SEISMIC PERFORMANCE ASSESSMENT AND RETROFIT OF RECTANGULAR BRIDGE PIERS WITH EXTERNALLY ENCASED CIRCULAR STEEL JACKETS

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SUMMARY

In this paper, the experimental results of four bridge piers retrofitted with externally encased steel jackets are presented in detail. These piers have a 1/5 scale factor and were designed according to the current seismic design codes for bridges in China that contains no provision on good ductility and seismic performance for piers. Design variables include concrete strength, axial load level, volume of transverse reinforcement and so on. Tests on all four piers are completed in two stages. First, all piers are tested under cyclic loading to an excessively damaged state and their seismic performance is assessed. Then these piers are retrofitted with externally encased circular steel jackets and retested to total failure under the same loading condition. Ductility, strength, energy dissipation capacity and the lateral jacket strain are investigated. Test results indicate that retrofitting rectangular bridge piers with steel jackets can significantly improve the displacement ductility and energy dissipation capacity due to the firm confinement provided by the jacket and can also increase the strength and stiffness of the piers to some degrees.

INTRODUCTION

Bridges designed according to the out-of-date codes are often found vulnerable to seismic strikes as observed during the past several earthquakes [Lee, 1990; Na *et al*, 1995]. This necessities the need for assessing the seismic performance of these bridges and conducting seismic retrofit program. During the last decade, several comprehensive research programs pertaining to assessing ductility, strength and retrofitting of bridge columns have been initiated in New Zealand, the United States and Japan. Many retrofitting techniques have been developed as a consequence, including steel jacket, concrete jacket, composite material jacket and so on.

The technique of steel jacketing was originally developed to retrofit circular bridge columns [Chai, 1991]. Two halves of steel shells are made slightly oversized with a clearance of 5-25mm for ease of construction and welded in-situ along the vertical seams. The gap between steel jacket and column concrete is filled with cement grout, which may contain some additives to make the grout dilate slightly so that some initial stress can be introduced and interaction between steel jacket and column can be improved. Typically, a gap of 20 to 50mm is left between the jacket and footing or cap beam to avoid the possibility of additional strength increase resulting from bearing of the jacket against its supporting members. On the other hand, column strength is indeed increased significantly due to the passive confinement provided by steel jackets. Under the combined action of axial and lateral loads, the concrete in compression zone tends to dilate but is restrained by the steel jacket. Consequently, the ultimate strain of core concrete and curvature of critical sections are increased.

For rectangular columns, it is a good practice to be retrofitted by oval or circular steel jackets. The large gap between steel jacket and column is filled with normal weight concrete after flushing with water. Because of the continuous curvature and membrane tension, better ductility and energy dissipation capacity of columns are expected. However, retrofitting rectangular columns with rectangular jackets to improve flexural ductility has been proved to be unsuccessful [Aboutaha, 1996].

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In this study, four 1/5-scale rectangular bridge piers were cast in lab and tested to failure under cyclic loading. Then all piers were rehabilitated and retrofitted with circular steel jackets. The piers have an aspect ratio of 5.0 and are expected to fail in a ductile mode. They were thus retrofitted only in a length of twice the width of the cross section. Test results indicate that the retrofitted piers have performed exceptionally well and steel jacket retrofitting is a viable technique for improving the seismic performance of substandard or deficient bridge piers.

TEST SPECIMENS AND SETUP

Four 1/5-scale RC bridge piers were cast in lab. They are 100cm high and have a cross section of 20x20cm as shown in Fig. 1. Two of them, called NP1 and NP2, have a concrete strength of 38.2Mpa and 37.2Mpa respectively. The other two, HP3 and HP4, have a higher concrete strength of 71.5Mpa. They were so designed that effectiveness of steel jacketing in retrofitting both normal and high strength concrete piers can be investigated.

The longitudinal deformed rebars, either 1cm or 1.2cm in diameter, have the yield and ultimate strength of 387.0Mpa and 538.0Mpa. They are evenly distributed around the perimeter of the cross section with a center to center spacing of 5.0cm. Table 1 summarizes the design parameters and material properties of the specimens. The piers designed according to the current seismic design codes of bridges in China except for a higher volumetric ratio for transverse reinforcements than the specified minimum value of 0.3% by the codes in order to obtain better confinement and ductility.

All specimens were tested under cyclic lateral loading at a moderate-to-high axial load level (Fig. 2). Lateral loads were applied with displacement control generated by the universal test machine. The cyclic displacement input is progressively increasing as shown in Fig. 3. It increases 2mm and 5mm respectively before and after it reaches the amplitude of 10mm. Three cycles are repeated at each stage until severe damage or rapid strength degradation is observed. However only two cycles is used when the displacement amplitude is less than 10mm. LVDTs were symmetrically placed in the potential plastic hinge areas and at middle height of the pier and the loading point.

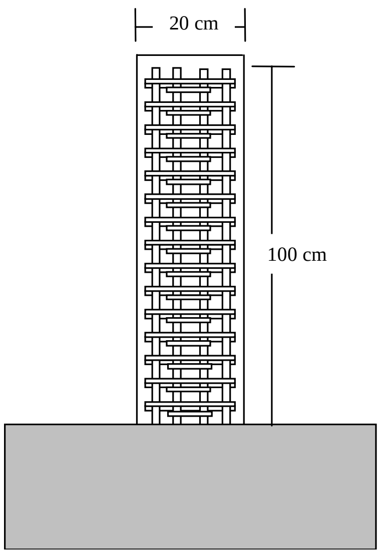


Fig. 1 Detailing of test pier

Fig. 2 Test setup

Table 1 Design parameters and material properties of piers

Pier	Concrete strength $f_c^{'}$ (Mpa)	Main rebar diameter d(mm)	Volumetric ratio of transverse hoops ρ_s (%)	Axial load level P/($f_c^{'}$A)	Remark
NP1	38.2	10	1.13	0.36	Transverse hoops: 6mm in diameter
NP2	37.9	12	0.81	0.15	
HP3	71.5	10	1.22	0.15	
HP4	71.5	10	0.81	0.15	

Damaged piers were then repaired and encased with 2mm thick, 40cm long circular steel jacket. The internal diameter of steel jacket is 29.5cm. After removing the loose concrete in the damaged areas and readjusting the reinforcements, the empty area was filled with concrete of the same strength as the corresponding virgin pier. A gap of 1cm is set between the lower end of steel jacket and footing surface. Then the retrofitted piers were subjected to the same load conditions as the virgin piers. The four retrofitted piers are referred to as NP1R, NP2R, HP3R and HP4R respectively.

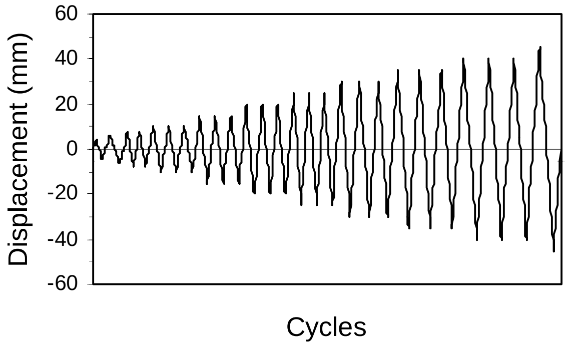
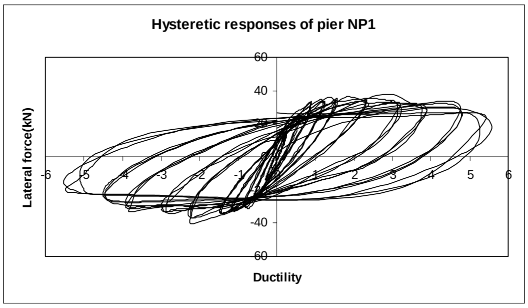


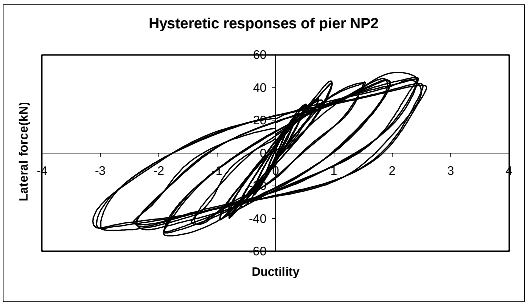
Fig. 3 Input displacement cycles

TEST RESULTS

The load-displacement relations of the four virgin piers are presented in Fig. 4. All hysteresis loops appear stable and fat, indicating that these piers can substantially dissipate energy under earthquake loads due to sufficient transverse reinforcement. It can be seen that HP3 and HP4 degrade in both strength and stiffness more rapid than NP1 and NP2 do. More stringent requirement in term of confinement is therefore necessary to improve the seismic performance of high strength concrete.

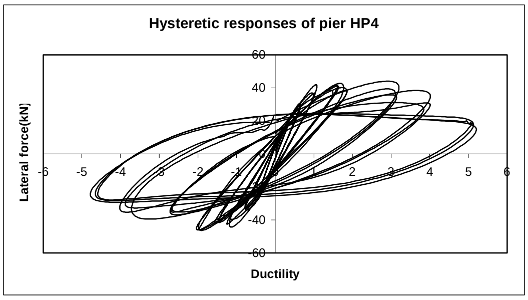
The test results also indicate that HP3 has a ductility factor of 4.0, slighter higher than that (3.7) of pier HP4. This is because HP3 has smaller spacing between transverse reinforcements and therefore stronger confinement as indicated in Table 1. NP1 and NP2 have a ductility factor of 5.5 and 4.0 respectively. The spacing of transverse reinforcements for pier NP1 is only 30mm, resulting in a volumetric ratio of 1.13%. As a result, good ductility performance is obtained even though NP1 has a high axial load level. It is shown that longitudinal reinforcement doesn’t have significant contribution to the improvement of ductility expect for the obvious enhancement in strength.

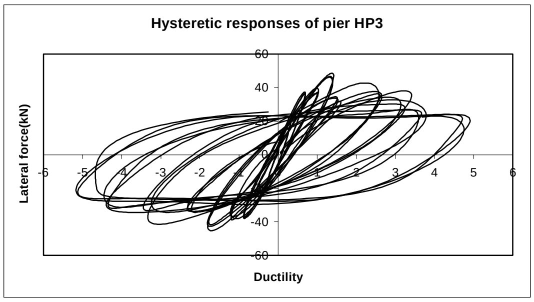




(a) Pier NP1

(b) Pier NP2





(c) Pier HP3

(d) Pier HP4

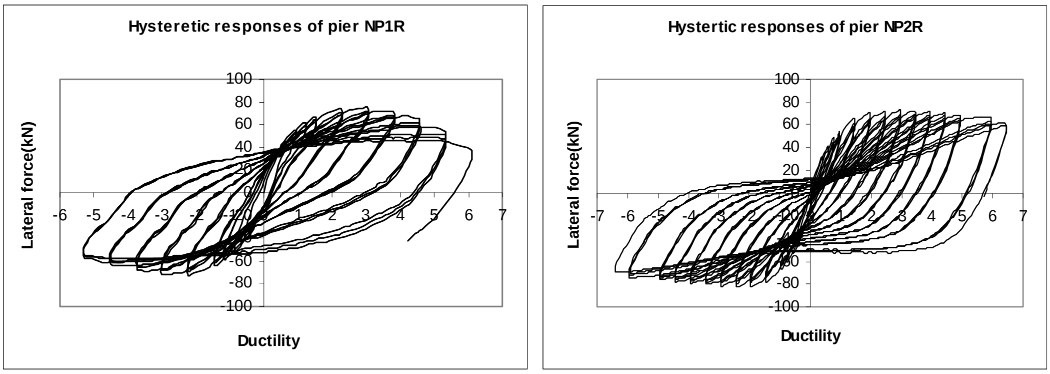
Fig. 4 Hysteretic responses of virgin piers

After retrofitting, the cross section is enlarged and strength increase is observed. The strength of both virgin and retrofitted piers in the push and pull modes under cyclic loading is listed in Table 2. It is seen that the strength of the piers increases appreciably in both directions. In addition to the enlargement of cross section, the strength increasing may be attributable to the confined concrete strength from steel jacketing (Mander *et al,* 1988).

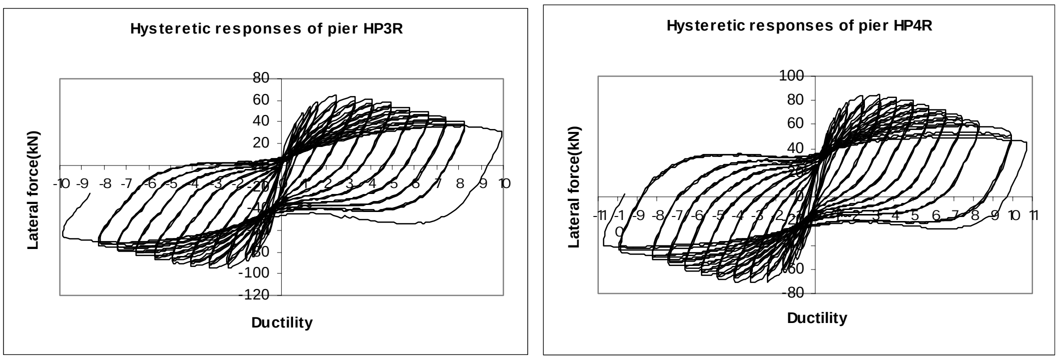
Table 2 Strength of virgin and retrofitted piers

Virgin pier	Loading cycle	NP1	NP2	HP3	HP4
Maximum lateral load(kN)	Push	36	50.6	45.2	46.3
	Pull	36.9	45.6	48.0	43.5
Retrofitted pier	Loading cycle	NP1R	NP2R	HP3R	HP4R
Maximum lateral load(kN)	Push	75	72.8	64.1	84.4
	Pull	72.4	82.4	95.0	71.2
Increase(%)	Push	108	44	42	82
	Pull	96	81	98	64

The corresponding load-displacement curves are plotted in Fig. 5 for the four retrofitted piers. It can be observed that both ductility and energy dissipation capacity are greatly increased with steel jacket retrofitting even for the high strength concrete piers. The more stringent confinement requirement for high strength concrete piers, concluded from the testing results of the virgin piers, can be met in this way. A ductility of 6.0 is obtained for piers NP2R, HP3R, HP4R and the responses are very stable. But for pier NP1R, the ductility is recovered successfully with significant increase. This is because NP1R has a higher axial load compared with the others.



(a) Pier NP1R (b) Pier NP2R



(c) Pier HP3R (d) Pier HP4R

Fig.5 Hysteretic responses of retrofitted piers

During the test, three strain gages were attached to the outer surfaces of steel jackets for specimen NP1R and NP2R. They are oriented along the perimeter of the jackets and at 5cm, 15cm, and 35cm form the bottom of jackets. The measured hoop strains under different stages are shown in Fig. 6. The strains at two ends of jackets are very small, indicating weak interaction and confinement in these areas. At the middle height, the strain is much higher showing good interaction between steel jacket and concrete. However, the strain is significantly smaller than the yield strain of material. Some measurements need to be taken to improve this issue in the future in order to make this technique more efficient.

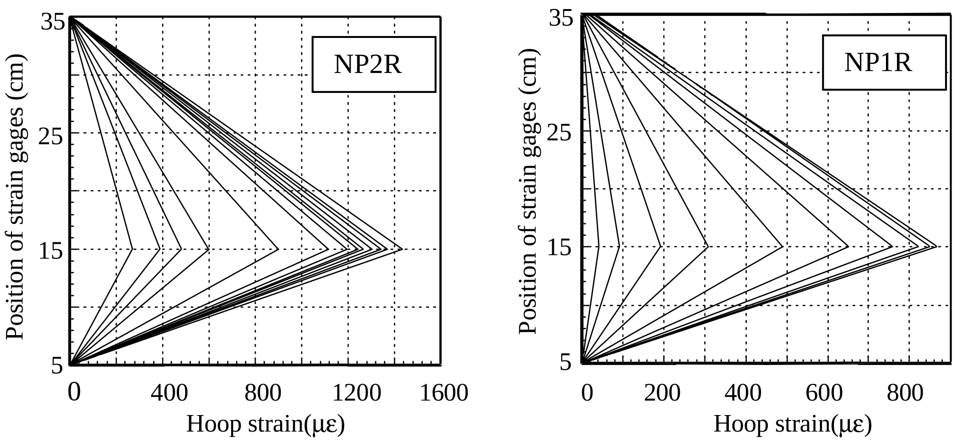


Fig. 6 Strain distribution along jacket

CONCLUSIONS

The following conclusions can be drawn from this study:

1. Confining effect by transverse reinforcement plays an important rule on the seismic performance of RC bridge piers.

2. Influence of main reinforcement on the ductility of bridge piers is insignificant.

3. High strength concrete has stricter requirement on the volume of transverse reinforcement than normal strength concrete to obtain higher ductility. A volumetric ratio of 1.22% is not enough for high strength concrete at high axial load level.

4. Steel jacket retrofitting is a viable technique for repairing and retrofitting RC bridge piers for enhanced seismic performance.

5. If circular steel jackets are used to retrofit rectangular columns, it is recommended that the columns be retrofitted in full length.

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