EXPERIMENTAL STUDY ON A NEW RETROFITTED SCHEME FOR SEISMICALLY DEFICIENT RC COLUMNS

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ABSTRACT :

This paper introduces a scheme to retrofit seismically deficient reinforced concrete (RC) columns or piers with square section is proposed that using prestressed steel plate hoop(PSPH) was presented and experimentally investigated. In the present study, six half-scale short RC column specimens were tested under cyclic loading and the failure mechanism, strength, ductility, hysteretic character, and energy dissipation capacity were examined. The main parameters of the present test were axial compressive ratio of the columns and the prestressed level of the PSPH. Based on the test results, it was concluded that the seismic performance of the retrofitted specimens can be successfully enhanced by the proposed method, provided that the shear strength and the shear details of the existed columns are substandard design.

KEYWORDS: prestressed steel plate hoop, concrete columns, seismic retrofit, experimental study, seismic performance

1 INTRODUCTION

Due to damages by earthquake, changes in codes, rezoning of seismic intensity, poor detailing practice, and wrong design et al, many existing reinforced concrete buildings are seismically deficient. A large number of seismic appraisals of existing buildings and bridges have indicated that the major deficient of reinforced concrete (RC) columns and piers is the lack of sufficient transverse reinforcement.

Transverse reinforcement in concrete columns is used to fulfill three main functions. These functions include restraining longitudinal reinforcement against buckling, increasing shear resistance, and confining concrete for improved deformability. The lack of sufficient transverse reinforcement in short columns (columns with small height-to-depth ratios) may lead to a brittle shear failure due to inadequate shear strength under earthquake excitations. These have been evidenced by the numerous brittle shear failures of columns in the most recent earthquakes, and clearly demonstrated a need for their retrofit.

Various retrofit schemes have been developed for strengthening of concrete columns with seismic deficiency, such as concrete jackets, steel jackets, and fiber reinforced polymer composite jackets. The use of steel jackets or tubes to enhance the strength of columns and to improve deformability has been well studied since the 1980s. In the recent studies, emphasis has been directed on the confinement effect of the steel jackets to column concrete. Tomii, Sakino, and Xiao (1987) investigated steel-tube short columns in building structures as a scheme to prevent shear failure and to improve ductility. In their study, gaps were deliberately located from the steel tube ends to the column ends to avoid the buckling of the steel tube. It was found that this scheme was ideal for circular columns, but a deterioration of strength and stiffness was inevitable for rectangular columns, particularly for columns with a compression ratio exceeding 0.30.

Chai et al. (1994) proposed a circular/elliptical jacket method to retrofit square/rectangular columns. In their studies, the square/rectangular columns were enclosed by grout-injected circular/elliptical jackets to enhance the shear strength of the columns and to improve their deformability. This technique, however, is not always suitable when significant changes to the size or shape of the column are not desirable, and the augment of the section may attract more seismic force. Xiao et al. (2003) developed another improvement jacketing method to retrofit square columns by using welded rectilinear steel jackets and stiffeners. Test results validated the efficiency of the partially stiffened rectilinear steel jacketing, which not only prevented brittle shear failure but also greatly improved the ductility of the column with achieving an ultimate drift ratio of more than 8%. Wu et al. (2003) investigated the composite partial interaction retrofit method to improve the strength and ductility of rectangular reinforced concrete columns. Their study results confirmed that the approach could successfully delay concrete crushing in the plastic hinge region and the strength and ductility of concrete columns could be increased.

Though the efficiency of steel jacket retrofitting RC columns is confirmed by a wealth of experimental studies, problems still exist as follows: (1) severe stress-lag between the retrofitted part and the original part usually exists; (2) the stress transfer capacity between the interface of the retrofitted and the original part depend to a large extent on the performance of the composite grout injected in the gap between the steel jacket and the concrete core of the retrofitted column; and (3) the steel jacket acts as passive confinement, and the confinement action lies on the dilation of the concrete during subsequent loading or on the dilation of flexural compression zone under transverse earthquake excitations.

Externally applied lateral prestressing of RC columns or piers not only improves the mechanisms of transverse confinement, but serves a good approach to solve those problems denoted above as well. Researches on external prestressing of RC columns or piers as a seismic retrofit methodology have been reported sine 1990s. Experimental investigation had been underway at the Structures Laboratory of the Univ. of Ottawa sine 1993 to develop a new retrofit technique for improved seismic shear resistance of existing concrete bridge columns (Yalcin 1997). The results indicated that the retrofit methodology could suppress shear failure, promote flexural behavior, and increase inelastic deformability substantially (Saatcioglu 2003).

Munawarz (2004) investigated into the behavior of externally confined columns under monotonic concentric loading and simulated seismic loading, with and without axial load. In their studies, a significant initial pretensioning force was applied to the bolts of the external steel hollow structural sections (HSS) collar. The test results showed that the effective core area of externally confined columns was significantly larger than that of conventional columns and could be taken as the full cross sectional concrete area, and the collared columns showed very good seismic behavior under severe cyclic loading.

From the limited experimental studies, it is recognized that the technique of transverse prestressing of columns or piers is a viable seismic retrofit technique of RC structures. Though large efforts have been made to confirm the seismic retrofit effect of this external prestressing technique, further study is still needed to make this retrofit technique more engineering practicable and to rationally take into account the transverse prestressing mechanism in estimating the seismic performance of a retrofit RC column.

2 EXPERIMENTAL PROGRAM

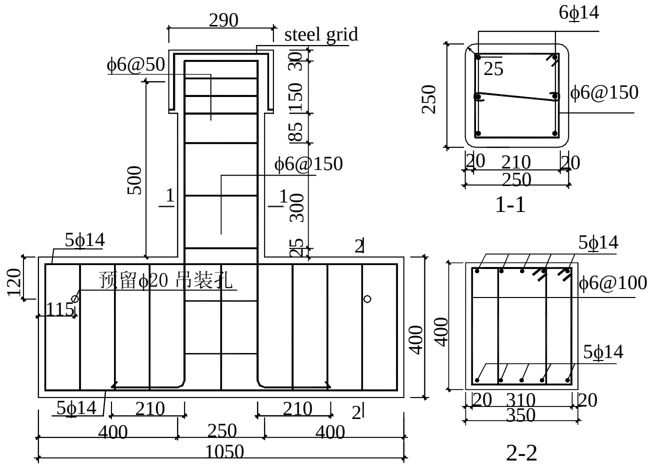
2.1 Description of Specimen

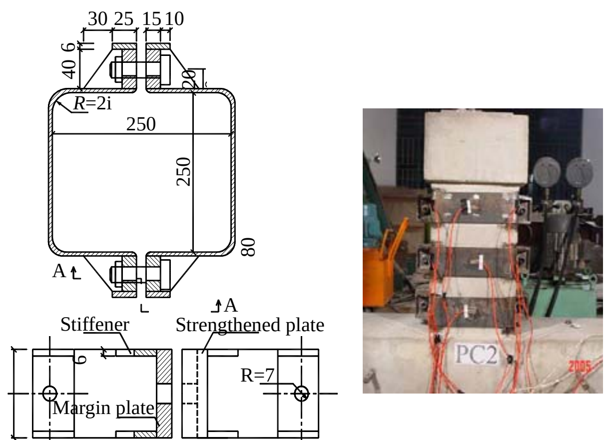
The retrofitted objective of the present test involved those RC short columns which were designed seismically deficient and a shear failure rather than flexural failure would be expected. Six half-scale column specimens involving 1 “as built” and 5 novel prestressing steel jacket retrofitted columns were designed. The details of the specimens with the same geometry size and reinforcement configuration were given in Fig. 1. The main test variables included prestressed level α and axial compressive ratio n, in which α represented ratio of the prestressed jacket strain to jacket yield strain. The corresponding test variables of specimens were listed in Table 1.

Table 1 Main test variables

No.	$n'(n)$	N(kN)	α
PC1-29	0.29(0.42)	234	—
PC2-29-55	0.29(0.42)	234	0.55
PC3-44-35	0.44(0.66)	354	0.35
PC4-53-35	0.53(0.80)	434	0.35
PC5-29-15	0.29(0.42)	234	0.15
PC6-29-35	0.29(0.42)	234	0.35

The details of the steel plate hoop (SPH) used in the present test is shown in Fig. 2, and a uniform configuration and dimension of SPH is adopted in all six retrofitted specimens. Transverse prestressing of SPH was applied by screwing the two high strength bolts on the opposite side of a SPH simultaneously with spanners, and a real time data of strain gauges on the SPH surface was monitored to control the prestressed level of SPH.





(a) Details of steel plate hoop

(b) Retrofitted specimen

Fig.1 Details of specimens (mm)

Fig.2 Details of steel plate hoops and its configuration

2.2 Material Properties

To reflect the character of low strength in an aged structure, the cubic compressive strength of specimens was controlled under 20.0 MPa. The average cubic compressive strength was 19.70 MPa after 28 days standard curing. The mechanical properties of the steel were listed in Table 2.

Table 2 Material properties of steel

Material type	f_y(MPa)	f_b(MPa)	E_s(MPa)	ε_y(με)
ϕ14	351.6	569.9	2.0×10^5	1758
ϕ6	393.4	593.7	2.1×10^5	1873
Jacket (t=4)	337.6	451.0	1.46×10^5	2306

2.3 Test Setup and Loading Procedure

Axial load was applied to the column end by a vertical jack, and a horizontal load was imposed by a bidirectional jack fixed on the reaction frame. The test setup is shown in detail in Fig. 3.
The loading procedure is as follows: First, a predetermined axial load was applied to the column, and then the transverse load was applied. The axial load was held constant during the application of the transverse load. The horizontal loading process was controlled by a displacement method. Before the drift ratio of specimens reached 1/75, one cyclic loading was performed under each displacement level of 1/500, 1/200, and 1/150; after the drift reached 1/75, the horizontal load was applied according to the following drift amplitudes, 1/75, 1/50, 1/35, 1/25, 1/20, and 1/15, and at each drift level, three loading cycles were performed. The whole loading procedure finished when the bearing capacity of the specimen was reduced to 85% of the maximum load (P_{max}) or the hysteretic curves appeared unstable.

2.4 Instrumentation

The horizontal and vertical loads were measured using two calibrated load cells. The horizontal displacement was measured using two LVDT with a travel stroke of 100 mm, which were mounted on both sides of the column top. The strain gages were used to measure the strains of steel bars, stirrups, and outside jackets. The detailed layout of all the strain gauges is shown in Fig. 4.

Fig.3 Test setup

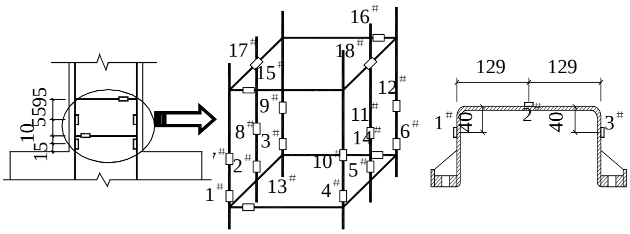


Fig.4 Layout of the strain gauges

3 EXPERIMENTAL RESULTS AND DISCUSSION

3.1 Final Failure Modes

It can be easily observed that the occurrence of shear crack in those retrofitted specimens were suspended when compared to the as built specimen's (PC1), and that a increasing of the prestressed level of SPH in those retrofitted specimens can further suspend the occurrence of shear crack.

Although all the specimens reached their maximum bearing capacity after the longitudinal bar yielded, the final failure modes varied according to whether retrofitted or the prestressed level of the SPH (Fig.5). For specimen PC1, a main diagonal crack developed quickly after yielding of the longitudinal reinforcing bars, and a typical shear failure shortly happen during the loading cycle of 1/25 drift ratio(Fig. 5a). For the retrofitted specimens PC2, PC3, PC4, and PC6, whose prestressed level α is larger than 0.35, the final failure modes were bending failure mode, although many shear crack were found in the critical region of the column(Fig. 5b). For the retrofitted specimen PC5, whose prestressed level α is only 0.15, the final failure modes showed bending failure mode with severe transverse expanse of concrete in the mid-height of the column (Fig.5c), which finally resulted the yield of SPH.

(*a*) (*b*) (c)

Fig.5 Ultimate failure modes

3.2 Strains of Reinforcement and SPH

3.2.1 Strain of SPH

Strains of SPH developed in good agreement with the global failure mode of the specimens. Fig.6 shows different relationships of story shear and strains of SPH with different prestressed level in the retrofitted specimens. From Fig.6a, the SPH strain of PC5 with low prestressed level(α =0.15) attained yielded strain at a displacement level of $4\Delta_y$, and increased with the development of displacement amplitude. This result is consistent with the severe transverse bulging failure phenomenon in PC5 (Fig. 5c). For the other retrofitted specimens PC2, PC3, PC4, and PC6, whose prestressed level α is larger than 0.35, because the core concrete of the columns were all effectively confined by PSPH, the strains of SPH still remained in elastic range when they reached ultimate failure (Fig.6b), and no obvious transverse deformation were observed during the entire test process even under a high axial load (Fig. 5b).

3.2.2 Strains of stirrups

The hysteretic curves of stirrup strains of specimens are shown in Fig.7. The stirrup strain of the as built specimen PC1 increased abruptly when the drift ratio reaches 1/200. The stirrup strain developed continually under subsequent loading and yielded when the drift ratio reaches 1/75 (Fig.7a). From Fig. 7, it can be found that the stirrup strains of the retrofitted specimens developed slowly than that of the as built specimen PC1.

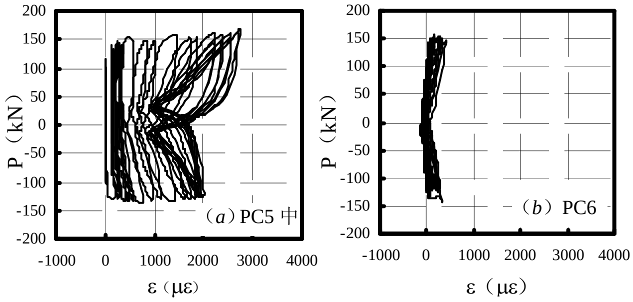


Fig.6 Load-strain hysteretic curves of plate hoops

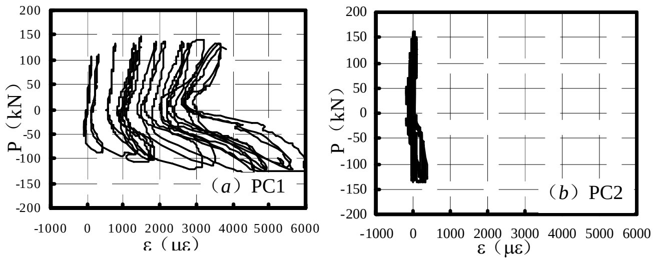


Fig.7 Load-strain hysteretic curves of stirrups

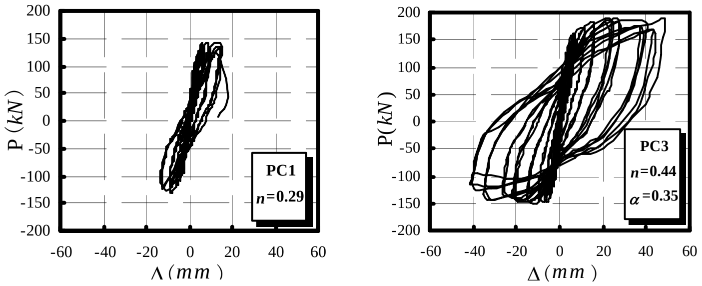
3.3 Hysteretic Characteristics and Skeleton Curve

3.3.1 Character of the hysteretic curves

The typical hysteretic curves of the as built specimen and retrofitted specimen are shown in Fig.8. The hysteretic loop of un-retrofitted specimen PC1 appeared the arc shape and the pinching in the curve was apparent because the open and close of the crack during loading and unloading (Fig.8a). The hysteretic loop of the retrofitted specimens shows a favorable plump model (Fig.8b) and indicates a satisfactory ductility and hysteretic energy dissipation capacity of the present retrofitted technique. The degradation of strength and stiffness in PC1is severe after the loading cycles of 1/35 drift ratio. Compare to the retrofitted specimens (Fig.8b), the ductility and energy absorption capacity of the un-retrofitted specimen PC1 is less desirable.

3.3.2 Skeleton curves

The *P-Δ* skeleton curves of the specimens are described in Fig. 9, and the experimental results of the key points for all of the skeleton curves are shown in Table 4, where the units of load and displacement are kN and mm, respectively, and the variables P_c and Δ_c respectively represent the load and the displacement of the specimens when the shear crack first occurs.



(*a*) (*b*)

Fig.8 Hysteretic curves of the specimens

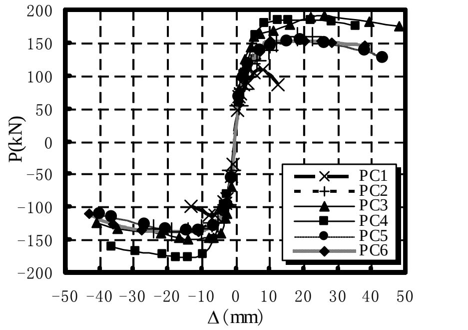


Fig.9 Skeleton curves of the specimens

Fig. 9 illustrates that the skeleton curves for those retrofitted specimens experienced a longer plastic phase and shows a much better ductility than that of un-retrofitted specimen PC1. Note that there exists a distinct difference between specimens PC1 and retrofitted specimens PC2-PC6. PC1 how a relatively short post-yielded strengthened phase than that of PC2-PC6. The failure mode of PC1 is obviously shear failure, and thus their strength degenerates earlier than the other specimens, and an increase in the prestressed level of SPH leads to a reduction in strength degradation.

From the Table 4, it can be found that the deformability of the columns with substandard shear strength and the shear details could be largely improved by the prestressed SPH scheme. The ultimate deformation and displacement ductility ratio of the retrofitted specimens increased over 3 times than that of the unretrofitted specimen. The specimen PC4 still showed a favorable ductility even when the design compressive ratio reached 0.8.

Table 4 Test results of key points in skeleton curves

Specimens	n	ε	P_c	Δ_c	P_y	Δ_y	P_m	$P_{m,i}/P_{m,1}$	P_u	Δ_u	$\Delta_{u,i}/\Delta_{u,1}$	$\mu=\Delta_u/\Delta_y$	μ_i/μ_1
PC1	0.28	−	57	1.0	106	3.80	125	1.00	120	8.68	1.00	2.28	1.00
PC2	0.28	0.55	90.6	2.5	117	3.80	138	1.10	30	25.43*	1.93*	6.70*	2.94*
PC3	0.44	0.35	103.5	2.5	141	5.45	151	1.21	127	40.00	4.60	7.33	3.21
PC4	0.53	0.35	84.5	1.47	150	6.61	177	1.37	153	37.15	4.28	5.60	2.46
PC5	0.28	0.15	28	0.64	110	3.73	138	1.10	117	28.39	3.27	7.61	3.34
PC6	0.28	0.35	41	0.65	111	3.76	136	1.09	119	40.14	4.62	10.67	4.68

Note： *the measuring process of PC2 was interrupted due to a too small travel stroke of LVDT.

4 CONCLUSION

(1)Prestressed SPH could efficiently postpone the occurring and mitigate the developing of diagonal cracks in substandard shear details short columns, and improve the failure mode of shear deficiency columns to obtain a favorable seismic behavior when seismic retrofitted is needed. This retrofitted technique is still effective even if the compressive ratio of column is high.
(2)The displacement ductility ratio of the retrofitted specimens increased over 3 times than that of the un-retrofitted specimen, and the ductility ratio could be further enhanced by increasing the prestressed level.
(3)The prestressed SPH could effectively confine the column concrete and thus a favorable seismic behavior can be expected. Nevertheless, the expected retrofitted efficiency diminished when the prestressed level of SPH reduced to 0.15.

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