SEISMIC BEHAVIOR OF STEEL PLATE REINFORCED CONCRETE SHEAR WALLS

C.J. Gan[1], X.L. Lu[2] and W. Wang[3]

[1] *Student, Research Institute of Structural Engineering and Disaster Reduction, Tongji Univ., Shanghai. China*
[2] *Professor, Research Institute of Structural Engineering and Disaster Reduction, Tongji Univ., Shanghai. China*
[3] *Lecture, School of Civil Engineering, Xi'an Univ. of Architecture and Technology, Xi'an, China, Email: chj.gan@gmail.com, lxlst@mail.tongji.edu.cn, wangwgh@xauat.edu.cn*

ABSTRACT :

The steel plate reinforced concrete shear wall (abbreviated as SPRCW) is an innovative type of lateral force resisting structural member potentially used in steel-concrete composite structures in high-rise buildings, especially in earthquake-prone regions. This kind of walls is formed by inserting both steel plate and steel sections (eg. steel channels) into the ordinary RC shear walls. This paper presents an experimental study on the seismic behavior of such structural members. The test was conducted through the cyclic loading scheme on a series of models at 1:2 scale. The primary parameters vary in thickness of the steel plate, aspect ratio of the wall, and thickness of the SPRCW. In order to determine the influence and relative importance of different detailing on integrity between the steel plate and concrete component under service load, such details as lateral ties and welding studs on the steel plate are designed as well. The experimental program is developed to evaluate the damage patterns, hysteretic response, strength, stiffness and ductility behavior of the walls under constant axial compressive loads and cyclic horizontal forces. In addition, the experimental results of SPRCWs are compared with those of other three ordinary RC shear walls. Through this investigation, it is observed that steel plate and steel sections incased in the wall plays a major role in carrying loads, while concrete provides lateral restraint for steel plate, and therefore, prevent the steel plate from premature failure in stability and increases deformation capacity of the wall.

KEYWORDS: Steel Plate, Concrete Shear Wall, Experimental Study, Seismic Behavior

1. INTRODUCTION

Over the past decade, a substantial amount of research has been conducted worldwide on composite and hybrid wall systems. With development in both structural and architectural demands for tall buildings, the following factors in lateral resistance structures play a more important role than in multi-storied buildings, including high stiffness to help reduce drift, good damping characteristics, and ability to prevent from brittle damage pattern. Consequently, solid structural steel plate has been introduced into composite wall systems as the main steel sections, based on the verified high capacity in strength, stiffness and ductility for steel plate shear walls (A. Astaneh-Asl and Zhao, 2000-2001).

Composite steel plate shear walls were firstly tested and put into practice in a frame-shear wall dual system of the bus station complex in Nagoya, Japan as early as 1960's (Design Corporation of Japanese Construction, 1964). In the United States, Degenbolb Engineers used steel shear walls covered on both sides with reinforced concrete in a hospital in San Francisco (Dean et al., 1977), followed by similar systems denoted as "Composite Steel Plate Shear Walls" in the AISC Seismic Provisions (AISC, 1997). During 1998-2001 periods, the composite shear wall which consisted of a steel plate shear wall with reinforced concrete walls attached to one side or both sides of the steel plate using mechanical connectors such as shear studs or bolts were discussed by cyclic testing at UC Berkeley (A. Astaneh-Asl and Zhao, 1998-2001). In China, a good example of application is the project of Beijing International Financial Center construction in process.

In summary, previous research achievement validates that reinforced concrete shear walls can perform better in high-rise structures as predicted when incorporated with steel plate than ordinary RC shear walls. These include larger lateral deformation capacity with the same shear capacity, smaller thickness and thereof less weight that are very advantageous from architectural point of view. However, these past experimental studies have typically used steel-frame members infilled with composite shear walls, and economy is not a primary concern compared with RC structures. And little effort has been devoted to the parametric analysis, that is, impacts for the wall systems under different combinations of geometric and material parameters.

In the paper herein, a composite steel plate concrete shear wall, namely "Steel Plate Reinforce Concrete Shear Wall" (termed as SPRCW), is proposed based on prototype of the steel reinforced concrete wall specimen (Xilin Lu et al., 2006), mainly substituting steel wide-flange sections with steel plates. By carrying out a series of cyclic load tests on 16 1:2 scale specimens, the paper reports many new aspects of the seismic behavior of the SPRCW derived from variation in thickness of the steel plate and the wall, the aspect ratio (ratio of height to width), concrete strength, steel percentage and other detailing, etc.. The incased steel plate is shown to be an effective way to improve the strength and deformation capacity of the shear wall by comparing findings and results between the composite structure and the traditional RC wall with equivalent configurations. This research also qualitatively reveals the load path at different levels of loading and the ultimate damage pattern for different geometric types of wall.

2. EXPERIMENTAL PROGRAM

2.1. Specimen Design

Fig.1 shows cross-sectional configuration of the SPRCW. In the SPRCW model, the steel plate acted as a chief component for load-resistance both to gravity load and cyclic lateral force. At both ends of the steel plate, were welded the steel channels, functioning similarly to the boundary elements in RC walls. The other reinforcement such as longitudinal and transverse bars were designed as 6mm in diameter at 150mm spacing. Reinforcement details are illustrated in Fig.2.

Table 1 shows the properties of test specimens, together with another three parallel specimens of traditional RC walls equivalent in dimensions. The steel plates were Q235Grade steel with nominal yield strength of $235\,N/mm^2$. The steel channels used in the specimens were hot-rolled made and welded to the steel plates by the fabrication company. Cold formed plain bars of 6mm diameter were applied for both of the vertical reinforcement and transverse ones. Rebars of 20mm diameter were mainly used for the reinforcement of the loading beam and the footing. Concrete was poured in-situ after formworks had been completed.

In order to avoid the premature deviation of concrete cover from the bare steel plate, the composite interaction is achieved through the lateral ties cross the prefabricated holes on the steel plate surfaces so as to constrain the expansion of reinforcement web and thereby the concrete. On the other hand, headed studs of 6mm diameter were designed to weld-anchored to the bare steel plate at tips of the headed stud in the process of fabrication. Fig.3 depicts the detailing that studs of 6mm diameter stagger at intervals of 300mm.

2.2. Test Setup And Test Regime

All of the 19 specimens were tested under constant axial compressive loads and cyclic horizontal forces. Fig.4 shows the test setup. On one end of the specimen, the MTS hydraulic actuator was connected between the reaction wall and the loading beam attached to the top of the panel, which ensured loading on the concrete, the steel sections and the steel plate simultaneously and uniformly. The lateral load was transferred from the top square-sectioned reinforced concrete beam (400×400 mm) to both concrete component of the SPRCW and the internal steel plate embedded in the footing, through four cold formed rebars of 25 mm diameter fixed in the loading beam by anchoring. Four ball-bearing hydraulic jacks were placed on the top surface of the loading beam, applying constant vertical forces to the wall. The RC bottom foundation beams of the specimen were tightened to the strong floor of the laboratory through 8 post-tensioned anchor rods, to establish a well-defined boundary condition at the foundation.

Throughout the experiment, both of the global deformations at different levels of height of the test specimen and the local strains at the locations of steel sections and rebars were documented. The shear force exerted to the specimen was measured by the load cell in the hydraulic actuator. The horizontal displacement applied on the top of the shear wall was also recorded by the actuator, which could be plotted by the computer for monitoring the test procedure. For higher precision in displacement values, three LVDTs were horizontally placed on the model at the level of mid-height, top and bottom of the specimen (shown in Fig.4).

The vertical loads remained constant and stable during the process of testing, the value of which were figured out by the axial compressive ratio, the concrete strength for design and the cross-sectional area. The force and displacement-controlled loading history was modified from the protocol for testing of specimens through consideration of the deformation characteristics of the SPRCW.

3. EXPERIMENTAL RESULTS

Extensive data were collected from the acquisition system recorders during these tests. One of the most important results are relationship curves for shear force versus lateral displacement. Quite an amount of valuable information for evaluating seismic behavior of the specimens can be given from these plots, such as strength, stiffness, ductility, and energy dissipation capacity of the specimen, and all key characteristics very important in design and analysis of structures. Also, skeleton curves shown in Fig.5, which are useful to define initial stiffness, stiffness degradation, etc., can be obtained by means of enveloping these force-displacement curves. Gan (2008) documented the important results for examining capacity of bearing load, deformation, and ductility behavior of all the tested specimens in further detail. Ductility index was termed as $\mu = \Delta_u / \Delta_y$. Energy dissipation capacity is herein calculated by the concept of equivalent viscous damping (Jacobson, 1930). The value of equivalent viscous damping ratio h_e , can be calculated through area enclosed by the hysteretic hoop.

3.1. Seismic Behavior Evaluation of the SPRCWs

3.1.1 Affects due to the aspect ratio

All of the 16 SPRCW specimens can be divided by the aspect ratio into two groups: SPRCW1-SPRCW8 with the value of 2 and SPRCW9- SPRCW16 with the value of 1.5. Distinct difference was observed throughout the test process: The taller walls were susceptible to bending mode damage, and developed more or less yielding in the steel members when reaching the maximum shear capacity. However, the shorter specimens did not show any such damage at the same level of displacement. During the next cycles, concrete damage deteriorated and finally failed as the result of severe crushing at bottom concrete corners and subsequent local buckling in the outmost rebars and the steel plate. The taller specimens lost stability globally. However, for the shorter specimens, the damage to the concrete panel of the shear wall was localized at the connection to the footing, almost dominated by the horizontal shear cracking and ultimate crushing, despite of yielding at the reinforcement grid. They usually underwent quite a few stages of displacement reversals in the post-peak strength period, (see Fig.6). Compared the main performance characteristics, it is also found that specimens with the aspect ratio of 1.5 have larger capacity of bearing load at the stages of initial cracking, yield point and maximum strength than those of 2.0 value ones, The values averagely go up by 52.0%, 27.1% and 23.8% respectively for those three stages above with decrease in the aspect ratio. Adversely, specimens with the aspect ratio of 2.0 show greater capacity of energy dissipation at the stages of maximum force and ultimate state point than those of 1.5 ones in light of the h_e values. Better energy dissipation is achieved largely through more cracking in the concrete panel. The value of Δ_u at the top of the wall grows by the aspect ratio, with average increase of 18.4% from 1.5 to 2.0 of height-to-width ratio. But the inter-storey drift capacity (that is, the top lateral displacement divided by the entire height of the panel) is not sensitive to this parameter.

3.1.2 Affects due to the thickness of the wall

The following pairs of test specimens have no difference but the thickness of the wall: SPRCW2 & SPRCW6, SPRCW4 & SPRCW8, SPRCW10 & SPRCW14, and SPRCW12 & SPRCW16. Through analyzing the characteristics concerning seismic behavior, several observations can be made from the data: firstly, such strength indexes as F_c , F_y and $F_{\max}$ are increased by enlarging the thickness of the wall from 125mm to 200 mm in its degree. In particular, the peak strength generally improves by 20.9%. This ascribes to the fact that the concrete participation lends great assistance to prevent the relatively thin steel plate from premature buckling and thereof exerts the bearing capacity of the steel plate to the fullest extent; secondly, the thickness of the covered concrete makes the ultimate drift capacity increased by an average of 26.3% as the thickness rises from 125mm to 200mm; thirdly, when increasing the thickness of concrete cover, the capacity of energy dissipation is upgraded at the ultimate failure time. This also demonstrates the rule that concrete plays an important role in avoiding instability in the steel plate; fourthly, it should be noted that the ductility index, μ , is dramatically enhanced by an average of 51.1% when the thickness of the wall increases from 125mm to 200mm.

3.1.3 Affects due to the thickness of the steel plate

Two groups of comparative test specimens, SPRCW1 & SPRCW2 and SPRCW9 & SPRCW10, are different in the thickness of the steel plate with 4mm and 6mm. With respect to failure process, it appears that the shear walls embedded with thicker steel plates such as SPRCW2 and SPRCW10 exhibited more ductile from the peak strength point to attainment of 85% $F_{\max}$ after that, and damage to steel sections were relatively limited as well, while significant buckling occurred in the steel bars and steel plates at the lowest quarter of the walls, together with disruption at concrete corners in specimen SPRCW1 and SPRCW9. Thinner steel plates caused quicker decline in loading capacity at the post-peak strength cycles. Contrast between SPRCW9 and SPRCW10 is shown in Fig.7. It should be noted that the steel plate thickness makes great sense to add to deformation capacity of the walls, such as Δ_c , Δ_y and Δ_u , especially for Δ_u raised by 56.5%. For SPRCW1 & SPRCW2 and SPRCW9 & SPRCW10, the percentage of h_e at the ultimate displacement point is raised from 12.58 to 18.0, from 9.42 to 11.98, respectively. And the ductility index, , is slightly increased from 2.29 to 2.62, from 2.45 to 2.81, respectively as the thickness of the steel plate increases from 4mm to 6mm.

3.1.4 Detailing influence

Two types of detailing, shear studs and lateral ties have been used in the test specimens to reinforcing integrity between the concrete and steel components. Some key results are discussed as follows:

(1) It can be seen that, such specimens with welded studs as SPRCW4, SPRCW5, SPRCW8, SPRCW12, SPRCW13 and SPRCW16 developed steadily in crack extension and less in crack amount in contrast to the other ones. This is understandable since the studs have an active effect in mitigating cracking and interface slip, and thus combine two components together well. Accordingly, this detailing is appropriate to improve the retention of post-peak strength as shown during the test process.
(2) When welded studs on the steel plate, specimens reflect advantages in seismic behavior such as the maximum shear strength, the ultimate lateral displacement capacity, and ductility and especially for energy dissipation capacity, by comparing results from SPRCW3, SPRCW4 and SPRCW5, SPRCW11, SPRCW12 and SPRCW13. Studs on the steel plate improve the maximum displacement with 55.5% greater than those bare steel plate incased specimens and 10.6% more than the specimens with lateral ties. They also result in 14.8% increase in the value of μ .These impacts can be traced to the fact that the studs not only work as shear connectors, but also resist to tensile action at locations subjected to buckling.
(3) Lateral ties contribute to lessening crack development barely at the early cycles of loading, but provide less improvement in strength, stiffness and ductility than expected. It is thus concluded that, shear studs turn to be a better device for strengthening capacity against seismic action.

3.2. Comparison between SPRCWs and Traditional RC Walls

3.2.1 Failure pattern

Through observations, the steel plate was shown to be an effective way to suppressing the brittle fracture modes of traditional RC walls. As an example of SPRCW2 versus RCW2, SPRCW2 basically stayed ductile until the corner concrete crushed and the outmost rebars buckled and some even ruptured, which were induced by serious deterioration in local instability of the steel members at bottom regions. Extensive development of concrete diagonal cracking up to the upper half height of the panel was also observed before global collapse of the wall. However, brittle failure such as concrete cracking and crushing was the dominant damage pattern for RCW2. And concrete cracks were accumulated near the base of the wall thereby degrading the concrete in sliding, compressive and tensile straining (see Fig.8). In addition, the steel channels at both ends acted as boundary elements to raise the strength capacity and to alleviate damage to concrete corners at the same time. Thus, concrete shear walls without reinforced steel members were more susceptible to brittle shear failure, instead of ductile flexible failure.

3.2.2 Strength, stiffness and ductility

Shear force-lateral displacement curves in Fig.9 address that: SPRCWs have larger capacity of loading at the points of initial cracking, first yielding and approach to the maximum, in particular increase in the peak strength by 87.6%, 112.3% and 62.0% respectively for three contrasts; The values of displacement nearly double by increasing 156.8%、93.3% and 121.9% for Δ_c, 32.3%、96.4% and 77.8% for Δ_y, and 99.0%、105.1% and 108.8% for Δ_u, respectively for those three contrasts. Besides, the ductility indexes increase by percentages of 50.4, 4.4 and 17.4 respectively. Thus, it can be seen that the steel plate and steel channels behave as the major components responsible for bearing loads and resisting deformation.

3.2.3 Energy dissipation capacity

Fig.9 mentioned above shows that, the figures of SPRCWs are shaped plumper like bows, while the hysteretic hoops of specimen RCWs are more pinched like cambiform. According to the rule that the more enclosed areas by hysteretic hoops the more capacity for energy dissipation, it is verified that SPRCWs are able to dissipate much more earthquake energy than the ordinary RC walls. As concerning the values of h_e, the three pairs of contrast yield increase by 15.4%, 9.2% and 14.9% respectively when reaching the maximum force, by 15.5%, 58.5% and 70.0% at ultimate damage.

4. CONCLUSIONS

The seismic behavior of the steel plate reinforced concrete shear wall is investigated by a series of 1:2 scale specimens loaded by both constant axial compression and cyclic lateral force. Through the experiment, mechanisms how this innovative type of steel-concrete composite shear wall resists gravity load and seismic effect can be observed; systematic comparison and parametric analysis are done in terms of the characteristics and figures from the test. In addition, this paper is intended to provide some preliminary recommendations for the structural engineers, architects, researchers and others involved in study and application on this new structural member. The following conclusions can be derived from this study:

(1) Through comparing seismic behavior of SPRCWs with traditional RC walls, it shows that this new member has the potential to offer the same strength appropriate for resisting the forces from earthquake with larger shear stiffness so as to have smaller thickness and less weight which seem architecturally-acceptable for those shear walls used at the bottom of high-rise buildings.
(2) The steel plates used in SPRCWs are relatively thin in general. Plates thinner than 8mm in practical structures are not recommended since such thin plates cannot guarantee the wall stable and are easier to buckle if no suitable bracing is provided. Consequently, such thin plates may require a large number of shear studs as the connectors or thicker concrete cover to postpone plate buckling.
(3) Concrete does act as an effective stiffener and prevent steel plate from buckling. It also helps to improve strength, stiffness and the retention of post-peak strength. However, from the viewpoint of flexibility for architectural function, higher concrete grade outweighs thicker concrete covers in providing adequate performance.
(4) Shear connectors used to connect steel plates to concrete are appropriate to better the capacity of deformation, ductility and energy dissipation. Welded shear studs appear to be more efficient detailing than lateral ties.
(5) As for the relative higher shear capacity of the panel of the composite wall, any connections of the shear wall to other boundary members such as beams and foundations should have enough strength and stiffness to transfer a considerable amount of shear force.

5. TABLES

Table 1 Properties of test specimens

No.	Width× thickness (mm×mm)	The aspect ratio	Concrete grade	Thickness of the steel plate (mm)	Steel percentage (%)	Axial force ratio	Detailing between steel plate and concrete	Boundary element
SPRCW-1	1000×125	2.0	C30	4	4.23	0.5	none	steel channel [6.3
SPRCW-2	1000×125	2.0	C30	6	5.67	0.4	none	steel channel [6.3
SPRCW-3	1000×125	2.0	C50	4	4.23	0.3	lateral ties	steel channel [6.3
SPRCW-4	1000×125	2.0	C50	4	4.23	0.3	shear studs	steel channel [6.3
SPRCW-5	1000×125	2.0	C50	4	4.23	0.3	both [a]	steel channel [6.3
SPRCW-6	1000×200	2.0	C30	6	3.72	0.4	none	steel channel [8.0
SPRCW-7	1000×200	2.0	C30	4	2.82	0.4	lateral ties	steel channel [8.0
SPRCW-8	1000×200	2.0	C50	4	2.82	0.3	shear studs	steel channel [8.0
SPRCW-9	1000×125	1.5	C30	4	4.23	0.4	none	steel channel [6.3
SPRCW-10	1000×125	1.5	C30	6	5.67	0.4	none	steel channel [6.3
SPRCW-11	1000×125	1.5	C50	4	4.23	0.3	lateral ties	steel channel [6.3
SPRCW-12	1000×125	1.5	C50	4	4.51	0.3	shear studs	steel channel [8.0
SPRCW-13	1000×125	1.5	C50	4	4.23	0.3	both	steel channel [6.3
SPRCW-14	1000×200	1.5	C30	6	3.72	0.4	none	steel channel [8.0
SPRCW-15	1000×200	1.5	C30	4	2.82	0.4	lateral ties	steel channel [8.0
SPRCW-16	1000×200	1.5	C50	4	2.82	0.3	shear studs	steel channel [8.0
RCW-1	1000×125	2.0	C30	none	-	0.5	lateral ties	none
RCW-2	1000×125	2.0	C30	none	-	0.4	lateral ties	none
RCW-3	1000×200	2.0	C50	none	-	0.3	lateral ties	none

[a] Here "both" means both lateral ties and shear studs were used.

6. ILLUSTRATIONS, DIAGRAMS AND PHOTOGRAPHS

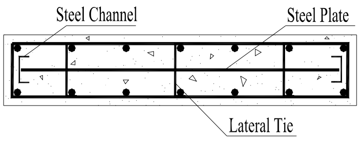
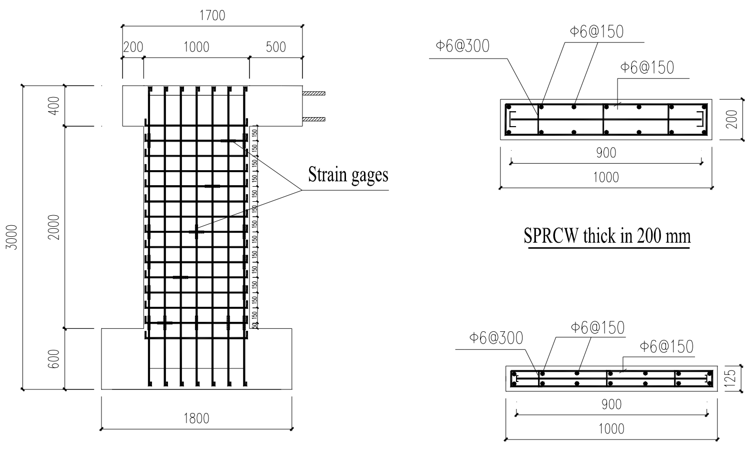


Figure 1 Schematic diagram for cross section



Elevation of SPRCW with aspect ratio of 2   SPRCW thick in 125 mm

Figure 2 Diagram for reinforcement design

(a) reinforcement grid and steel channel   (b) shear studs welded on the steel plate

Figure 3 A view of detailing

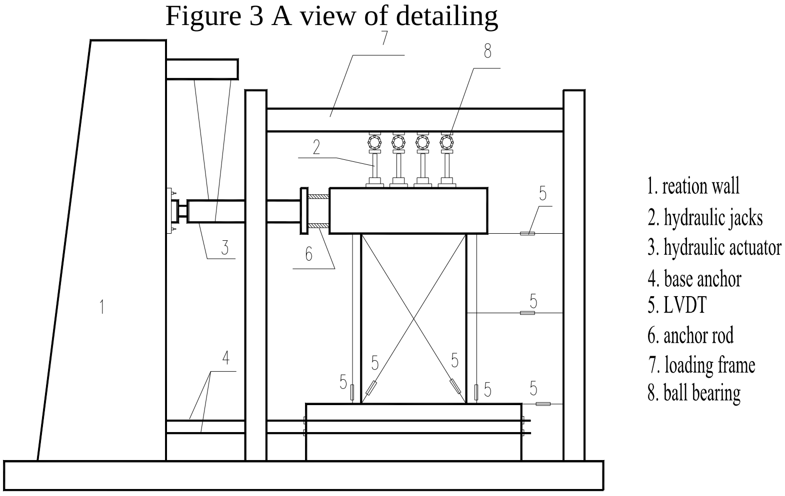
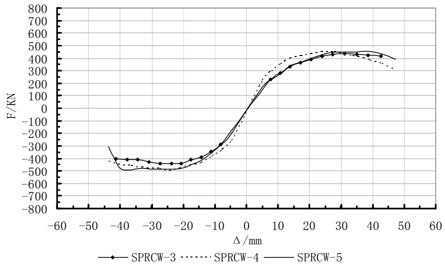
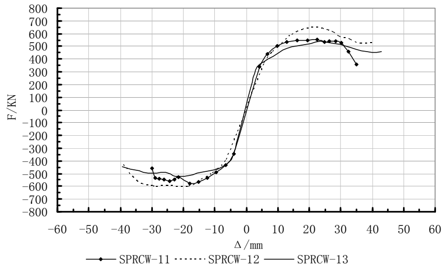


Figure 4 Test setup and instrumentation



(a) SPRCW-3, 4, 5



(b) SPRCW-11, 12, 13

Figure 5 Skeleton curves

(a) specimen with the aspect ratio of 2

(b) specimen with the aspect ratio of 1.5

Figure 6 Difference in failure pattern with variation in the aspect ratio

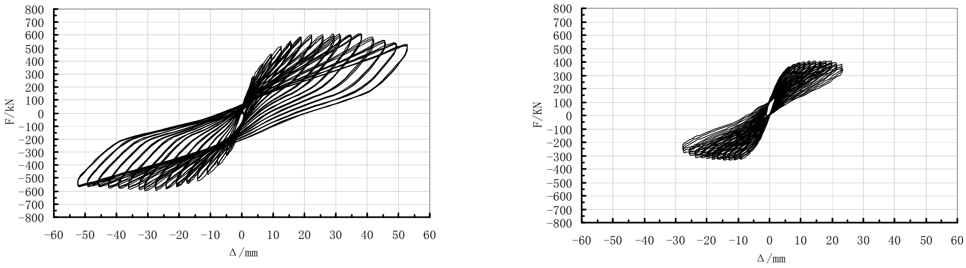
(a) thickness of the steel plate: 4mm

(b) thickness of the steel plate: 6mm

Figure 7 Difference in failure pattern with variation in the thickness of the steel plate

(a) RCW-2 (b) SPRCW-2

Figure 8 Comparing failure pattern between SPRCWs and RCWs



(a) SPRCW-8 (b) RCW-3

Figure 9 Relationship curves for shear force versus lateral displacement

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