column subassemblages, was provided. For instance, given that the longitudinal reinforcement of the subassemblages was continuous and sufficiently anchored in the foundation block, a continuous increase in maximum steel strain to post-yield values was observed during the consecutive cycles of loading. According to Ehsani and Whight (1985) the latter indicates the absence of bar slipping and the ductile lateral response of the columns, while stable or decreasing strain values reflect hysteresis deterioration due to the slippage of the bars, as long as buckling has not taken place. In particular, strain values significantly higher than yield strain (1.87‰) were observed during the tests (2.74‰ Fig. 12(b), 12.33‰ Fig. 12(c), 13.82‰ Fig. 12(d), 1.91‰ Fig. 12(e), 4.46‰ Fig. 12(f), 1.87‰ Fig. 12(g) and 14.90‰ Fig. 12(h)). The yielding of the longitudinal bars demonstrates that the columns developed their nominal flexural strength. Moreover, all the retrofitted specimens performed in a ductile manner, since both the HSSFC jacket system and the conventional RC jacket system adequately confined the potential plastic hinge region and prevented seismic damage of the post-earthquake strengthened columns. It is noteworthy that the strengthened subassemblages showed yielding of the longitudinal bars (see Figs. 12(e)-(h)), which had also yielded previously during the seismic tests of the original columns and had subsequently been straightened during the retrofit process.

In the case of the original specimens, the concrete at the lower part of each column, where the plastic hinge was formed, was unconfined between the ties over a height of 250 mm. This resulted in the early formation of hairline shear cracks, which gradually dilated with the increase of the lateral drift angle. Afterwards, the rupture of the concrete in the compression zones and the exhaustion of both the aggregate interlock shear transfer capacity and the dowel action of reinforcement, resulted in the brittle shear failure and collapse of the original columns under the axial load. This explains the minimum strain values of the ties (see Fig. 12(a)), which were located above the plastic hinge region of the columns.

7. Conclusions

An experimental program was conducted for eight cantilever column subassemblages of approximately 1:1.5-scale to evaluate the effectiveness of a proposed innovative retrofit scheme in improving the seismic performance of the strengthened specimens. The strengthening method included the use of High-Strength Steel Fiber-reinforced Concrete (HSSFC) for the jacketing of earthquake-damaged columns found in existing pre-1970s RC structures. The innovative jackets lacked longitudinal and transverse reinforcement. Four identical original column subassemblages were designed and constructed with poor detailing of reinforcement, low-strength concrete (C12/15), plain steel reinforcement (S220), widely spaced transverse reinforcement and continuous longitudinal steel bars. The original columns were subjected to earthquake-type loading to failure. The spalled concrete was removed and the buckled reinforcement of the seismically damaged subassemblages was realigned. The specimens were subsequently retrofitted. Three columns were strengthened by HSSFC jacketing, while one column was enhanced with a conventional RC jacket. The strengthened columns were subjected to the same lateral displacement history as the original specimens. The influence of the HSSFC jacket width, the influence of the steel fiber volume fraction, as well as the contribution of mesh reinforcement (which was solely attached to the existing reinforcement but not anchored in the foundation block) in the hysteresis behaviour of the retrofitted columns was investigated. Moreover, the seismic response of the strengthened specimens with the innovative HSSFC jackets was compared to the hysteretic performance of a subassemblage which was enhanced with a conventional RC jacket. The following conclusions are drawn based on the work presented herein:

- The original subassemblages developed their flexural nominal moment capacity, due to the continuous longitudinal reinforcing bars, which were well anchored in the foundation block of the columns. However, the specimens eventually exhibited brittle shear failure and showed excessive disintegration of the core concrete in the plastic hinge region. As a result, the columns collapsed due to the loss of axial load carrying capacity.

• Due to the absence of lap splices of the column longitudinal reinforcement, the seismic performance of the original specimens was optimal for columns found in pre-1970s RC structures. Nevertheless, the subassemblages developed a brittle failure mechanism, which resulted in the catastrophic collapse of the columns.
The HSSFC jacket substantially improved the overall seismic performance of the strengthened subassemblages, which showed a remarkable increase in lateral strength, peak-to-peak stiffness and energy dissipation capacity. Thus, the retrofitted columns achieved an indisputable superiority in their overall hysteretic response with respect to the behavior of the corresponding original specimens, while remained intact at the end of the seismic tests.
• The reduction of lateral strength of subassemblages $HSFV_2, HSFV_3$ and $HSFMV_4$ is solely attributed to the rupture of the low-strength concrete (C12/15) of the foundation block in the interface with the HSSFC jacket. This is clearly depicted in the failure mode of the enhanced columns, which showed no damage after being subjected to a large number of inelastic cyclic lateral deformations.
• The additional longitudinal reinforcement of the RC jacket significantly increased the flexural capacity of specimen CV_5 with respect to the original column V_5. Moreover, the energy dissipation capacity of CV_5 was substantially improved.
• The columns which were retrofitted according to the proposed innovative HSSFC jacket strengthening system showed a lateral performance particularly close to that of subassemblage CV_5, providing a reliable, and effective solution with less time and labor demands, for the earthquake-resistant rehabilitation of existing structures.

Acknowledgements

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