Seismic performances of concrete coupled shear walls retrofitted with laterally restrained steel plate coupling beams

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Abstract

Existing deep reinforced concrete (RC) coupling beams with low shear span ratios and conventionally reinforced shear stirrups tend to fail in a brittle manner with limited ductility and deformability under reversed cyclic loading. Previous studies have developed a new retrofitting method with unstiffened and stiffened laterally restrained steel plate (LRSP) for existing deep RC coupling beams. By utilizing the post-buckling loading capacity of unstiffened steel plate, the deformability and energy dissipation of the retrofitted coupling beams were enhanced while maintaining flexural stiffness during an earthquake. And by adding additional stiffeners, plate buckling could be prevented which ensured that the steel plate had a wider yield area and hence higher energy dissipation. In this paper, based on the experimental results, nonlinear finite element analysis was conducted by using OpenSees software to accurately predict the seismic behavior of coupled shear walls with or without laterally restrained steel plate coupling beams under different earthquake actions. By comparing the internal force, deformation and energy dissipation of coupled shear walls , it was found that this retrofitting method could improve the seismic performances of coupled shear walls structures under moderate and rare earthquakes actions.

Keywords: **coupled shear walls, coupling beams,retrofitting, laterally restrained steel plate, seismic performance**

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 behavior of coupled core walls, coupling beams should be sufficiently strong and have good deformability and 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, frequent buckling of steel plate was observed and the influence of local buckling on the behavior 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] also experimentally studied deep coupling beams retrofitted by LRSP with stiffened steel plate. And by utilizing the additional stiffeners, plate buckling could be prevented which ensured that the steel plate had a wider yield area and hence higher energy dissipation.

Cheng and Yang [8] further developed a nonlinear finite element model based on the OpenSees software to accurately predict the seismic behavior of coupled shear walls with or without laterally restrained steel plate coupling beams. A nonlinear static analysis (pushover analysis) was conducted based on the combined model to investigate the seismic performances of the retrofitted coupled shear walls structure. Their studies revealed that retrofitted with LRSP coupling beams, seismic performances of coupled shear walls can be significantly enhanced. And the desirable seismic performance could be achieved by retrofitting the coupling beams in the middle and upper floors of the coupled shear walls. However, the seismic behaviors of coupled shear walls with LRSP retrofitted coupling beams under different earthquake actions have not to be investigated in their studies.

In this paper, based on the previous experimental results, a nonlinear time history analysis for a 12-story coupled shear wall by using OpenSees software was conducted to investigate the seismic performance of coupled shear walls retrofitted with LRSP coupling beams under different earthquake actions. By comparing the internal force, deformation and energy dissipation of coupled shear walls, it can be found that this retrofitting method could greatly improve the seismic performances of coupled shear walls structures under moderate and rare degree earthquakes actions.

2. PREVIOUS EXPERIMENTAL STUDIES

2.1. Experimental procedure

Six specimens with the same dimensions and reinforcement details (see Figure 1), but different retrofitting schemes, were fabricated and tested. The retrofitting schemes of all the specimens are shown in Table 1. 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 [5]. While Specimens DCB5 and DCB6 [6] were retrofitted with 3 mm and 4.5 mm steel plates, respectively, with a buckling control device was mounted onto the beam span. The buckling control device is composed of two steel angles (L70 ×70×5mm) along the top and bottom edges of the steel plate, then 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. For Specimens DCB10 and DCB11 [7], stiffeners were added symmetrically along the span of coupling beams. 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.

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

The load frame shown in Figure 2, designed by Kwan and Zhao [9] 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.

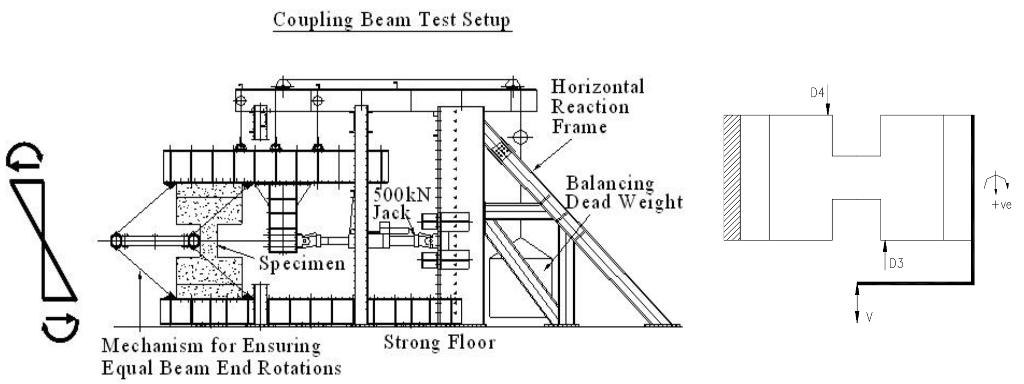


Figure 2. Test setup

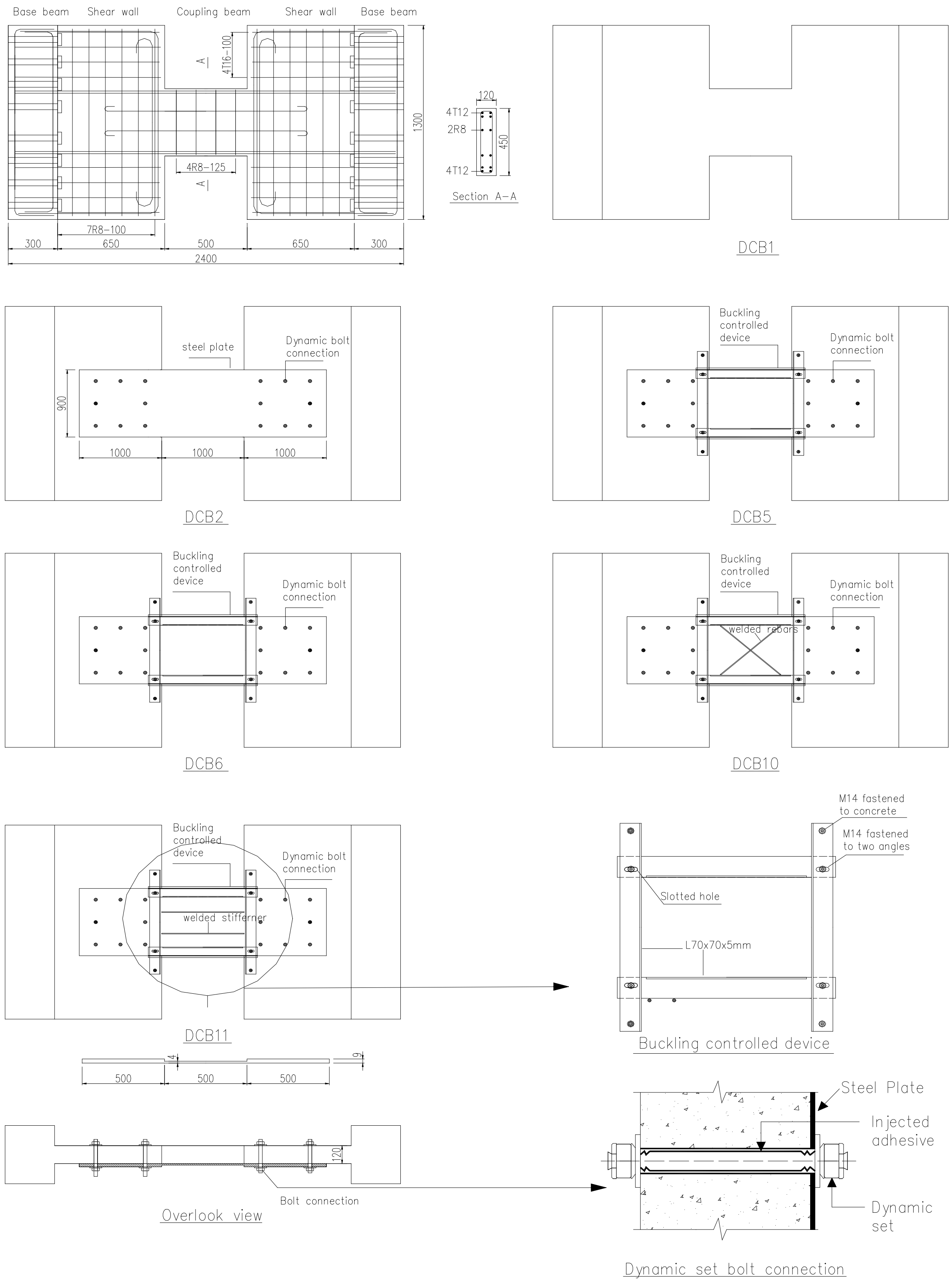


Figure 1. Details of specimens

2.2. Experimental results and discussion

2.2.1. Strength, deformation and ductility

Several parameters were defined to interpret the results of the tests. The ultimate rotation θ_u is defined by Park [10] 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.

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

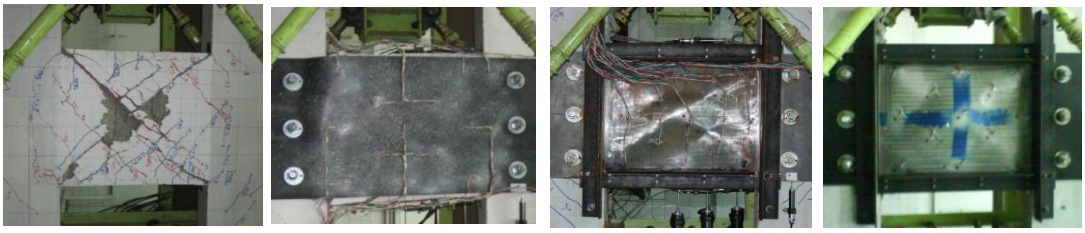
2.2.2. LRSP retrofitting method without stiffeners

2.2.2.1. 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 Figure 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 [5] , 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 [6] , 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

2.2.2.2. 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.

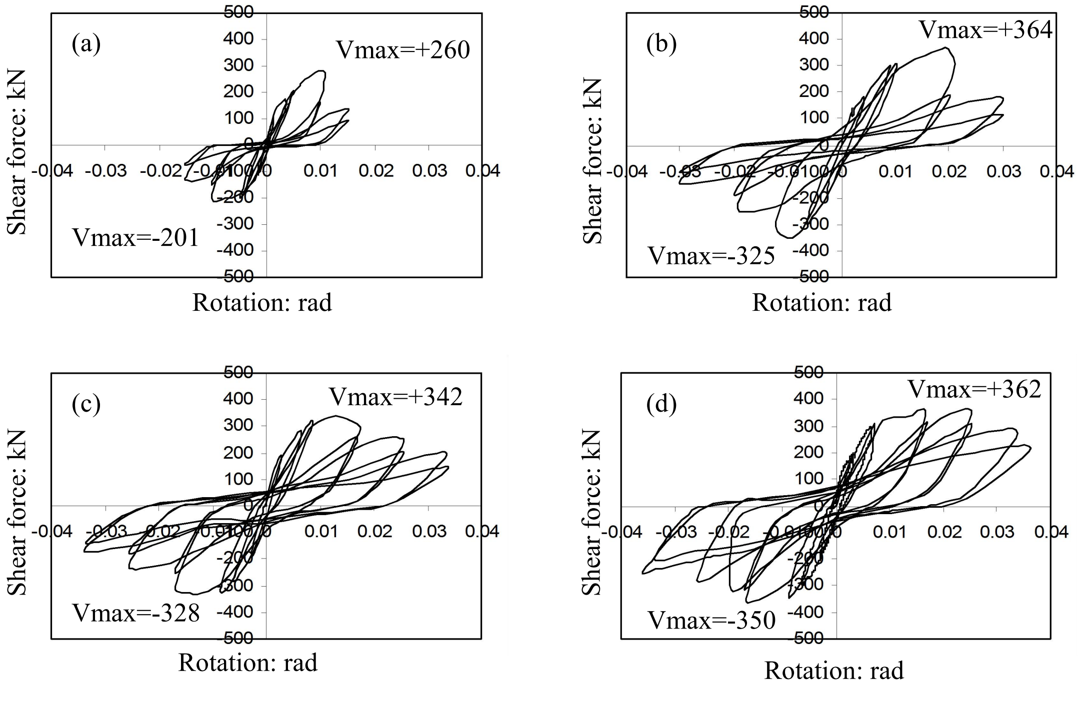


Figure 4. Load-Rotation curves: (a) DCB1; (b) DCB2; (c) DCB5 (d) DCB6

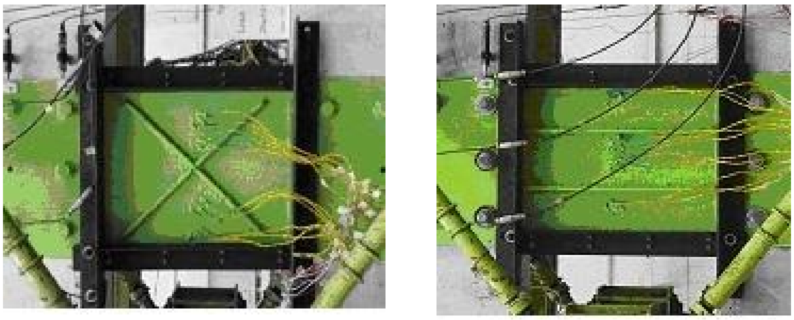
2.2.3. LRSP retrofitting method with stiffeners

2.2.3.1. Behaviors of steel plate in LRSP coupling beams with stiffeners

Different type of stiffeners are added in DCB10 and DCB11 [7] , 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.

2.2.3.2. Load-rotation curves of LRSP coupling beams with stiffeners

Figure 6 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.



DCB10 DCB11

Figure 5. Steel plate with stiffeners

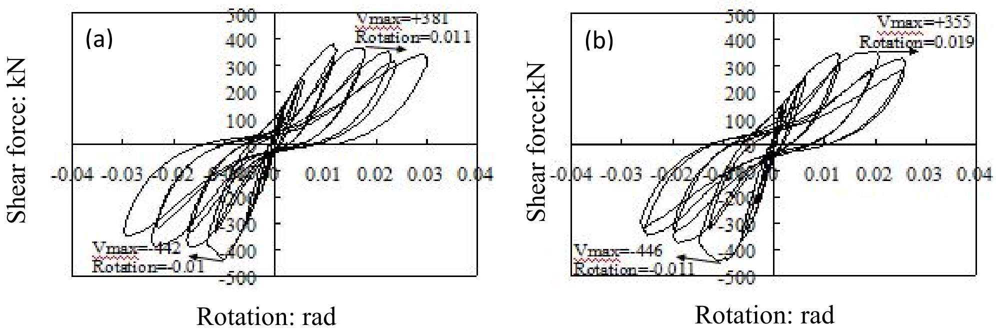


Figure 6. Load-rotation curves: (a) DCB10; (b) DCB11

3. NONLINEAR FINITE ELEMENT ANALYSIS

From previous experimental studies, it can be found that for existing deep concrete coupling beams, LRSP retrofitting method with diagonal stiffened steel plate can achieve better deformability and energy dissipation ability. To investigate the influence of retrofitted coupling beams on the seismic performance of coupled shear walls, a 12-story coupled shear wall structure is used to conduct the nonlinear time history analysis in an 8-degree seismic fortification intensity zone, where the site classification for construction is III according to the Chinese Code for Seismic Design of Buildings (GB 50011-2010) [11]. The coupled shear wall model is established by adopting the refined model of shear wall and the combined model of coupled shear wall proposed by Cheng and Yang [8] as shown in Figure 7, which can accurately simulate the load-deflection curves of coupled shear wall systems. The two retrofitted coupled shear walls keep the same dimensions as the previous experimental specimens DCB1 and DCB10. RCSW and CSW denote retrofitted coupled shear walls and coupled shear walls, respectively. The Earthquake waves for the nonlinear time history analysis are obtained from the PEER earthquake wave database [12]. To meet the requirements of earthquake response spectrum in the design code [11], 5 natural earthquake waves and 2 artificial earthquake waves are horizontally inputted in X-direction. Figure 8 indicates that the average response spectrum of the 7 earthquake waves agrees well with the standard response spectrum under rare earthquake with intensity of 8 degree. Based on this, different earthquake actions are obtained by adjusting the seismic peak acceleration a_{max} of the 7 earthquake waves.

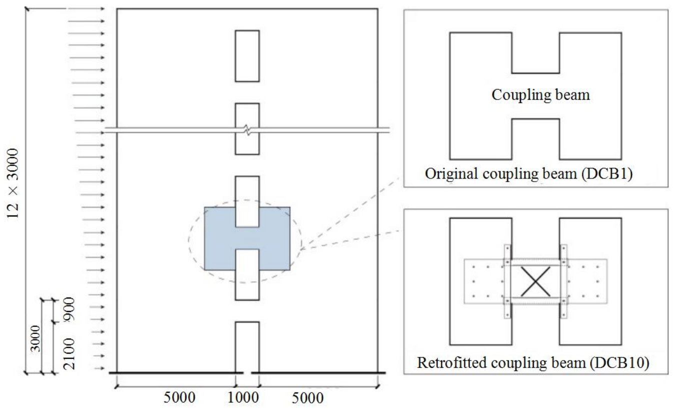


Figure 7. Coupled shear walls structure

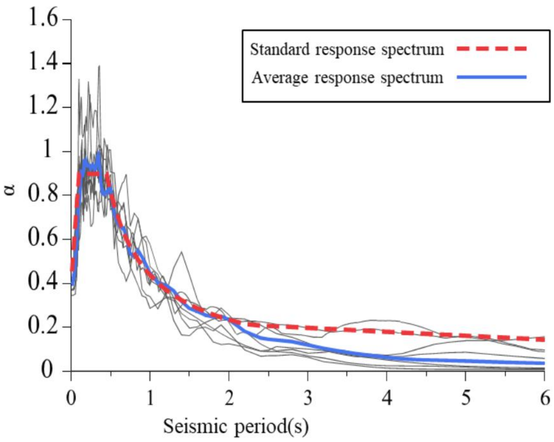


Figure 8. Seismic effect

3.1. *Yielding and failure mechanism of coupling beams*

Any coupling beam in the 12-story coupled shear wall structure is considered to yield when the displacement of beam exceeds yield displacement tested in previous experimental studies. Similarly, it is also considered to be damaged when the displacement of the coupling beams exceeds the displacement at their maximum loads. Table 3 shows the numbers of yielding and failure of coupling beams in RCSW and CSW under frequent, moderate and rare earthquakes, which are the average results of 7 different seismic waves.

It can be observed that the numbers of yielding coupling beams in CSW and RCSW are equal under different earthquake actions, mainly due to the proposed retrofitting method does not change the initial stiffness yield strength of the original coupling beams [5]. Therefore, the yielding mechanism of coupled shear walls with coupling beams LRSP retrofitted will not change. Moreover, coupling beams in CSW and RCSW are all yielded under moderate earthquakes and rare earthquakes.

Figure 9 shows the mean value of beam-end shear force in coupling beams of RCSW and CSW under 7 seismic waves in the cases of frequent, moderate and rare earthquakes.

It can be seen that in the case of frequent earthquakes, the maximum shear force of coupling beams in CSW and RCSW are 165 kN and 174 kN, which are almost the same. The shear forces of coupling beams in CSW and RCSW does not reach their ultimate strength capacity and all the coupling beams are not damaged.

In the case of moderate earthquakes, the maximum shear force of RCSW (359 kN) is about 47% higher than that in CSW (243 kN). Most coupling beams in CSW had reached their ultimate strength capacity while the coupling beams in RCSW have not yet reached their ultimate strength capacity.

In the case of rare earthquakes, the maximum shear force in RCSW (411kN) is about 67% higher than that in CSW (245 kN). all the coupling beams in CSW are destroyed, while only 6 coupling beams in RCSW in 3~8 stories are destroyed. Overall, the LRSP retrofitting method can effectively improve the strength capacity of coupling beams, and avoid serious failure of coupling beams and then largely improve the seismic performance of the coupled shear wall structures under moderate and rare earthquakes actions.

Table 3. Statistics of yielding and failure of coupling beams (mean value)

Earthquake actions	Quantities of yielding coupling beams		Quantities of failed coupling beams	
	CSW	RCSW	CSW	RCSW
Frequent earthquake	7	7	0	0
Moderate earthquake	12	12	10	0
Rare earthquake	12	12	12	6

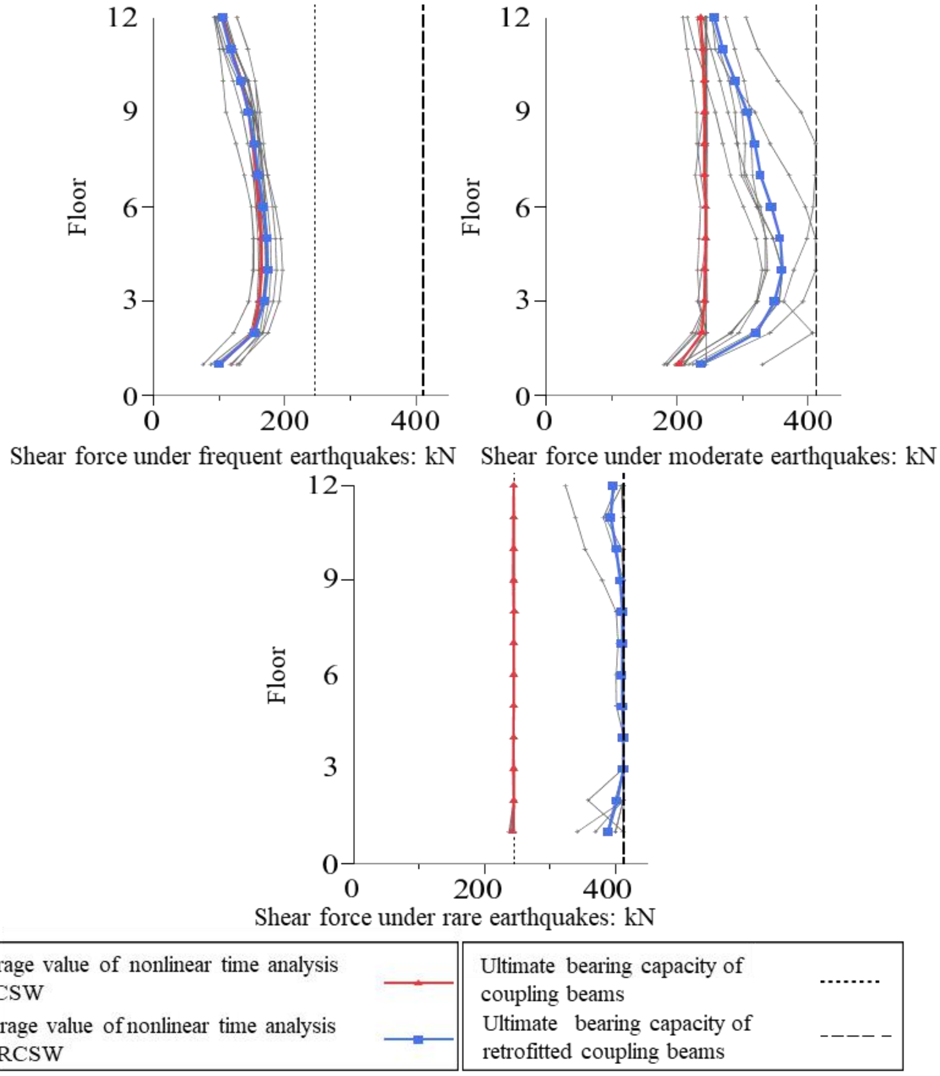


Figure 9. Beam-end shear force

3.2. Deformation of coupling beams

The inter-story drift ratio (IDR) has been widely used in earthquake engineering design and various design codes as an important index to evaluate the damage of structures. According to the design code [11], the maximum inter-story drift ratio (MIDR) of coupled shear walls should not exceed 1%. The MIDR of RCSW and CSW under rare earthquakes is shown in Figure 10. It can be observed that the IDR of CSW under each seismic wave exceeds the allowable value 1/100 of elastic-plastic inter-story drift ratio. The MIDR of CSW under the action of and artificial wave G reaches more than 1/66, which is far exceeding the allowable value 1/100 of elastic-plastic inter-story drift ratio. However, the IDR of RCSW is obviously smaller than that of CSW, and the MIDR of RCSW does not exceed the allowable value 1/100 of elastic-plastic inter-story drift ratio. Therefore, the LRSP retrofitting method can effectively control the IDR of the structures under rare earthquakes.

Table 4 shows the roof displacement ratio, maximum inter-story drift ratio and maximum rotation angle of RCSW and CSW under different earthquake actions.

It can be observed that in the case of frequent earthquakes, the displacement indexes of RCSW and CSW are almost same, and the LRSP retrofitting method makes little effects on improving the seismic performance of the coupled shear wall structure.

In the case of moderate earthquakes, the roof displacement ratio of RCSW (0.29%) is about 17% smaller than that of CSW (0.35%), the MIDR of RCSW (0.35%) is about 20% smaller than that of CSW (0.44%), and the maximum rotation of coupling beams in RCSW (0.51%) is about 52% smaller than that in CSW (1.06%), respectively.

In the case of rare earthquakes, the effect of the constraints of retrofitted coupling beams on displacement is more remarkable. The roof drift ratio, MIDR and the maximum rotation of coupling beams of RCSW are 33%, 38% and 44% smaller than that of CSW, respectively.

Table 4. Deformation comparison of CSW and RCSW (average value)

Earthquake actions	Roof displacement ratio (%)		Maximum inter-story drift ratio (%)		Maximum rotation of coupling beams (%)	
	CSW	RCSW	CSW	RCSW	CSW	RCSW
Frequent earthquake	0.09	0.09	0.12	0.12	0.15	0.14
Moderate earthquake	0.35	0.29	0.44	0.35	1.06	0.51
Rare earthquake	0.93	0.62	1.33	0.82	3.97	2.24

Due to higher ultimate bearing capacity and better ductility of the retrofitted coupling beams in RCSW, the deformation of the coupled shear wall structures can be limited to the requirements of the design code [11], and the serious damage or collapse of the coupled shear wall structures can be avoided.

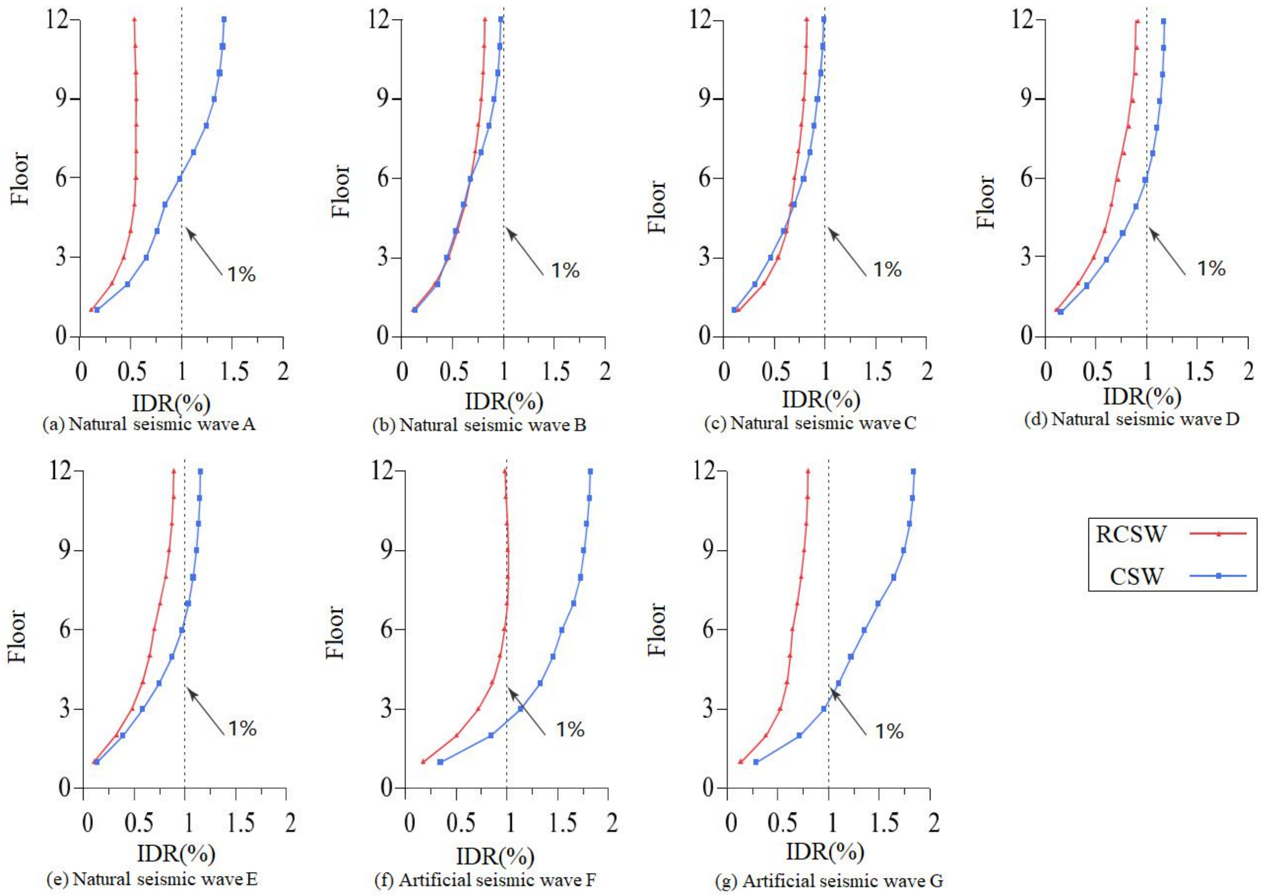


Figure 10. Maximum inter-story drift ratio (MIDR) under rare seismic action

3.3. Energy dissipation of coupling beams

Figure 11 compared the cumulative energy dissipation of coupling beams in RCSW and CSW under rare earthquakes to investigate the influence of LRSP method on the energy dissipation ability of coupled shear walls.

It can be observed that in the case of rare earthquakes, the hysteretic energy dissipation of coupling beams at the middle and upper floors in RCSW and CSW are larger than that at lower floors, as well, the cumulative energy dissipation of coupling beams at each floor in RCSW is significantly greater than that of CSW, resulting in the total hysteretic energy dissipation of coupling beams in RCSW is about 63% higher than that of CSW. The results show the LRSP retrofitting method can effectively improve the energy dissipation of coupling beams, so as to improve the seismic performance of the coupled shear wall structures.

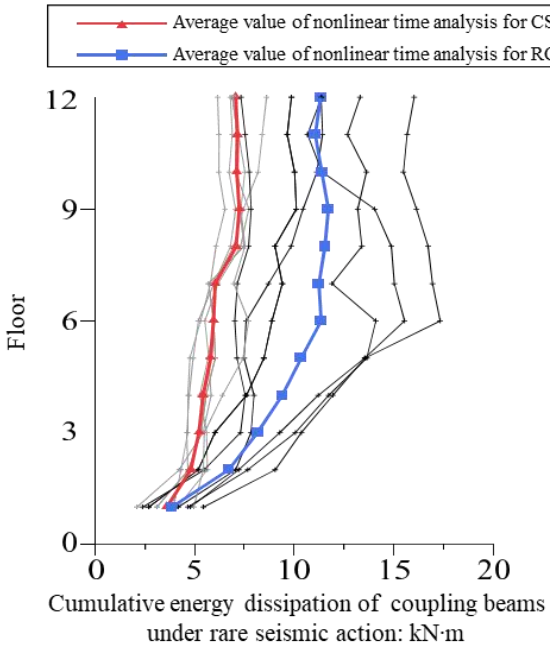


Figure 11. Cumulative energy dissipation of coupling beams under rare seismic action

4. CONCLUSIONS

Nonlinear time analysis is conducted by the finite element model of 12-story coupled shear wall. Based on the nonlinear analysis of coupled shear walls with LRSP retrofitting method (RCSW) or without LRSP retrofitting method (CSW), combined with the previous experimental results, the following conclusions can be drawn:

1. From the results of nonlinear time history analysis, the LRSP retrofitting method has a significant effect on the shear strength capacity and deformability of coupling beams. The maximum shear forces of coupling beams in RCSW are about 47% and 67% higher than that in CSW under the actions of moderate earthquakes and rare earthquakes, respectively. As well, the maximum rotation of coupling beams in RCSW are about 52% and 44% smaller than that in CSW under the actions of moderate earthquakes and rare earthquakes, respectively.
2. The deformability of coupled shear walls are considerably enhanced after retrofitting the coupling beams with LRSP method. The roof drift ratio of RCSW can be effectively reduced, and the MIDR of RCSW is effectively limited to the allowable value 1/100 of code standard, which is about 20% and 38% smaller than that of CSW under the actions of moderate earthquakes and rare earthquakes, respectively.
3. The energy dissipation of the deep coupling beams retrofitted with LRSPs could be significantly improved. In the case of rare earthquakes, the total hysteretic energy dissipation of coupling beams can be increased by about 63% after LRSP retrofitting. Moreover, the damage of the coupling beams under moderate and rare earthquakes can be remarkably alleviated due to its improved energy dissipation and ductility.

4. In the case of frequent earthquakes, the shear strength capacity of coupling beams and structural deformability of RCSW and CSW behave almost the same, and the improved seismic performance of coupled shear walls after retrofitting the coupling beams with LRSP method is limited. Thus, it is suggested that the LRSP method for existing deep RC coupling beams could be applied on buildings in moderate and rare seismicity regions.

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