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Experimental study on seismic behavior of reinforced concrete column retrofitted with prestressed steel strips

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Abstract. In this study, a new retrofitting method for improving the seismic performance of reinforced concrete column was presented, in which prestressed steel strips were utilized as retrofitting stuff to confine the reinforced concrete column transversely. In order to figure out the seismic performance of concrete column specimen retrofitted by such prestressed steel strips methods, a series of quasi-static tests of five retrofitted specimens and two unconfined column specimen which acted as control specimens were conducted. Based on the test results, the seismic performance including the failure modes, hysteresis performance, ductility performance, energy dissipation and stiffness degradation of all these specimens were fully investigated and analyzed. And furthermore the influences of some key parameters such as the axial force ratios, shear span ratios and steel strips spacing on seismic performance of those retrofitted reinforced concrete column specimens were also studied. It was shown that the prestressed steel strips provided large transverse confining effect on reinforced concrete column specimens, which resulted in improving the shearing bearing capacity, ductility performance, deformation capacity and energy dissipation performance of retrofitted specimens effectively. In comparison to the specimen which was retrofitted by the carbon fiber reinforced plastics (CFRP) strips method, the seismic performance of the specimens retrofitted by the prestressed steel strips was a bit better, and with much less cost both in material and labor. From this research results, it can be concluded that this new retrofitting method is really useful and has significant advantages both in saving money and time over some other retrofitting methods.

Keywords: prestressed steel strip; retrofitted methods; reinforced concrete column; seismic performance; quasi- static tests; experimental research

1. Introduction

In 1968, a large number of reinforced concrete columns whose ratios of shear span to section height, which were named as shear span ratio for simple and labeled as λ here,were relatively small and were severely damaged in the Tokachioki earthquake (Suzuki 1971). From then on, the importance and significance of seismic performance of short column was realized. And the short-column failure was then focused on and concerned. For the short column, its performance

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and design considerations are always determined according to the shear span ratio λ. For example, according to the *Building Seismic Design Code* in China, when the ratio λ of the column is smaller than 2.0, then the column is short column, and should be specially considered. And if its λ is smaller than 1.5, then it is regarded as ultra short column and some special design and detailing must be taken.

Nowadays, most of seismic codes of the world all provide special seismic design procedure for short columns, but seismic design of short column is still an actual problem. The short-column failure also frequently takes place and is always the main cause of severe damage of reinforced concrete columns in earthquake. In 1999 Taiwan M 7.6 Chi-Chi earthquake, plenty of reinforced concrete columns of school building were severely damaged, and most of their failure modes were also the short-column failure (Wang and Bai 2001). Unfortunately, in 1999 Turkey M 7.4 Izmit earthquake, 2011 Turkey M 7.2 Von downtown earthquake and in 2007 Peru M 8.0 Pisco-Chincha earthquake, especially in 2008 Wenchuan M 8.0 Earthquake, there were lots of reports of short-column failure in the reinforced concrete buildings (Li 2014, Dogan 2013, Kaplan 2004, Kwon and Kim 2010, Li 2009, Civil and Structural Groups of Tsinghua University 2008).

The main reason why so many short-column failures happened is almost clear. For the short column, it absorbed large shear force due to its large lateral stiffness, and the shear bearing capacity of the reinforced concrete column is not adequate due to low shear strength of concrete. So, the short column fails mainly in shear failure mode. As well known, shear failure is very brittle, abrupt and always results in severe damage, which were verified by lots of real earthquakes and experiments.

One direct method to improve the seismic performance of short column and then avoid the short-column failure is improving its shear capacity. For new buildings, it could be easily to accomplish by either enlarging the amount of transverse stirrups or using higher strength concrete. But for those old buildings which were built in early years with little consideration or without sufficient consideration of short-column failure, their seismic performance and their safety must be concerned, especially those short columns with low strength concrete. Furthermore, with the increasing of performance demand and safety demand of seismic building according to the new seismic design codes, the designing earthquake forces on buildings are mostly enlarged, while the concrete strengths are mostly unfortunetaly decreased. Therefore, according to the newest building seismic design codes, lots of short columns in the old buildings will be definitely insufficient in shear capacity and will be severely damaged in short-column failure. Therefore, in order to ensure the safety of those old reinforced concrete buildings, some strengthening work or retrofitting work have to do to improve the seismic performance of short columns.

In these 20 years, many researchers have conducted plenty of experiments on seismic performance of reinforced concrete columns retrofitted by lots of different retrofitting methods. Among those retrofitting methods, the method of wrapping carbon fiber reinforced plastics (CFRP) strips is mostly studied and mostly accepted (Wang *et al.* 2009, Chastre and Silva. 2010, Promis and Ferrier 2012, Biskinis and Fardis 2013), steel tube (Zhou *et al*. 2011, Zhang *et al*. 2011). In the CFRP strips retrofitting method, a series of CFRP strips are glued or bonded to the surface of the columns, and formed a series of transverse CFRP hoops while confined the columns at the same time. That means, the CFRP strips can not only act as the transverse reinforcement to resist the shear force directly, but also can confine the concrete to improve both the ultimate strength and the ultimate strain, which will all finally result in improving both the shear capacity and the deformation capacity.

However, for the CFRP strips retrofitting method, the actual retrofitting results and effect are not as perfect as thought. As well known, for the bonded CFRP strips method, only when transversely deformation occurred, the CFRP strips will stressed, and then the CFRP strips could confine the concrete, if there is no transversely deformation, the bonded CFRP strips will not confine the concrete and will only act as the transversely reinforcements. In other words, the bonded CFRP strips retrofitting method is a kind of passive retrofitting method. There is an obviously second-stages stressing process, which will lead to stress-lag and strain-lag of retrofitting CFRP strips, and will lead to a result that CFRP strips will remain a low stress level while columns are loaded to failure.

To achieve active lateral confinement of concrete, many retrofitting methods have been proposed, such as prestressed steel strand (Guo *et al*. 2014, Budek *et al*. 2001), prestressing steel plate (Guo *et al*. 2006, Guo *et al*. 2010, Garden and Hollaway. 1998), the shape memory alloy stirrup (Andrawes *et al*. 2010, Chen *et al*. 2014), prestressing FRP reinforcement method (Shin and Andrawes 2011, Zhou *et al*. 2013, Yamakawa *et al*. 2005, Shahab and Hasan 2007, Wight *et al*. 2001, You *et al*. 2012) and etc.. Based on those current methods, Yang Yong *et al*. developed the prestressed steel strip retrofitting method (Yang *et al*. 2011). A special packaging technique is introduced to provide active transversely confinement for reinforced concrete. Comparing to others methods, the prestressed steel strips method has obvious advantages of both easily construction and low cost. The retrofitting stuff including the tools and steel strips are cheap and easy to get without any special procedures to produce. And furthermore, the retrofitting steps are very easy to study and conduct.

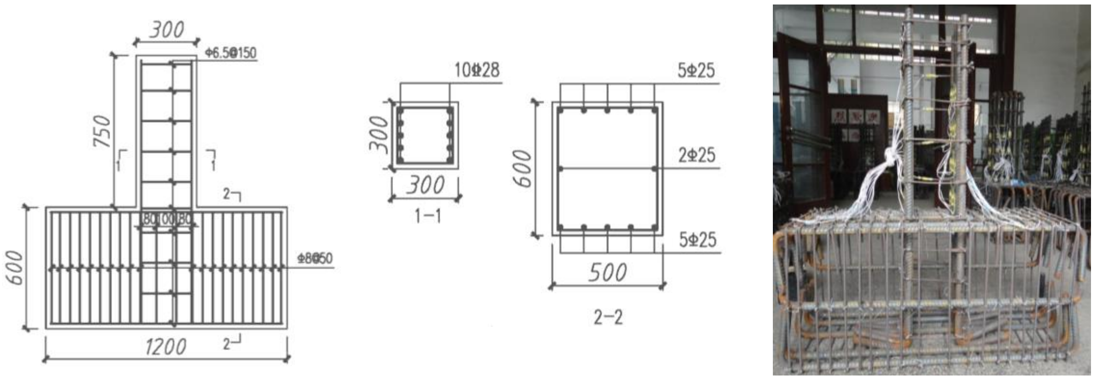
Form 2011, Yang *et al*. conducted a series of experiments on this retrofitting method, in order to study and figure out the retrofitting effect both on beams and columns. From those experiments results, this retrofitting method was verified useful, and a set of design methods were also established. This paper presents the results of a series experiments conducted to study the seismic performance of reinforced concrete short column retrofitted by this new method. In this paper, experiment design and experimental results as well as the results analysis of six retrofitted specimens and two control specimens were fully introduced.

2. Test Program

2.1 Specimens preparation

In the experiment, there were eight specimens were tested. All the eight specimens were designed as T- shape subassembly, which composed of a large base beam and a half-column. The base beam was just used to fix and set up the specimens to the ground, while the half-column was used to simulate the short columns, and it was thereby named as column specimen.

The reinforced concrete column specimens were sorted as two series mainly according to the shear span ratios. For series I, the shear span ratios were all 1.5, and for series II, the shear span ratios were all 2.0. All the two series of column specimens were designed to square cross section, whose width and height were 300 mm and 750 mm respectively. The longitudinal reinforced bars of the specimens were all HRB400 grade bars, whose diameter were 28 mm. The tested yielding stress and ultimate stress of the reinforced bars was 498 MPa and 619 MPa respectively. The transverse stirrups of each specimen were all HPB235 grade bars, whose diameter and spacing were 6.5 mm and 150 mm respectively, and the tested yielding stress and ultimate stress of the reinforced bars was 313 MPa and 482 MPa. The average cubic compression strength of concrete in



(a) Dimensions (b) Cross section detail (c) Photos of reinforcement skeleton

Fig. 1 Dimensions and details of specimens

specimens of series I was 25.3 MPa, and that of concrete in specimens of series II was 31.6 MPa. The loading points or loading positions of two series specimens were different, the series I specimens were loaded at the 600 mm-height point, while the series II specimens were loaded at the 450 mm-height point. Therefore, the shear span ratios of series I specimens and series II specimens were 2.0 and 1.5 respectively. The dimensions and cross sectional details of the specimens were denoted in Fig. 1.

2.2 Specimens retrofitting

Among those eight prepared column specimens, two specimens labeled as RC-1 and RC-2 were two control specimens, which were not retrofitted. And specimens PSRC-1, PSRC-2, PSRC-3, PSRC-4 and PSRC-5 were specimens retrofitted with prestressed steel strips methods, the specimen CFRP-1 was retrofitted with bonded CFRP strips method for comparison. Based on the shear span ratios, these specimens were sorted as two series. There were four specimens in series I, which including specimen RC-1 and specimen PSRC-1, PSRC-2 and PSRC-3. The other four specimens were sorted as series II.

For the five specimens retrofitted with prestressed steel strips method, the retrofitting methods and retrofitting design were same. Firstly, the four corners of the column specimens were chamfered to round corners to reduce the friction influence and smooth the force transfer, with the aim of producing uniform transverse confining stress to concrete. The chamfer radius of all the corners were 50 mm. Secondly, a series of steel strips were set up to the specimens by a set of devices, which including an air compressor, a stretching machine, which are very commonly used in packing. The five specimens were all retrofitted with a kind of extra-high strength steel strips, whose ultimate tensile strength was 871 MPa. The width and the thickness of the steel strips were 32 mm and 0.9 mm respectively. With the stretch machine, the steel strips were stretched, fixed and finally formed a series of transverse hoops around the column specimens, the spacing of the steel strips hoops in specimen PSRC-4 was 150 mm, while that of other four specimens were all 100mm. Because the steel strips were stretched before fixed, and thereby some prestress were produced and were kept in the steel strips. By measuring the stains of the steel strips during the whole setup process, the pre-strains of the steel strips in those retrofitted specimens were recorded, and the measured average pre-strain of all the steel strips was around 2.0×10^{-3}, that means the


(a) Air compressor


(b) Stretching machine


(c) Steel strips


(d) Steel strips connectors

Fig. 2 Setup devices and retrofitting stuffs

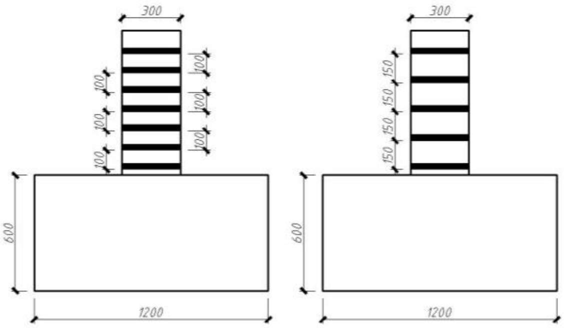
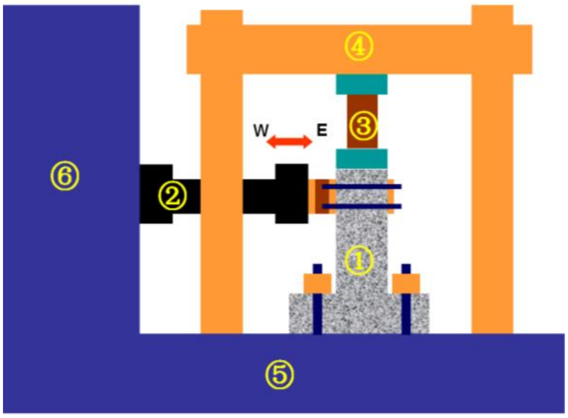


Fig. 3 Schematic diagram of specimen reinforcement

Table 1 Experimental parameters

Series	specimen number	Concrete cubic compression strength f_c /MPa	Experimental Axial compression force ratio n	Shear span ratio λ	Spacing of steel strip or CFRP strips S_{ps}/mm
Series I	RC-1	25.3	0.53	2.0	—
	PSRC-1	25.3	0.53	2.0	100
	PSRC-2	25.3	0.42	2.0	100
	PSRC-3	25.3	0.28	2.0	100
Series II	RC-2	31.6	0.42	1.5	—
	PSRC-4	31.6	0.42	1.5	150
	PSRC-5	31.6	0.42	1.5	100
	CFRP-1	31.6	0.42	1.5	50

average pre-stress of the steel strips was about 400MPa. The retrofitting devices were showed in Fig. 2, and the retrofitted specimens were denoted in Fig. 3. The key parameters of all the specimens were listed in Table 1.



1-tested column specimen;2-actuator; 3-vertical jack;
4-vertical reaction frame; 5- lab ground; 6-reaction wall.

(a) Schematic diagram

(b) Photo of the load-setup

Fig. 4 Test setup

For the specimen CFRP-1 which was retrofitted with the bonded CFRP method, the procedure was strictly according to the retrofitting instructions. The spacing of the CFRP sheets was 50mm, the width and of the thickness of the CFRP sheets were both 50mm and 0.333mm. The CFRP sheets were formed by two layers of 0.167 CFRP sheets.

2.3 Test procedure

The experiment was conducted in *Structural Engineering Key Laboratory at Xi'an University of Architecture and Technology*. All the eight specimens were tested by quasi-static experiment method, in which the specimens were loaded horizontally and reversely in cycles by electro-hydraulic servo actuators. The quasi-static experiment method is the most popular test method of seismic performance researching nowadays. The specimens loaded reversely in low frequency cycles, and the load-displacement hysteresis curves could be directly measured, and the seismic performance such as the bearing capacity, deformation capacity, the stiffness as well as the restoring force features and so on could be obtained.

The loading setup of this experiment was illustrated in Fig. 4. Seen from the Fig. 4, the column specimen was firstly fixed to the ground by anchoring the base beam, and then the vertical load was loaded by a jack to a constant force, whose magnitude was controlled by the designed compression force ratio. Thirdly, the specimen was reversely loaded by MTS electro-hydraulic servo actuators in horizontal direction to failure. In this experiment, all the horizontal loading steps of all the specimens were controlled by the load-point displacements and in terms of rotation angles of the specimens, which were designed as 1/1500, 1/1000, 1/750, 1/500, 1/250, 1/200, 1/150, 1/100, 1/75, 1/50, 1/40, 1/30, 1/20, before the step of 1/500, all the steps were reversely loaded in one cycle, and after the step of 1/500, all the steps were loaded for three cycles, all the column specimens were planned to be keeping loaded till their fail. Here the failure of specimens was defined as two conditions, one of which was the horizontal load was decreased to less 85 percent of the peak load and another condition was the vertical load cannot be sustained.

3. Test results

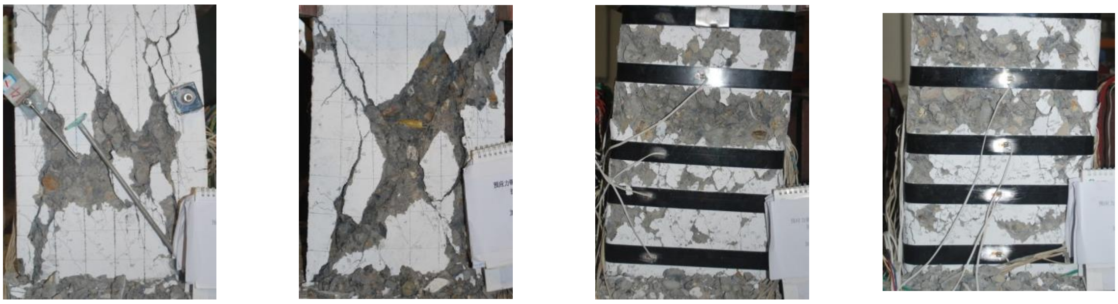
3.1 Failure modes

1) Control specimens RC-1 and RC-2

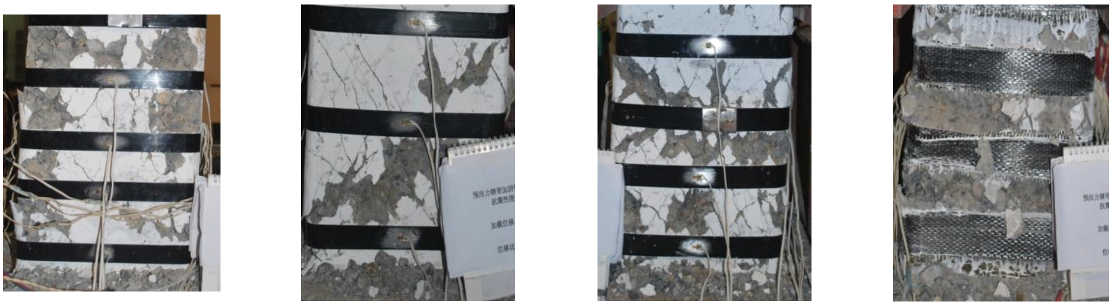
As for unretrofitted reinforced concrete column specimens RC-1, RC-2, no distinctive change occurred at the initial loading stage, and just after some small and irregular diagonal cracks in the south and north surface of the specimen occurred, diagonal cracks increased quickly and extended thoroughly. With the horizontal displacement increasing, diagonal cracks widened and extended quickly, accompanying some surface concrete of the specimens crushing and falling. On the east and west surface of the specimens, concrete also fell off and some diagonal cracks extended both to columns bottom and loading point directions, and a obvious X-shape crack was formed. Eventually, a large piece of concretes fell off in the shearing zone and concrete bulged at the east side of the specimen. As showed in Figs. 5(a)-(b), both two specimens failed in typical short-column shear failure.

2) Specimens retrofitted by prestressed steel strips methods

The specimens PSRC-1, PSRC-2, PSRC-3, PSRC-4 and PSRC-5 which retrofitted by prestressed steel strips methods were observed very similar in failure modes. At the initial loading stage, when the horizontal rotation angle was less 1/200, there was no visible crack. When the horizontal rotation angle was up to around 1/200, some small and irregular diagonal cracks (longitudinal reinforcement position) occurred on the south and north surface of the specimen. After that, more and more cracks occurred and were observed. When the horizontal rotation angle reached a certain level, which was a little different for different specimens and averaged 1/100, the diagonal cracks developed and extended, and some concrete fell off slightly. But different to the unretrofitted specimens, the falling and crushing of concrete of the retrofitted specimens were slightly and restricted in the areas between the steel strips, and with keeping load, the steel strips became more and more tight, and some obvious sound of the steel strips were heard. Finally, when the rotation angles were increased up to around 1/30, which were a little different for five specimens and meant the specimens were severely deformed, the concrete in shear zone fell off and some steel strips broke, and the experiments were stopped. As showed in Figs. 5(c)-(g), the



(a) Specimen RC-1 (b) Specimen RC-2 (c) Specimen PSRC-1 (d) Specimen PSRC-2



(e) Specimen PSRC-3 (f) Specimen PSRC-4 (g) Specimen PSRC-5 (h) Specimen CFRP-1

Fig. 5 Failure modes of test specimens

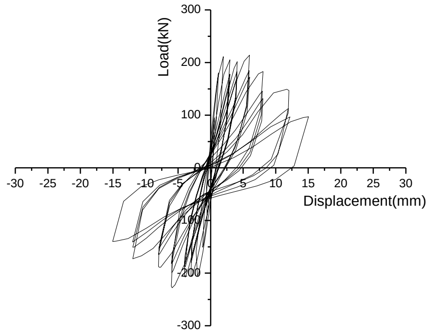
failure modes of these retrofitted specimens were a shearing failure but more ductile and completely then the typical short-column shear failure modes.

3) Specimen retrofitted by bonded CFRP sheets

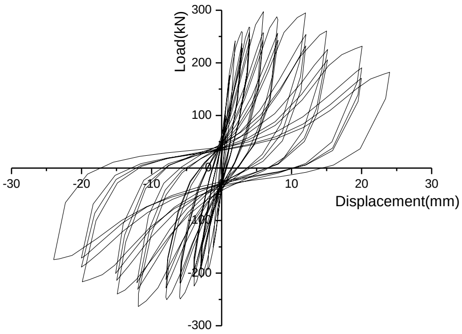
The specimen CFRP-1 that retrofitted by the bonded CFRP sheets, the failure process and the failure mode were somewhat similar to that of the specimens retrofitted by the prestressed steel strips method. Similarly, at the early loading stage, when the horizontal loading was small, the specimen was almost in elastic stage, and when horizontal rotation angle reached 1/150 and the horizontal load reached 411.69kN, some cracks were observed both in the south and north surface of the specimen, and a small amount of tiny diagonal cracks were also observed on the north surface. When the rotation angle reached 1/100, the crack developed quickly and some new diagonal cracks appeared in the north of the specimen, and the trend of diagonal cracks was observed clearly. When the rotation angle reached 1/50, the diagonal crack fully extended and widened on the north and south surface of the specimen. When the horizontal rotation angle reached 1/37.5, cracks became much wider and concrete started falling off. With the load increasing, when the rotation angle finally reached 1/20, some CFRP sheets were broken and peeled off, and concrete at bottom was crushed, which was shown in Fig. 5(h).

3.2 Load-deformation hysteresis curves

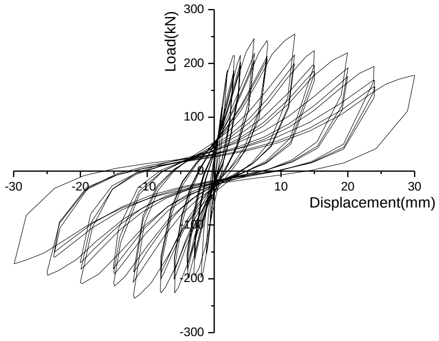
The tested hysteresis curves of the specimens were shown in Fig. 6. Seen from the hysteresis curves, the hysteresis performances of retrofitted specimens were much better than the control specimens. Although all the hysteresis curves were not plump, due to the concrete cracking in the



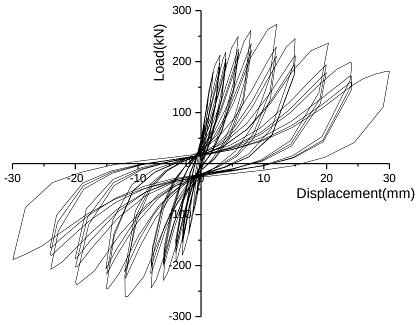
(a) Specimen RC-1



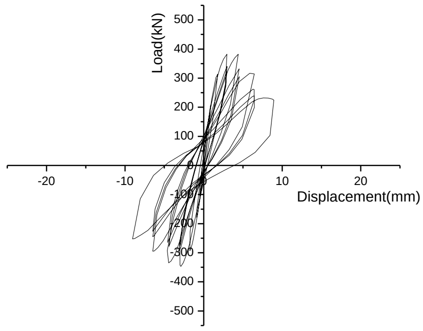
(b) Specimen PSRC-1



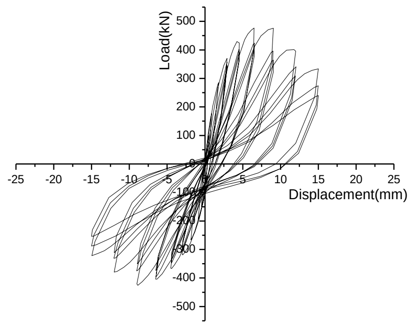
(c) Specimen PSRC-2



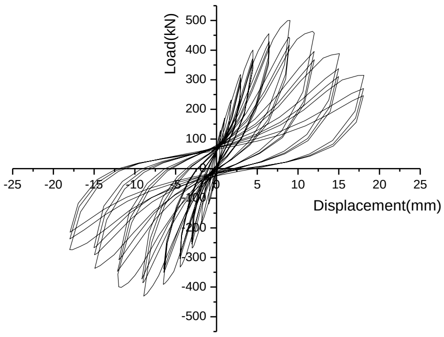
(d) Specimen PSRC-3



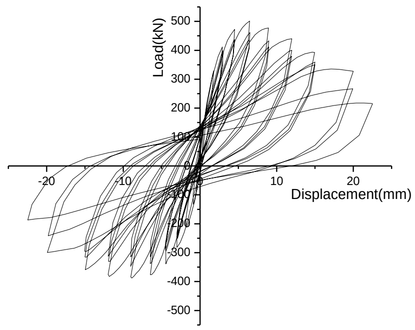
(e) Specimen RC-2



(f) Specimen PSRC-4

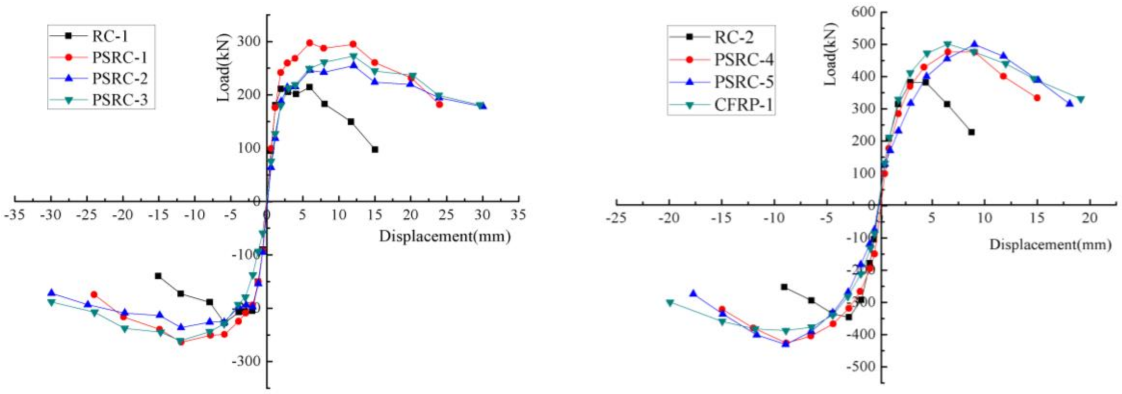


(g) Specimen PSRC-5



(h) Specimen CFRP-1

Fig. 6 Load-displacement hysteresis curves of test specimens



(a) series I (b) series II

Fig. 7 The backbone curves of specimens

Table 2 Experiment results at main loading stages

Specimen Number	Yielding stage		Peak load stage		Ultimate stage		Ductility coefficients		Ultimate rotation angle
	P_y/kN	Δ_y/mm	P_O /kN	Δ_O /mm	P_u/kN	Δ_u/mm	$\mu=\Delta_u/\Delta_y$	improvement (%)	$\theta=\Delta_u/H$
RC-1	191.92	1.93	214.23	5.90	193.76	8.13	4.22	—	1/73.8
PSRC-1	249.73	2.41	297.37	5.97	252.76	16.52	6.86	62.6	1/36.3
PSRC-2	212.06	3.04	254.88	12.07	216.65	19.65	6.47	53.5	1/30.5
PSRC-3	205.19	2.81	272.96	12.06	232.01	19.96	7.11	68.6	1/30.1
RC-2	329.48	2.88	382.49	2.96	325.11	8.32	2.89	—	1/54.1
PSRC-4	382.78	3.25	476.47	6.51	405.00	11.75	3.62	25.2	1/38.3
PSRC-5	387.97	3.51	500.04	9.02	425.03	13.44	3.82	32.3	1/33.5
CFRP-1	403.74	2.90	501.23	6.48	426.04	12.72	4.39	52.0	1/35.4

Note: the lower label of y, o, u represent yield, peak and limit state respectively.

shear failure process, that of the retrofitted specimens were much plump than that of the control specimens RC-1 and RC-2.

From the above load-deformation hysteresis curves, the backbone curves of specimens in series I and series II were obtained and were shown in Fig. 7.

3.3 Shear capacity

From the recorded data and the load-deformation hysteresis curves, the loads and deformations at three different which named yielding stage, peak load stage and the ultimate stages of all the specimens were obtained and were all listed in Table 2. From the loads and deformations of those three stages, the shear capacities, the ductility coefficients and the ultimate rotation angles of all the specimens were obtained and also listed in Table 2.

From the tested results showed both in Table 2 and Fig. 6 and Fig. 7, the shear capacities of all the specimens could be obtained. It could be observed that the shear capacities of retrofitted specimens were larger than that of the unretrofitted specimens. In series I, the shear capacity of specimen RC-1 was 214.23kN, while that of the control specimen PSRC-1 was 297.37kN, the shear capacity was improved 38.9 percent, and the average improvement rate of the shear capacities was 28.3 percent, which meant that the prestressed steel strips contributed well in shear capacity. And from the results of specimens PSRC-1, PSRC-2 and PSRC-3, it was denoted that the improvement rate of shear capacities were also influenced by the axial compression force ratio. In series II, the average improvement rate of the shear capacities of retrofitted specimens was 28.8 percent. And the improvement rate of the specimen of CFRP-1 and PSRC-5 was almost same, while the retrofitting cost of specimen CFRP-1was almost three times as that of specimen PSRC-5. Based on some theoretical analysis, it could be simply found that the prestressed steel strips were almost fully utilized in resisting the shear force and improving the shear capacities of the specimens.

3.4 Deformation performance

Form the tested results, the ultimate deformation rotation angles of two unretrofitted specimens RC-1 and RC-2 were 1/73.8 and 1/54.1 respectively, but that of the retrofitted specimens were all larger. For example, specimen PSRC-1 and specimen RC-1 were same in cross section details, shear span ratio and axial force ratio etc. but the ultimate deformation rotation angles of them were 1/36.3 and 1/73.8 respectively, it denoted that the retrofitted method did work very well. And same conclusions could be also found in series II, the specimen PSRC-4 and specimen RC-2.

From the results of specimens of series I and series II, it could be concluded that the shear span played important roles in the deformation performance and ductility, for series I specimens whose shear span ratios were 2.0, the deformation capacities were much better than that of series II whose shear span ratios were 1.5.

From the results and backbone curves shown in Fig. 7 of specimen PSRC-1, PSRC-2 and PSRC-3 in series I, there was no obvious influence of axial force in deformation performance. It can be explained as follows, because the prestressed steel strips confined the concrete very well and then highly improve the compression strength of concrete, the difference of axial compression force became relatively smaller, and thereby no obvious influence was observed.

From the tested results of specimen PSRC-4 and specimen PSRC-5, which were only different at the spacing of steel strips, it could be found that ductility coefficient of specimen and the ultimate rotation angle increased with the decreasing of spacing of prestressed steel strips. This indicated that using smaller spacing of steel strips will get better deformation performance.

The ultimate rotation angle of specimen PSRC-5 was larger than that of specimen CFRP-1, although the ductility coefficient of specimen PSRC-5 was a little smaller than that of CFRP-1. It showed that the deformation performance of specimen retrofitted with the prestressed steel strip method could as good as that of specimen retrofitted with the bonded CFRP sheet method.

3.5 Equivalent viscous damping coefficient

The equivalent viscous damping coefficient h_e is always used to evaluate the energy dissipation performance of the specimens. The greater the equivalent viscous damping coefficient is, the better the energy dissipation performance has. The equivalent viscous damping coefficient h_e can usually be calculated as follows

$$h_e = \frac{1}{2\pi} \cdot \frac{S_{(\mathrm{ABCDA})}}{S_{(\mathrm{OBE+ODF})}} \tag{1}$$

Table 3 Energy dissipation of test specimens

Specimen number	Total energy consumption /kN·mm	Energy dissipation ratio
RC-1	5492.45	1.00
PSRC-1	40121.92	7.30
PSRC-2	45057.61	8.20
PSRC-3	36923.68	6.72
RC-2	12359.74	1.00
PSRC-4	35890.33	2.90
PSRC-5	43586.50	3.53
CFRP-1	57747.38	4.67

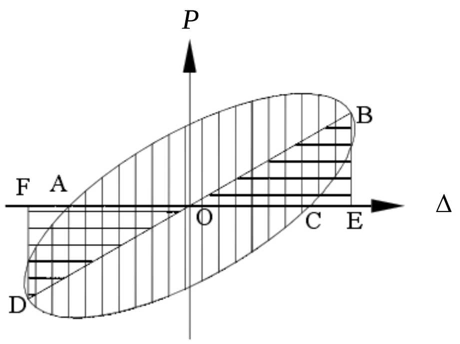
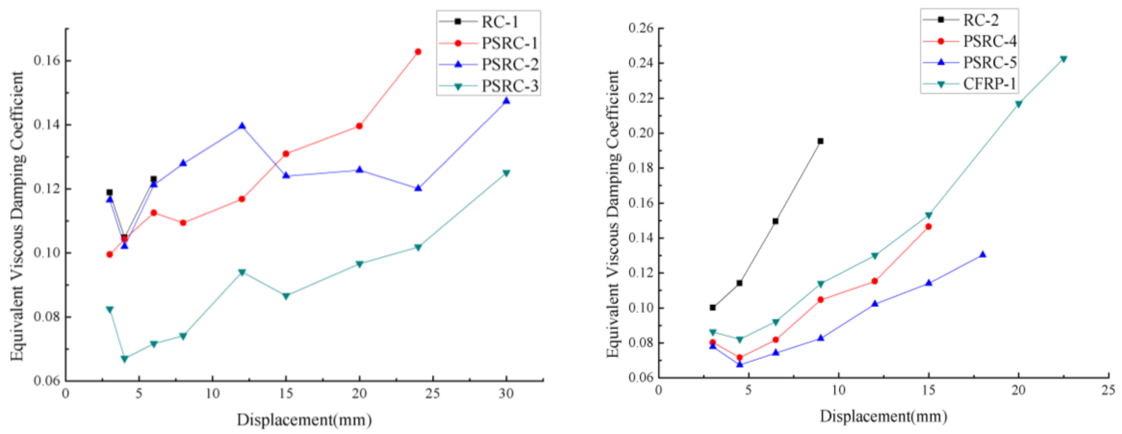


Fig. 8 The schematic of equivalent viscous damping coefficient



(a) series I (b) series II

Fig. 9 The equivalent viscous damping coefficient of test specimens

in which $S_{(ABCDA)}$ represents the area of the hysteresis loop ABCDA, $S_{(OBE + ODF)}$ represents the area of the triangles OBE and ODF, as indicated in Fig. 8.

The equivalent viscous damping coefficients of the specimens were shown in Fig. 9 and were also listed in Table 3. As shown in Fig. 9, the equivalent viscous damping coefficient of prestressed steel strip reinforced concrete column PSRC-1 was much higher than that of unretrofitted specimen RC-1. According to Table 3, the total energy dissipation of retrofitted specimen was substantially higher than the unretrofitted specimens. From Fig. 9, the equivalent damping ratio of CFRP-1 was higher than specimen PSRC-5. Taking into the total energy consumption into consideration, CFRP-1 was also slightly higher as well. This illustrated that the energy dissipation performance of CFRP reinforced concrete columns in this experiment was a little superior to prestressed steel strip reinforced concrete columns. Also from the results, the equivalent viscous damping coefficient of specimen PSRC-4 (150 mm spacing) was slightly higher than that of specimen PSRC-5 (100 mm spacing) at later loading period, but the total energy dissipation of specimen PSRC-5 was higher that of PSRC-4. It was shown that small spacing steel strips will improve the energy dissipation performance of the specimens.

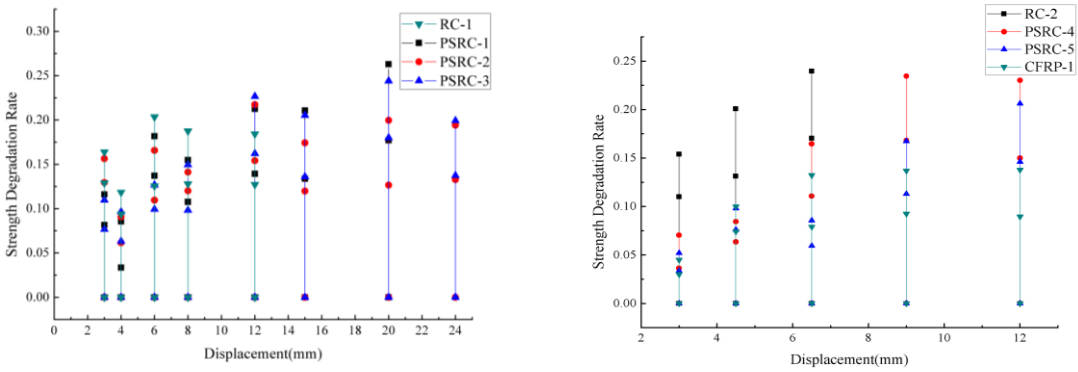
3.6 Bearing capacity degradation

At every controlled displacement loading step, the bearing capacity of specimen usually decreases with the increasing of load cycles. Seismic performance was affected by this kind of bearing capacity degradation greatly. Here the degradation rate of bearing capacity was calculated by the formula (2)

$$\gamma = \Delta P / P \tag{2}$$

And here P was the peak load value in the first cycle under different control displacement and ΔP was difference value between the peak load value of second cycle to the peak load value of the first cycle.

The decreasing rates of bearing capacity of the specimens were shown in Fig. 10. As shown in Fig. 10, the decreasing rates of bearing capacity of unretrofitted control column specimens were much larger than that of the retrofitted column specimens. From Fig. 10, it could be observed that the decreasing rate became larger when the axial compression ratio of specimen increased. it could also be found that the decreasing rate of the specimens retrofitted with the prestressed steel strips method was faster than that of the specimens retrofitted with the bonded CFRP sheets methods. And it was also shown that reducing the spacing of prestressed steel strips of retrofitted specimens can reduce the decreasing rate of bearing capacity to some extent.



(a) series I (b) series II

Fig. 10 The rate of strength degradation of test specimens

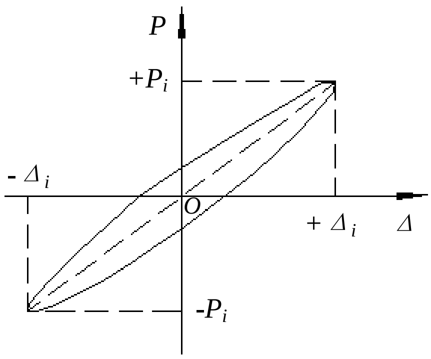
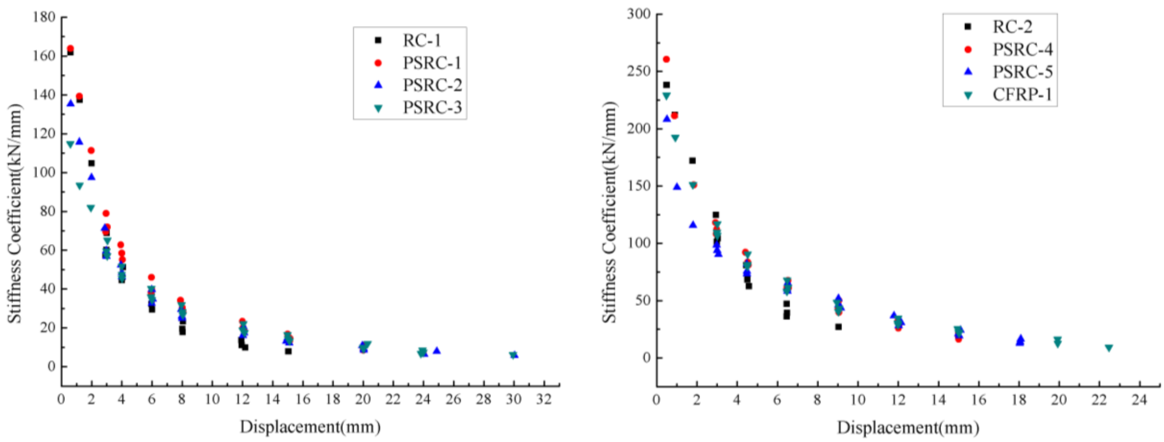


Fig. 11 Calculation of secant stiffness



(a) series I (b) series II

Fig. 12 The stiffness degradation curves of test specimens

3.7 Stiffness degradation

In the quasi-static experiment of reinforced concrete members, the degradation of specimen stiffness is usually observed, and which was sometimes the major cause of degradation of seismic performance of specimens. Here the secant stiffness index K_i was used to investigate the stiffness degradation of each specimen, which was calculated by formula (3).

$$K_i = \frac{|+P_i| + |-P_i|}{|+\Delta_i| + |-\Delta_i|} \tag{3}$$

Here the variable P_i was the peak load of the *i*-th loading step, and the variable Δ_i was the loading point displacement of the same loading step, which was indicated in Fig. 11.

The stiffness degradation of all the specimens was shown in Fig. 12. As shown in Fig. 12, the stiffness degradation rates of unretrofitted control specimens were much larger than that of the retrofitted specimens, while the axial compression ratio of specimens had played little influence on the stiffness degradation of specimens. Seen from Fig. 12, in the early loading period, stiffness degradation rates of specimens retrofitted with the prestressed steel strips method were much smaller than that of specimens retrofitted with the bonded CFRP sheets method. However, when the loading-point displacement reached the value of 15 mm while the displacement rotation angle reached 1/30, the stiffness degradation rates of retrofitted specimens became larger, that meant the stiffness of the specimens decreased faster. And it was also shown that reducing the spacing of prestressed steel strips of retrofitted specimens can reduce the decreasing rate of the stiffness degradation.

4. Conclusions

In this study, a new method to retrofit the reinforced concrete columns which named as the prestressed steel strip retrofitted methods was introduced. And a series of quasi-static experiments of eight specimens were conducted and presented. According to the experimental results, comprehensive analyses of the seismic performance of all the specimens were conducted, and some initial conclusions could be drawn as following.

(1) During the testing process, the prestressed steel strips were stressed, and then a kind of substantial confining effect was imposed on concrete in the retrofitted specimens, which was a kind of active confinement and was very helpful to delay and postpone the emergence and development of diagonal cracks.

(2) The failure modes of the retrofitted specimens transformed from brittle short-column shear failure to more ductile failure modes, in which more small cracks occurred and the concrete were confined and fully utilized.

(3) Due to the effectively confinement of the steel strips, the shear capacities of the retrofitted specimens were improved to around 28 percent, and fortunately the steel strips were fully utilized, which meant more shear capacity could be improved with more steel strips added.

(4) The deformation performance of the retrofitted specimens was much better than that of the control specimens, which were concluded from the improvement of the ultimate rotation angles and the ductility coefficients. The ultimate rotation angles of the short column specimens were improved from 1/78.3 to 1/36.3, which was enough to meet the requirements of the design codes.

(5) The energy dissipation capacities of retrofitted specimens were also highly improved, which resulted in enhancing the seismic performance of the short columns.

(6) The rates of both the bearing capacity degradation and the stiffness degradation of unretrofitted column specimens were much larger than that of retrofitted column specimens. It was also shown that reducing the spacing of prestressed steel strips of retrofitted specimens can reduce the decreasing rate of both the bearing capacity degradation and the stiffness degradation.

(7) With the comparing analysis, the seismic performance of specimens retrofitted with the prestressed steel strips retrofitting method was similar to and sometimes even better than that of the specimens retrofitted with the bonded CFRP sheets retrofitting method. And furthermore, if take the cost into account, the prestressed steel strips retrofitting method will much more economical and convenient than the bonded CFRP sheets methods.

(8) From all these tested results, it could be concluded that this new retrofitting method is really a good and effective method to strength, repair and retrofit the reinforced concrete column, especially short columns.

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