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SEISMIC RETROFITTING OF CONCRETE COUPLING BEAMS BY STEEL PLATE WITH OR WITHOUT STIFFENERS

B. Cheng&C. Shi&C.H.Shi&M.J.Zhou
Assistant Professor, Department of Civil &Transportation Engineering, Beijing University of Civil Engineering and Architecture, China

ABSTRACT: Coupled beams in coupled shear walls are very important structural components that provide the necessary lateral strength, stiffness and deformability for the whole building to resist extreme wind and earthquake loads. In past decades, the design of many concrete buildings in China and Hong Kong has not taken into account earthquake actions. Following the introduction of the new design codes, many existing coupling beams are found to be deficient in shear capacity. In this study, experimental studies were conducted on a new retrofitting method with an unstiffened or stiffened laterally restrained steel plate (LRSP) for existing deep RC coupling beams. It can be found that for the retrofitting method with unstiffened LRSP, the deformability and energy dissipation of the retrofitted coupling beams were enhanced by utilizing the post-buckling loading capacity of steel plate. However, the occurrence of early plate buckling usually results in reduced strength, stiffness and energy dissipation capacity accompanied by significant pinching. For the retrofitting method with stiffened LRSP, the deformability and energy dissipation of the retrofitted coupling beams were also enhanced because the additional stiffeners can prevent plate buckling and ensure that the steel plate has a wider yield area and hence higher energy dissipation.

Keywords: Deep coupling beams, seismic retrofitting, laterally restrained steel plate, with or without stiffener

1 INTRODUCTION

Reinforced concrete (RC) coupled shear walls and core walls are widely employed as a lateral load resisting system for high-rise buildings to resist earthquake and wind loads. In this system, a number of individual wall piers are coupled together by coupling beams to increase the lateral strength and stiffness of the buildings. To ensure the desired behaviour of coupled core walls, seismic resistant coupling beams should be sufficiently strong, deformable and have good energy dissipation ability [1]. In past decades, the design of many concrete buildings in moderate seismicity regions, such as Hong Kong, Bangkok, etc. have not taken into account earthquake actions. Following the introduction of the new design codes, many existing coupling beams are found to be deficient in shear capacity. Paulay [2] has pointed out that deep reinforced concrete (RC) coupling beams are prone to brittle failure in the form of diagonal or sliding failure when insufficient shear reinforcement is used. Under strong earthquake loads, brittle failures of these coupling beams could significantly affect the structural safety of the entire building. To improve the seismic resistance of existing buildings, many coupling beams deficient in shear or lacking in deformability need to be retrofitted.

However, little research had been conducted aiming at improving the seismic performance of existing reinforced concrete coupling beams. Harries et al. [3] studied a shear strengthening method for existing coupling beams with a span-to-depth ratio of 3.0. In their study, the retrofitting involved a number of different attachment methods to fix the steel plate to one side of the coupling beams. They found that the hybrid method of bolting and epoxy bonding to attach the steel plates both in the span and at the ends performed the best. Su and Zhu [4] studied a shear strengthening method for RC coupling beams with a span-to-depth ratio of 2.5. They bolted the steel plate to both ends of the wall panels without adhesive bonding. Their experimental studies showed that this retrofitting method could greatly increase the shear capacity of medium length coupling beams, while fastening the retrofit plate to the span of a concrete beam could prevent local buckling of the steel plates, but this led to serious concrete damage at the failure stage. In all their experiments, minor buckling of steel plate was observed and the influence of local buckling on the behaviour of composite coupling beams was not investigated. However, most of the previous studies focused on the coupling beams with span-to-depth ratios larger than 2.0. Many coupling beams above the openings are rather short and deep, while the research about retrofitting method of deep coupling beams is rare. Su and Cheng [5, 6] experimentally studied the use of a laterally restrained steel plate (LRSP) without stiffeners to retrofit deep concrete coupling beams with a span-to-depth ratio of 1.1. In their test, thin steel plates were utilized. The steel plate started to develop a diagonal tension field after the onset of global buckling at the early stages of loading and exhibited nonlinear behavior at relatively small inter-story drift ratios. Due to the post-buckling loading capacity and tension field action in the steel plate, LRSP retrofitted coupling beams failed in a ductile manner. However, plate buckling is always accompanied by significant pinching and stiffness degradation in the hysteresis response of structures. Cheng and Su [7] conducted a numerical parametric study to investigate the influence of plate buckling on the LRSP retrofitted coupling beams. One approach of increasing the carrying capacity of the steel plate is to add some appropriate stiffeners. In this way, the retrofitted coupling beams could achieve its full plastic strength and avoid overall buckling. Therefore, the steel plate can resist large lateral forces and dissipate the energy induced by earthquakes. In this study, the experimental results of a series of LRSP retrofitted coupling beams with or without stiffeners are compared and the performances of specimens with or without stiffeners are investigated.

2 EXPERIMENTAL PROCEDURES

2.1 Description of Test Specimens

Six specimens with the same dimensions and reinforcement specifications (see Fig. 1), but different retrofitting schemes, were fabricated and tested. The retrofitting schemes of all the specimens are shown in Table 1. The concrete compressive strength of DCB1, DCB2, DCB5 and DCB6 is about 35Mpa. The concrete compressive strength of DCB10 and DCB11 is about 60Mpa. The yield strength of all the steel plate is about 350Mpa. The first specimen DCB1 with a plain RC arrangement was used for control purposes. Specimen DCB2 was retrofitted with a 3 mm grade 50 steel plate without buckling controlled device. While Specimens DCB5 and DCB6 were retrofitted with 3 mm and 4.5 mm steel plates, respectively, with a buckling control device was mounted onto the beam span. By providing two steel angles (L70×70×5mm) along the top and bottom edges of the steel plate, the possible lateral buckling of the steel plate in the span at the edges was suppressed. To avoid adding extra strength and stiffness to the composite coupling beam, the lateral stiffeners were connected to a steel plate by four bolt connections with slotted holes which allowed the two lateral stiffeners to freely rotate and move in the longitudinal direction.

Stiffeners were all added symmetrically along the span of coupling beams of Specimens DCB10 and

DCB11. Stiffeners are structural elements connected to the steel sheet by continuous fillet welds. Rigid stiffeners are used to ensure that the plate can reach its full plastic strength and avoid overall buckling. Horizontal or diagonal stiffeners are adopted. Rectangular steel plates with a width and a thickness of 20 mm and 9 mm, respectively, are selected as horizontal stiffeners and steel rebars with a diameter of 10 mm are used as diagonal stiffeners.

To avoid plate buckling in the anchor regions, thicker steel plates (which are twice the thickness of the plate in the span) were used in the anchor regions. Dynamic set bolt connection were adopted which minimizes any possible slippage between various components at the connections by injecting adhesive to fill the gaps between the bolt shank and surrounding concrete.

2.2 Test Setup, Loading Procedure and Instrumentation

The load frame shown in Figure 4, designed by Kwan and Zhao [8], was used in the tests. The specimens were tested under reversed cyclic loading according to the details of test setup and loading procedures by Su and Cheng [5].

The beam rotations (θ), defined as the differential displacement between the two beam ends (Δ) in the loading direction divided by the clear span (l), were calculated using the displacements measured by the linear variable displacement transducers (LVDTs) D3 and D4, as illustrated in Figure 2.

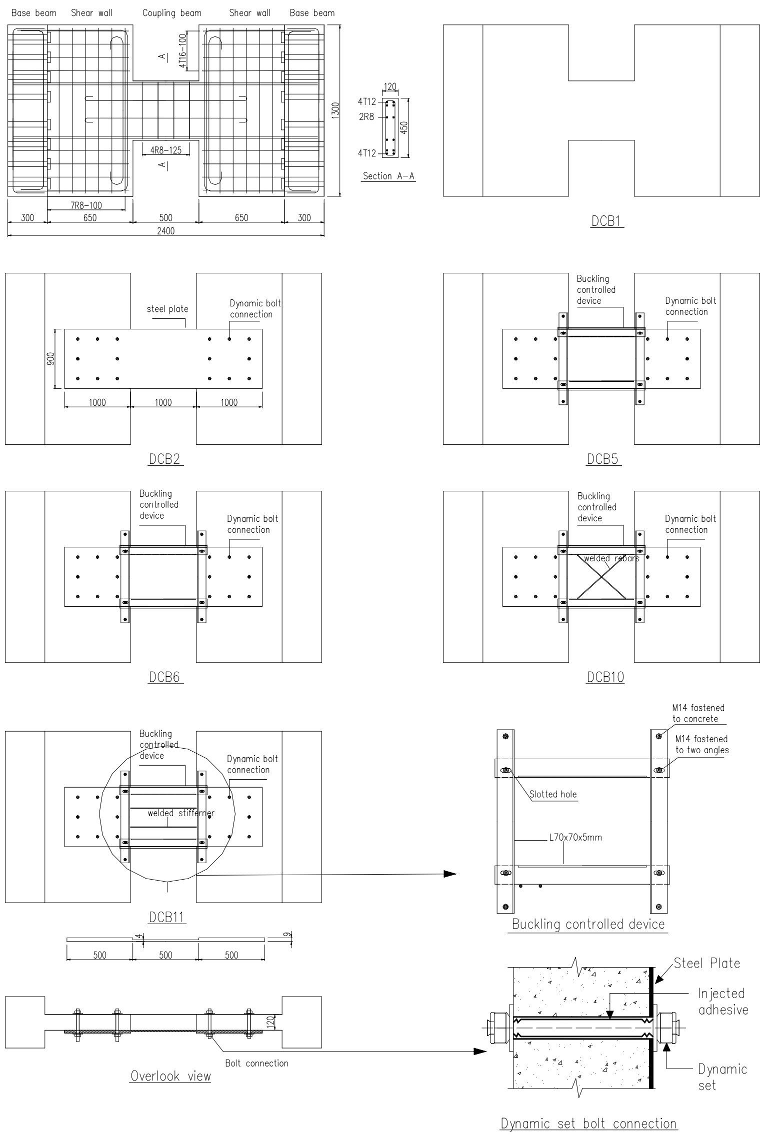


Figure 1. Details of test specimens

Table 1. Coupling beam details

Specimen	Stiffeners	Plate thickness at span	Plate thickness at ends	LRSP method	Bolt property	Type of bolt connection
DCB1	N/A	N/A	N/A	N/A	N/A	N/A
DCB2	N/A	3mm	3mm	N/A	High-tensile steel	Dynamic set
DCB5	N/A	3mm	4.5mm	Added	High-tensile steel	Dynamic set
DCB6	N/A	4.5mm	9mm	Added	High-tensile steel	Dynamic set
DCB10	Two diagonal	4.5mm	9mm	Added	High-tensile steel	Dynamic set
DCB11	Two horizontal	4.5mm	9mm	Added	High-tensile steel	Dynamic set

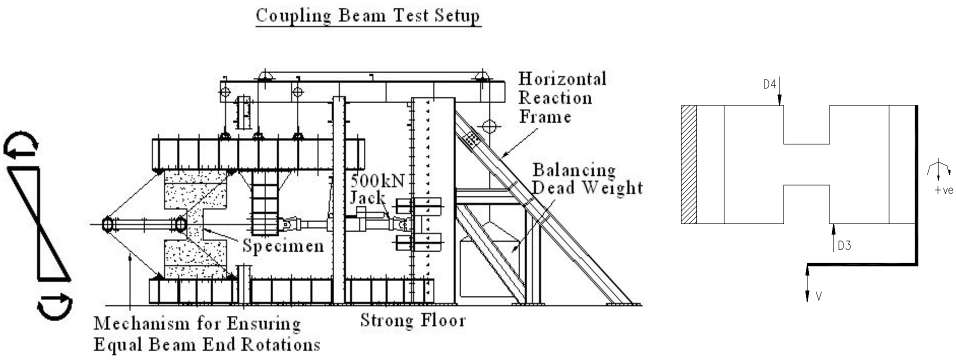


Figure 2. Test setup

3 SUMMARIES OF STRENGTH, DEFORMATION AND DUCTILITY

Several parameters were defined to interpret the results of the tests. The ultimate rotation θ_u is defined by Park [9] as the chord rotation angle at 0.8 V_u of an envelope curve on the softening branch and the yield chord rotation θ_y is defined as the chord rotation angle at 0.75 V_u of an envelope curve divided by 4/3 on the increasing branch. The maximum ductility μ is equal to the ultimate rotation θ_u divided by the yield chord rotation θ_y. As the test values for the positive cycles were not the same as those for the negative cycles, the values from the positive and negative cycles were averaged.

Table 2 shows that the retrofitted steel plates increased both the ultimate capacity and deformability of the coupling beams. The difference between DCB2 and DCB5 is the adding of buckling controlled device. It can be found that the strength and deformation of DCB5 are all increased by comparing the results of DCB2. When a thicker steel plate 4.5 mm was used for DCB6, the ultimate rotation θ_u increased much while the strength only slightly increased compared with DCB5. These results show that by adding a buckling restrained steel plate, the increase in the rotation deformability is much higher than the increase in the strength. The ductility of DCB2 (without a buckling control restraint) was slightly reduced due to the increase in the yield rotation, compared with the control specimen DCB1. The displacement ductility values of DCB5 and DCB6 increased by 30% and 41%, respectively.

By comparing the results of DCB10 and DCB11, the effects of the stiffener arrangement can be investigated. The only difference between these two specimens is the arrangement of stiffeners, two diagonal stiffeners for DCB10 and two horizontal stiffeners for DCB11. The results show that the ultimate strength of DCB10 and DCB11 are near to each other. Therefore, the stiffener arrangement can affect the deformability but not the shear capacity of the retrofitted beams. Diagonally arranged stiffeners can increase the deformability much more than horizontally arranged stiffeners for the retrofitted coupling beams.

The theoretical shear capacity (V^*) of DCB1 determined according to the British Standards is 215kN which is 9% less than the average shear capacity (238kN) obtained from the experiment. However, the theoretical shear capacities (V^*) of the specimens DCB2, DCB5 and DCB6 were found to be 414kN, 412kN and 517kN, respectively, which overestimated the test results by 20 to 45%. The result demonstrates that for the LRSP retrofitted coupling beams without stiffeners, by using the full plastic assumption without taking into account the effects of plate buckling and bolt slips, could cause a very large error in the prediction of shear capacity. While, for the LRSP retrofitted coupling beams with stiffeners (DCB10 and DCB11), this error can be reduced to 10% to 20%.

Table 2. Summary of experimental results

Specimen	Failure Mode	V^* *kN*	V_u kN	θ_y rad	θ_u rad	μ
DCB1	brittle	215	222	0.0043	0.011	2.56
DCB2	brittle	414	344	0.0085	0.019	2.2
DCB5	ductile	412	335	0.0067	0.022	3.3
DCB6	ductile	517	356	0.0067	0.032	4.8
DCB10	ductile	460	411	0.0092	0.0403	4.4
DCB11	brittle	460	400	0.0083	0.0287	3.5

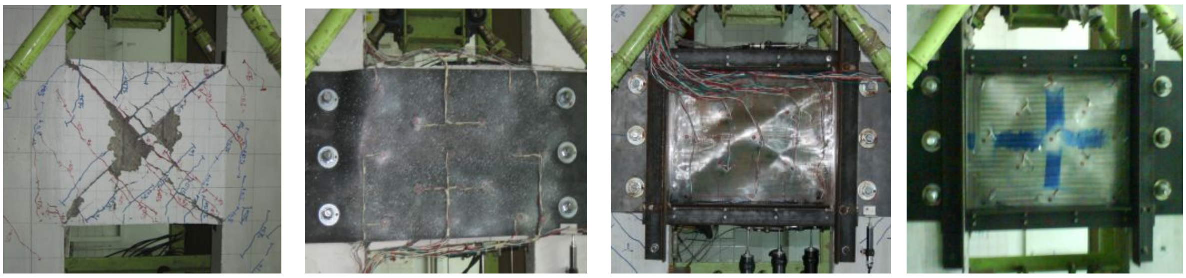
4 LRSP RETROFITTING METHOD WITHOUT STIFFENERS

4.1 Concrete crack pattern and buckling modes of steel plate

The concrete crack patterns of all the test specimens were similar. The extensive diagonal cracks indicate that the shear capacity of the beams was insufficient, as shown in Fig.3. This result agrees with the anticipated brittle shear failure mode as a sufficient amount of longitudinal steel was provided. The wall piers, including the joint regions, only experienced slight damage when the beams failed.

For DCB2, serious local buckling occurred at a rotation of 0.01 rad near the beam-wall joints. After that, the shear strength of the beam increased further. This result revealed that diagonal tensile stresses have been developed in the steel plate, which is also known as tension- field action, for resisting the shear load in the post-buckling stage. The compressive force that was originally assumed by the steel plate was transferred to the concrete. This resulted in more serious concrete crushing at the beam-wall joints. An earlier onset of plate buckling would accelerate the rate of concrete deterioration, explaining why the failure mode of DCB2 was more brittle.

For DCB5 and DCB6, buckling began at a later stage and the plates developed a complete tension-field action after local buckling. For DCB6, with a thicker steel plate, buckling occurred at a rotation of 0.02 rad and the steel plate accommodated more shear force and dissipated more energy as the chord rotations increased. As the loading could be increased further in the post-buckling stage for DCB6, the tension-field effect on the steel plate was also significant. After plate buckling, the buckling deformations at the beam-wall joints could be suppressed by the buckling restraining device and the compressive force transferred to the concrete was smaller than that of DCB2 due to the additional lateral restraint provided by the steel plate. Therefore, concrete crushing at the beam-wall joints was alleviated and the failure modes of DCB5 and DCB6 were more ductile.



DCB1 DCB2 DCB5 DCB6

Figure 3. Concrete crack pattern and buckling modes after the tests

4.2 Load-rotation curves of LRSP coupling beams without stiffeners

Figure 4 shows the load-chord rotation hysteresis loops for the specimens without stiffeners. The load-rotation curve of the control specimen DCB1 exhibited substantial pinching after reaching the peak load. This pinching is associated with a rapid stiffness degradation and reduced energy dissipation in the post-peak regime. For DCB2, the pinching effect after the peak load was also significant as the steel plate could not be effectively activated when buckling occurred at beam-wall joints. For DCB5, the pinching was less serious and for DCB6, which had a thicker steel plate, only slight pinching was observed.

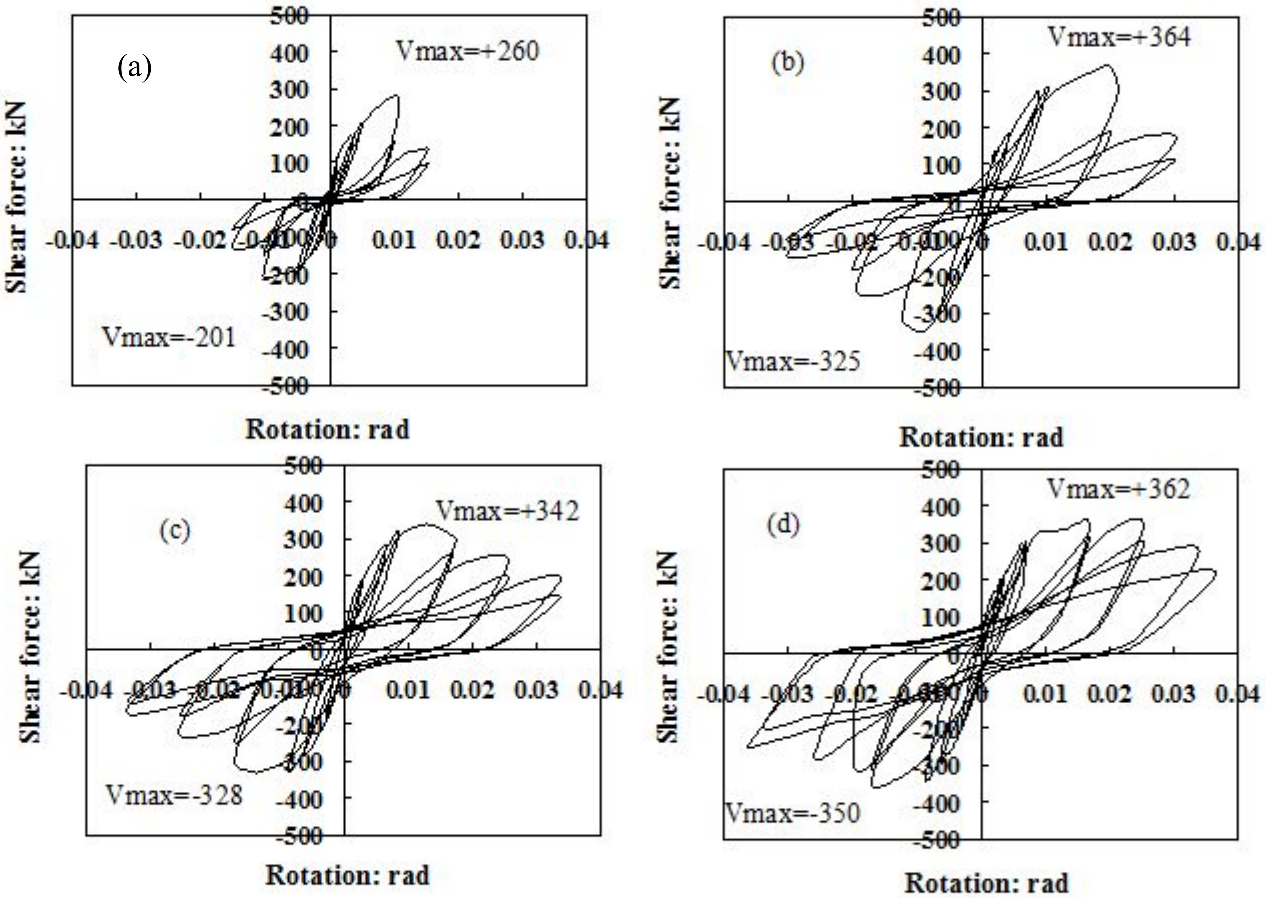


Figure 4. Load-Rotation curves: (a) DCB1; (b) DCB2; (c) DCB5 (d) DCB6

5 LRSP RETROFITTING METHOD WITH STIFFENERS

5.1 Behaviors of steel plate in LRSP specimens with stiffeners

Different type stiffeners are added in DCB10 and DCB11, as shown in Figure5. For DCB10 with diagonal stiffeners and dynamic set connections, the steel plate remained in an elastic state. Diagonal stiffeners helped to delay the diagonal crack opening and resisted much of the compressive force at the beam-wall joints region, which resulted in the alleviation of concrete crushing at the beam-wall joints and the expectation that the concrete beam could resist more shear capacity. Major diagonal cracking occurred at about 0.01 rad and the shear capacity of the concrete beam reduced suddenly. After that, a significant portion of the shear force transferred from the concrete beam to the steel plate. With the rotation increased, for DCB11 with two horizontal stiffeners, due to the larger stiffness of steel plate, more shear force could be resisted. On the other hand, due to elongation of the steel plate with large axial stiffness, more compressive force was applied to the concrete beam which resulted in earlier concrete crushing. This can explain why DCB11 has higher strength but poorer deformability and ductility.

5.2 Load-rotation Curves of LRSP coupling beams with stiffeners

Figure6 shows the load-chord rotation hysteresis loops for the LRSP specimens with stiffeners. For DCB10 and DCB11, it can be seen that the pinching was less serious. Compared with the LRSP specimens without stiffeners, we have found that stiffeners can reduce the pinching effect, resulting in a more stable hysteresis behavior and higher energy dissipation. However, using stiffeners cannot completely mitigate the pinching effect.

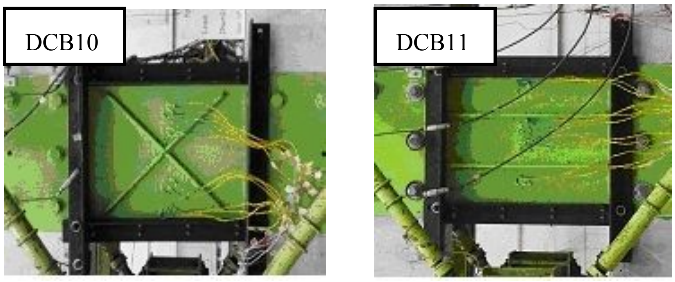


Figure 5. Steel plate with stiffeners

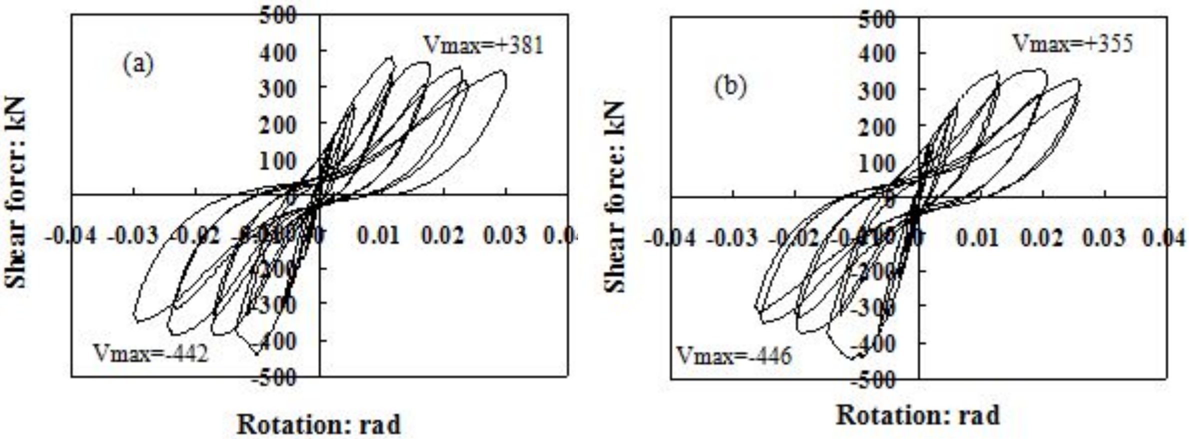


Figure 6. Load-rotation curves: (a) DCB10; (b) DCB11

6 CONCLUSIONS

Experimental study was conducted on laterally restrained steel plate with or without stiffeners for the seismic retrofitting of concrete coupling beams. The main findings of this study are summarized as follows:

1. Steel plate buckling has adverse effects on the structural performances of the coupling beams. The strength could not be further increased when buckling began to occur. Without restraining the plate buckling, buckling can result in a very brittle shear failure mode, high rate of strength degradation and poor energy dissipation.
2. Using laterally restrained steel plates, the deformation, ductility and energy dissipation ability of coupling beams can be greatly increased.
3. The use of laterally restrained steel plate with stiffeners for the seismic retrofitting of concrete deep coupling beams has demonstrated effectiveness in increasing deformability and energy dissipation while reducing strength and stiffness degradation. Also, the retrofitted beams failed in a less brittle manner.
4. The stiffener arrangements also have significant effects on the performances of the retrofitted coupling beams. Providing stiffeners can prevent pinching of the hysteresis curves, increase the shear strength and enhance the energy dissipation capacity. It can be found that an optimum amount of diagonal stiffeners should be used to simultaneously achieve desirable strength and deformability.

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