Experimental Performance Evaluation of Multi-Storey Steel Plate Shear Walls Designed by Different Methods

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Experimental performance evaluation of multi-storey steel plate shear walls designed by different methods

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Abstract. In accordance with two different design methods including the technical specification for steel structures and the shear bearing capacity method for infilled steel wall plates, two types of steel plate shear wall with unstiffened panels have been designed and constructed. All shear wall specimens are exposed to ultimate static monotonic and low horizontal cyclic loading conditions in order to determine their structural behaviors under an idealized severe earthquake event. The seismic performances of these two types of specimens are identified by the overall roof displacement angle, lateral stiffness, ductility, different distribution of horizontal force and overturning moment, and inclined angle of diagonal tension field. These two types of steel plate shear wall exhibit excellent seismic performance. However the specimens with thin infill plate thickness of 1.1mm perform better than the thicker specimens with plate thickness of 3.75mm. In terms of serviceability performance, the experimental results exhibit that the thicker specimens designed by the technical specification tend to be more conservative. Their over-strength factor, strength assurance coefficient and drift angle are 4.98, 6.3 and 1/1335, respectively. However, the thiner specimens designed by the shear capacity method for shear panel yield the serviceability performance factors of 2.21, 2.71 and 1/407, respectively. It is important to note that design practice generally adopts the over-strength factor between 2 and 3. and The strength assurance coefficient is often designed for 3 and the maximum inter-story drift limit given by the design specification is 1/300. On this ground, it is apparent that shear bearing capacity method enables relatively more economical compared to the technical specification for steel structures.

Keywords: Steel plate shear wall, performance evaluation, ductility, over-strength factor, strength assurance coefficient

1. Introduction

A steel plate shear wall (SPSW) is an innovative lateral load-resisting system capable of effectively bracing a building against both wind and earthquake forces. The force-resisting unit is composed of an internal steel plate, a vertical edge member (column or vertical stiffener) and a horizontal edge member (beam or horizontal stiffener). A steel shear wall system is formed when the steel plate is continuously arranged from top to bottom along a certain span of the structure. In the past few decades, experimental and analytical studies into steel plate shear walls as the main lateral load resistant elements in buildings have been carried out. Driver et al. [1] conducted a cyclic loading test on a four-story, single-span common steel plate shear wall, and the results show that the steel plate shear wall has high energy dissipation capacity and good ductility. Chen and Jhang [2] conducted a seismic test study on the steel plate shear wall with low yield point. The test results showed that the energy dissipation capacity and ductility of the low-yield steel plate shear wall was relatively high, and then the simulation study was carried out. The test results were in a good agreement. Shishkin et al. [3] carried out a parametric study using the modified strip model to examine the effect of varying the angle of inclination of the tension strips on the predicted inelastic behavior of the model. They reported that the ultimate capacities of steel plate shear walls with a wide variety of configurations vary slightly with the variation of the inclination of the strips. Hitaka and Matsui [4] proposed a steel shear wall with slit and conducted a pseudo-static test for the first time. The results showed that reasonable slit does not affect the bearing capacity and stiffness of the steel plate shear wall. Zirakian and Zhang [5-6] assessed the structural behavior as well as plate-frame interaction characteristics of unstiffened low yield point steel plate shear wall systems using finite element and analytical approaches. Tsai et al. [7-9] conducted a cyclic loading test study on two single-span full-scale steel plate shear walls, and the proposed design method is in a good agreement with the experimental results.

A large number of research experiments have highlighted the advantages of using SPSW as a lateral force-resisting system in buildings including more stable hysteretic characteristics, higher plastic energy absorption capacity, and enhanced stiffness, strength, and ductility [10-13]. Steel plate shear walls are well-suited for either new constructions or as a technique for seismic upgrading of existing structures. It is anticipated that this system will be economical compared with reinforced concrete shear walls since a SPSW has many advantages such as the light weight, reduced foundation costs, good ductility, saving steel and rapid construction [14-18]. In this paper, two types of steel plate shear wall (SPSW)have been designed respectively based on the different design methods given by "Technical specification for steel structure of tall buildings" and by the shear bearing capacity method for infilled steel wall plates. All of the specimens are subjected to the static monotonic pushover and low horizontal cyclic loading conditions to investigate their structural behaviors. The load actions simulate a situation of severe earthquake event. The structural seismic performance of all specimens with two different design principles is then evaluated comprehensively by a variety of performance indices including overall roof displacement angle, lateral stiffness, ductility, redistribution of horizontal force and overturning moment, inclined angle of diagonal tension field, over-strength factor, and strength assurance coefficient of structure.

2. Experimental method

A prototype structure of the unstiffened three-span ten-story thick SPSW has been designed strictly in compliance with the corresponding specification in Chinese code for seismic design of buildings and the principle of yielding in shear prior to buckling specified in the appendix B of the "Technical specification for steel structure of tall buildings" for the infilled plate. Another three-span ten-story thin prototype of the moment-resisting steel frame-SPSW system has been designed using the shear bearing capacity-based seismic design method of infilled plates, which consider shear buckling prior to yielding. Two of scaled three-story single-span specimens using both thick and thin infilled plates are built as the shear wall structures. The similarity coefficient of 1/4 has been chosen based on comprehensive consideration for the geometric dimensions of prototype SPSW structure, loading equipment and fabrication feasibility of steel frame beams and columns.

The thick specimen designed by the Chinese codes is referred to as SPSW1 and the thin specimen designed by the shear bearing capacity of infill plates is referred to as SPSW2. The discrepancies between both specimens are the different thickness of infilled panels, and the connection details between the infilled steel shear plate and the boundary steel members. The cross sectional dimension of each component for both specimens and the average value of steel mechanical properties are shown in Table 1.

Table 1 Section size and mechanical properties of two specimens

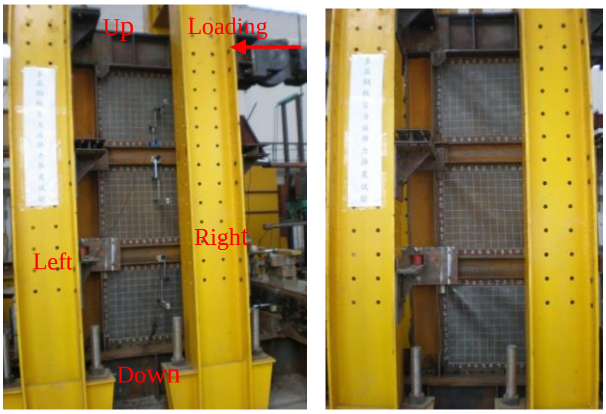
Element		Section size (mm)	Position	Mechanical properties			
				Sectional area A(mm^2)	Yield strength δ_y(MPa)	Ultimate strength δ_u(MPa)	Elongation δ(%)
Frame steel beam	1-2 storey	H150×100×6×9	Web	158.0	274.4	409.9	29.6
			Flange	206.0	308.8	471.1	26.3
	Roof	H300×200×8×12	Web	190.1	258.1	413.9	29.7
			Flange	260.2	269.3	428.1	30.5
Frame steel column		H200×200×8×12	Web	183.9	281.7	437.6	27.1
			Flange	262.9	274.0	429.8	28.5
Measured shear	SPSW1	910×610×3.75		91.5	267.9	411.2	31.3

plate (width×height×thickness)	SPSW2	930×630×1.1	27.6	261.7	339.9	34.8

The thickness of infilled shear panels welded to the connection fish plates is 3.75mm in the specimen SPSW1,while the thickness of shear plate fixed on the fish plates (using structural adhesive JIN-A, high strength bolt and angle steel) is only 1.1mm in the specimen SPSW2. This is because the panels are too thin to be welded to the connection plates. SPSW1 has been tested under the static pushover monotonic loading, and the SPSW2 has been tested under the low horizontal cyclic loading. The detailed loading schemes and device of SPSW1 and SPSW2 are shown in Fig.1.

Fig.1 Loading device

The whole out-of-plane deformations of infilled shear plates for the two specimens before loading and after ultimate failure are shown in Fig.2 and Fig.3, respectively.



(a) Before loading (b) Ultimate deformation

Fig.2 Overall deformation of specimen SPSW1

(a) Before loading (b) Ultimate deformation

Fig.3 Overall deformation of specimen SPSW2

As presented in Fig.2 for the thick SPSW1, an inclined tension field, which is about 45 degrees along the horizontal direction, appears in the infilled shear plate at the ground story and second story. The out-of-plane deformation of shear panel occurs uni-directionally and a hump wave is formed. However, the uneven out-of-plane deformation of the shear plate at the roof occurs in two directions. At the same time, two inclined tension fields appear approximately 45 degrees along the horizontal direction.

As shown in Fig.3, the out-of-plane displacement of infilled plate for the thin SPSW2 occurs bi-directionally and the deformation can be visually observed. In addition, a number of inclined tension fields form about 45 degrees along the horizontal direction in every shear plate. The thickness of the thin SPSW2 is so thin that out-of-plane displacement can occur when the horizontal load is still small. Therefore, the out-of-plane deformation and tension fields of thin SPSW can be rather more apparent than that of thick SPSW.

3. Structural performance evaluation

3.1 Base shear versus roof drift

The integral relationship between the base shear V versus roof drift angle θ for the specimens SPSW1 and SPSW2 obtained from the model tests can be found in Fig.4. The monotonic pushover behaviour of SPSW2 has been obtained from the hysteretic response shown in Fig.5.

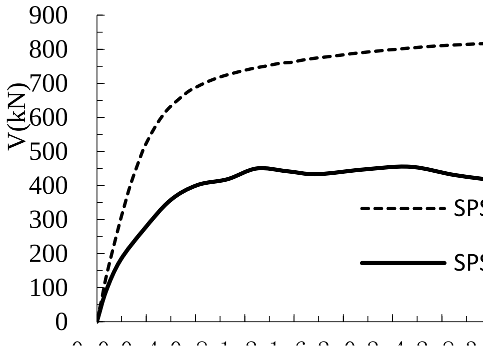


Fig.4 Integral *V*-*θ* curve

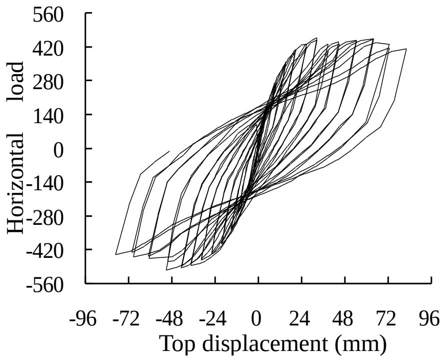


Fig.5 Hysteretic behavior of specimen SPSW2

As seen in Fig.4, the roof drift angle of the specimen SPSW1 reaches 0.0335rad at the ultimate condition, while that of specimen SPSW2 reaches 0.0328rad. Note that the elasto-plastic inter-storey drift limit given in the "Code for seismic design of buildings" is 0.02rad. The experiments in this study reveal that both specimens can continue to resist additional horizontal load when the drift angle exceeds the limit of 0.02rad while still can further take the vertical burden. When further loaded until structural failure, the bearing capacities of both SPSW1 and SPSW2 decline very slowly after reaching the peak ultimate condition. Although the inter-story drift is very large, the horizontal bearing load slightly decline to 85% of its maximum bearing capacity. It can be seen that both specimens have an acceptable serviceability performance in terms of drift angle. Considering from the full load-deformation curve, it can be observed that the bearing capacity of the specimen SPSW1 is greater than that of SPSW2. Under identical performance-based conditions specified by the code requirements, it is apparent that the design method for SPSW1 is overly conservative. On the other hand, the design method for SPSW2 using the shear bearing capacity method of infilled plate offers more optimal performance indices taking into account the economic factor, reliability and safety.

3.2 Lateral stiffness

Lateral stiffness is the ratio of horizontal load to the corresponding displacement. The specimen SPSW2 has been subjected to the low horizontal cyclic load. For comparisons, a type of lateral stiffness called "equivalent stiffness" based on the structural hysteretic response of horizontal force-displacement curve is defined. It describes a slope of the line connected between the peak load point of the hysteretic loop and the zero load point of the last step.

The lateral stiffness of SPSW1 and the equivalent stiffness of SPSW2 at different loading stages are shown in Fig.6.

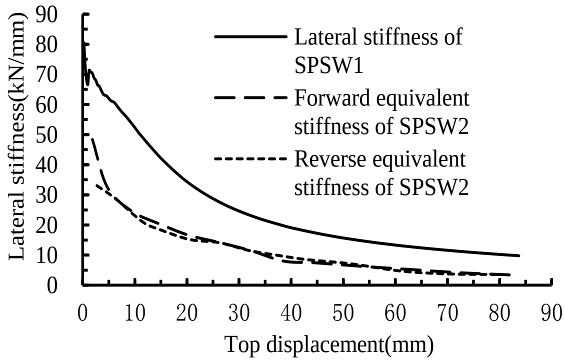


Fig.6 Lateral stiffness

As seen in Fig.6, the maximum lateral stiffness of both specimens appears at the beginning of loading stage and the structures behave elastically. The maximum integral lateral stiffness of SPSW1 is 80.5kN/mm. The equivalent stiffness of SPSW2 is 48.4kN/mm when the SPSW2 is being pushed forward. When the SPSW2 has been reversely pulled, the equivalent stiffness becomes 33.7kN/mm. The stiffness of SPSW1 is greater than that of SPSW2 because infilled steel plate of SPSW1 is thicker than that of SPSW2 and SPSW1 has better ability to resist deformation under the horizontal load compared with SPSW2.

During the elastic and elasto-plastic stage of the structure, the lateral stiffness for both specimens decreases significantly. The specimen SPSW1 shows a small rebound, while the SPSW2 decreases slightly compared with SPSW1. During the yield stage, the decrease in magnitude of lateral stiffness can be observed from these curves. However, the decrease in lateral stiffness of SPSW2 is still gentle compared with that of SPSW1. In the post-yield stage, the decline rate of lateral stiffness decreases significantly and tends to be stable. This is due to the fact that the out-of-plane buckling of a shear plate occurs instantly during the loading process. It is found that the larger the loading displacement, the more the residual out-of-plane displacement, and the smaller the lateral stiffness of structure. However, when the resulting displacement increases to a certain extent, the out-of-plane deformation of infilled plate tends to be stable. Since the embedded steel shear plate of SPSW2 is thinner, the plate can form out-of-plane buckling earlier but tends to be more stable than SPSW1 during the loading process. On this ground, the overall equivalent stiffness of SPSW2 decreases slightly. Due to the thickness of shear panels, the lateral stiffness of thin SPSW is lesser than that of thick SPSW.

3.3 Ductility

Ductility is a measure of plastic deformation capacity of a structure, a component or material. It is usually expressed by a ductility factor. Based on experimental measurements, is the load-deformation curve can inform curvature ductility coefficient and displacement ductility coefficient. The displacement ductility is the ratio between ultimate displacement and yield displacement.

$$\mu = \frac{X_u}{X_y} \tag{1}$$

In which, μ is the displacement ductility coefficient of a structure or a component, and X_u is the ultimate displacement, and X_y is the yield displacement.

Due to the intrinsic characteristics of high ductility of steel plate shear wall, the shear bearing capacity of each specimen declines very slowly, even though the inter-story drift is very large. The horizontal bearing load slowly declines to 85% of its maximum bearing capacity. At this condition, the steel column base appears to have large distortion deformation, and the displacement at the end of test is considered as the ultimate displacement. It is found that the ultimate displacement is much smaller than the specific displacement when the maximum bearing capacity declines to about 85%. The displacement ductility coefficients of both specimens are then calculated by the ultimate displacement at this condition as shown in Table 2.

Table 2 Yield load, displacement and displacement ductility coefficient at each story of specimens

Position	SPSW1				SPSW2							
					Forward loading				Reverse loading			
	Yield load (kN)	X_y (mm)	X_u (mm)	μ	Yield load (kN)	X_y (mm)	X_u (mm)	μ	Yield load (kN)	X_y (mm)	X_u (mm)	μ
Ground story	689.2	5.05	22.82	4.52	374.1	4.75	26.7	5.62	394.3	5.37	15.19	2.83
Second story	661	6.91	35.74	5.17	382.6	6.94	33.66	4.85	410.7	10.95	34.67	3.17
Roof	616.9	4.21	25.5	6.06	368.5	5.84	22.45	3.84	420.4	4.59	22.28	4.85
Overall	646.8	16.51	86.58	5.24	338.1	14.0	81.92	5.85	408.1	22.57	79.32	3.51

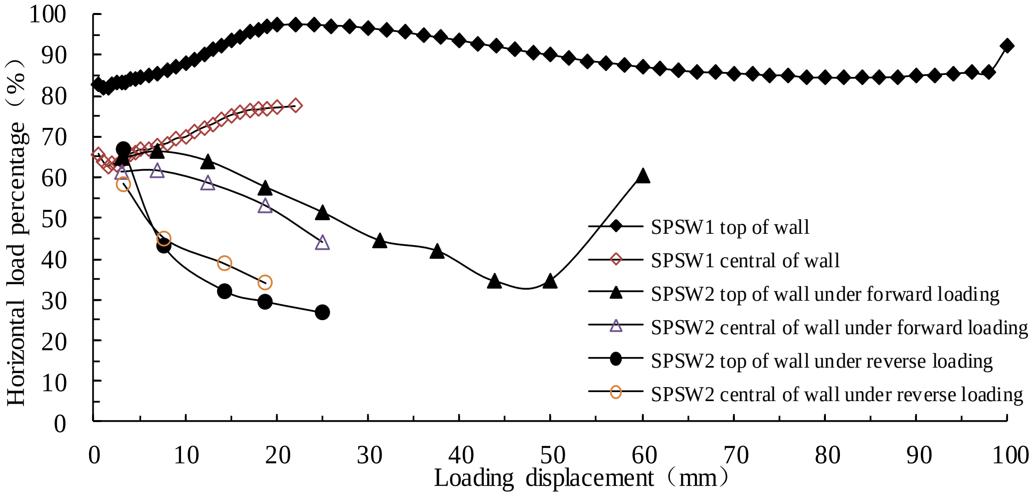
It can be seen from Table 2 that, from the overall yield displacement, the yield displacement of SPSW1 is 16.51mm, while that of SPSW2 is 22.57mm. This is because the steel shear plate embedded in SPSW2 is thinner than that of SPSW1, and the overall lateral stiffness is smaller. Therefore, larger deformation is more likely to occur in SPSW2 towards the yielding stage. However, on a basis of overall ultimate displacement, the maximum value of SPSW2 is 81.92mm, while that of SPSW1 is 86.58mm. This was due to the fact that, from the stage of yielding to the failure limit, SPSW2 is more prone to early failure due to thin steel plate and low lateral stiffness. As a result, its overall ultimate displacement is smaller than that of SPSW1.

The overall displacement ductility coefficient of specimen SPSW1 is 5.24. The displacement ductility coefficients obtained by the forward loading and the reverse loading of specimen SPSW2 are not equal, and the difference is large. The difference of displacement ductility factor between the forward loading and the reverse loading at the bottom story is the largest, up to

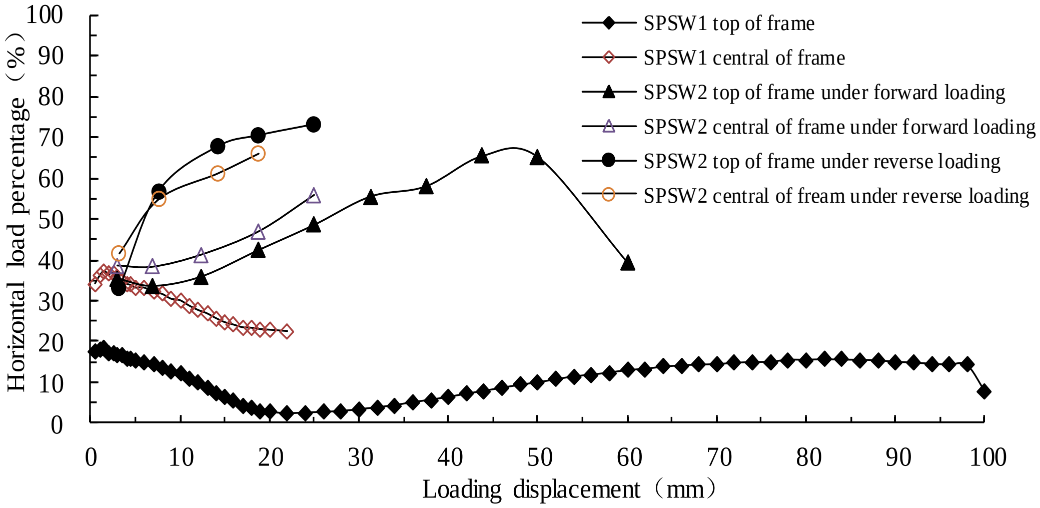
2.52 times, while the difference between the positive and reverse loading at the second story and the roof is relatively small. The overall displacement ductility coefficient of SPSW2 under forward loading is 5.85, which is 1.67 times of 3.51 under reverse loading. The specimen SPSW2 has relatively higher ductility compared to SPSW1. Therefore, the thin SPSW tends to be more ductile than the thick SPSW.

3.4 Distribution of horizontal force and overturning moment

Axial strain gauges have been installed on the inner and outer flanges of the frame steel column, and the three-direction strain rosettes are set on the middle of web. Based on the data derived from strain gauges and sensors (LVDT), the deformations and stresses of the frame column can be obtained, and the corresponding shear and axial forces of this column can be calculated. Then, the shear bearing capacity and overturning moment percentages of the frame and the shear plate can be respectively calculated, so that the insight into force transmission mechanism of SPSW structure can be established accurately in great details. Taking the bottom story of both specimens as an example for comparative analyses, the horizontal shear bearing capacity and overturning moment percentages of the boundary frames and the infilled plate at different parts of the bottom floor under different loads are shown in Fig.7 and Fig.8, respectively.

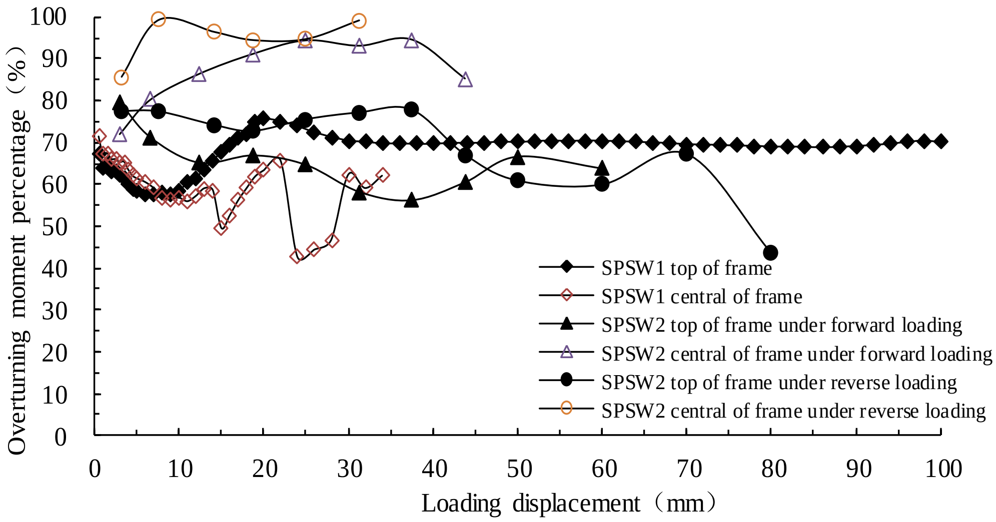


（a）Infill wall plate at the bottom story

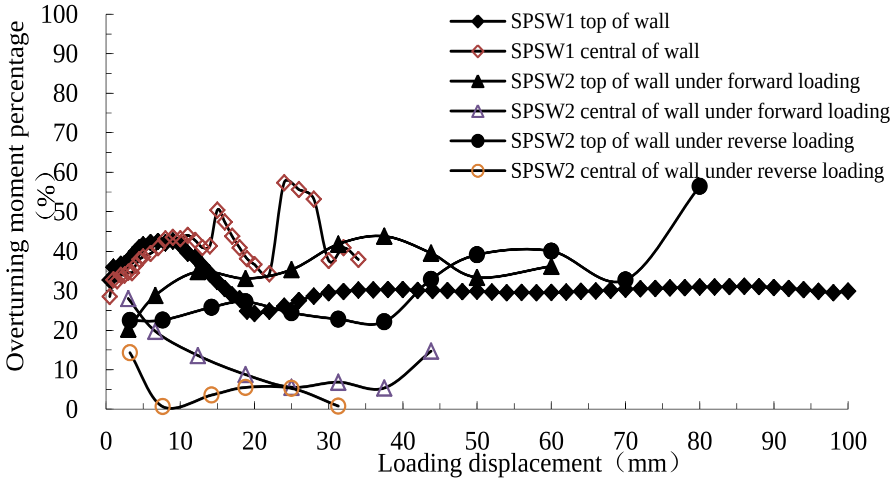


（b）Frame at the bottom story

Fig.7 Percentage of horizontal force between frame and shear wall under various load peaks



（a）Frame at the bottom story



（b）Infill wall plate at the bottom story

Fig.8 Percentage of overturning moment between frame and shear wall under various load peaks

It can be seen from Fig.7 that the percentage of horizontal load across the boundary steel frame and shear plates changes with the increase in loading displacement. For SPSW1, the horizontal load percentage of the infilled shear panels is much higher than that of the steel frame. In general, the proportion of horizontal load arisen from shear plates is about 70%-80%, while that of the frame is about 20%-30%. For SPSW2, when the shear panels are in elastic state prior to yielding, the proportion of horizontal load assumed by the infilled plates is approximately 60%-65%, while that of the frame is about 35%-40%. The proportion of horizontal load arisen from the shear plates decreases gradually with the increase in loading displacement, while that from the steel frame increases gradually. For SPSW2, the percentage of shear force in the boundary steel frame is larger than that of SPSW1, while the percentage of horizontal load arisen from the shear wall is smaller than that of SPSW1. This is because the shear plate of SPSW2 is very thin, and the shear plate is also prone to out-of-plane elastic buckling under a low horizontal load. The larger the loading displacement is, the higher the buckling degree of infilled plates will be, and the smaller the lateral stiffness of shear walls will be. Therefore, the proportion of horizontal load taken by shear wall will gradually decline, whilst the bearing proportion of the frame will gradually increase.

Fig.8 shows that the overturning moment percentage taken by the boundary steel frames and shear walls of both specimens change with the increase in loading displacement, but the behaviour is different from the characteristics of horizontal load proportion. Overall, the steel frame carries a larger proportion of overturning moment than the shear wall. As for SPSW1, the proportion of overturning moment taken by the frame is about 60-70%, while the shear wall takes 30%-40%. For the specimen SPSW2, the overturning moment of the steel frame is much larger than that of the shear wall. The proportion of overturning moment taken by the frame is as high as 75%-95%, while the shear wall carries only 5%-25% of the overturning moment.

Considering the whole structural system, the horizontal force shall be mainly taken by the shear wall and overturning moment should be mainly carried by the steel frame. Based on the structural system design, in terms of the horizontal load, SPSW1 complies well with the design method "shear wall bear most of the shear force" for SPSW structures. The percentage difference of horizontal force taken by the shear wall between both specimens varies by 10%-15%. With respect to the overturning moment, SPSW2 satisfies completely the design method "frame bear most of the overturning moment" for SPSW structures. Its performance is also better than that of SPSW1. The percentage difference of overturning moment taken by the steel frame between both specimens is 15%-25%. Clearly, SPSW2 has performed better for the overturning moment redistribution. Based on the experimental results, the thin SPSW performs better than the thick SPSW.

3.5 Inclined angle of tension field

According to the measured data derived from the three-direction strain rosettes installed at all four corners of each shear plate, the inclined angle α of tension field between the first principal stress and the horizontal direction of panel can be identified by using the following formula.

$$tg\left(2\alpha\right)=\frac{-\gamma_{xy}}{\varepsilon_x-\varepsilon_y} \qquad (2)$$

It is found that the overall deviation ranges between the first principal stresses and the horizontal stress of the infilled shear plates of SPSW1 are 37°-53°, while those of SPSW2 are 30°-51°. Therefore, it can be suggested that the inclined angle of the tension field of shear panels should reasonably be taken approximately as 45° for the design of SPSW structures.

3.6 Structural design performance

In terms of elastic behaviour, the performance parameters including over-strength factor, strength assurance coefficient and drift angle are used to evaluate the structural design criteria. Strength assurance coefficient refers to the ratio of actual structural ultimate strength to design strength, which reflects a degree of structural safety. As shown in Fig.9, over-strength factor Ω is expressed by the following equation [19].

$$\Omega=V_y/V_d \qquad (3)$$

Where V_y is the base shear corresponding to the maximum inelastic displacement, and V_d refers to the seismic force after reduction, namely the seismic design base shear based on strength.

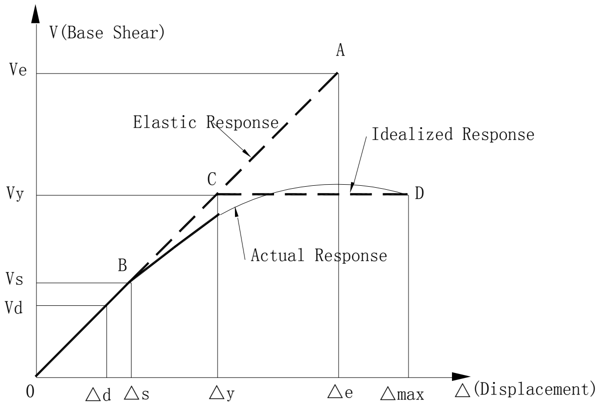


Fig.9 Typical structural response

The designed similarity ratio of both specimens is 1/4. However, due to the negative tolerance of fabricated material size, the actual similarity ratio of both specimens is somewhat different from the initial value. Since the shear wall panels in the SPSW structure carry most of the horizontal forces, according to the similarity principle, the relationship between the shear plate thickness of an actual specimen and the design prototype can be used to determine the horizontal design load of each specimen.

The design loads of these two prototype structures are both 1562kN.

For SPSW1, the design load is based on the following equation.

$$V_{\mathrm{d}}=\frac{V}{\left(t_1/t_2\right)^2}=\frac{1562}{\left(13/3.75\right)^2}=130\mathrm{kN}$$

Where V is the design load of prototype structure, t_1 and t_2 refer to the shear plate thickness of the prototype structure and the specimen, respectively.

For SPSW2, the design load is as follows.

$$V_{\mathrm{d}} = \frac{1562}{\left(3.2/1.1\right)^2} = 184.6\mathrm{kN}$$

The static pushover curve of SPSW1 and the predicted skeleton curve of SPSW2 as well as the design load and yielding force of both specimens are shown in Fig.10.

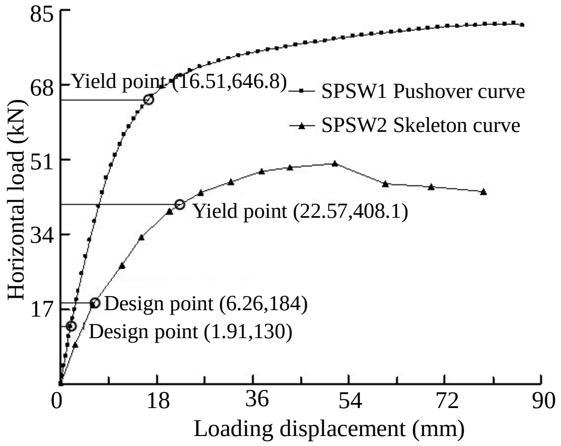


Fig.10 Horizontal load versus roof displacement, design point and yield point of two specimens

It can be seen from Fig.10 that, with the increase in loading displacement, the shear bearing capacity of SPSW1 is rising until it reaches the structural failure. For SPSW2, the bearing capacity increases at the beginning. However, when it is loaded to a certain level of displacement, the horizontal load decreases. It can also be concluded from Fig.10 that the design strength V_{d} of SPSW1 is 130kN and its corresponding yield strength V_{y} is 646.8kN. In addition, the design strength of SPSW2 is 184.6kN and its yield strength is 408.1kN.

3.6.1 Over-strength factor

The structural over-strength factors of both specimens are as follows.

For SPSW1： $\Omega_0 = \dfrac{V_{\mathrm{y}}}{V_{\mathrm{d}}} = \dfrac{646.8}{130} = 4.98$

For SPSW2： $\Omega_0 = \dfrac{V_{\mathrm{y}}}{V_{\mathrm{d}}} = \dfrac{408.1}{184.6} = 2.21$

According to the American Seismic Provisions for Structural Steel Buildings [20], the over-strength factor for economical, reasonable and safe lateral force-resisting building systems should be between 2 and 3. It can be observed that, the over-strength factor of SPSW1 is 4.98 and that of SPSW2 is 2.21. Therefore, the structural load carrying capacity of SPSW1 is obviously too conservative. On this ground, the thin SPSW designed by the shear capacity of infilled plates is more reasonably economical.

3.6.2 Strength assurance coefficient

From Fig.10, the strength assurance coefficients of both specimens can be calculated as follows.

For SPSW1： $k = \dfrac{V_{\mathrm{u}}}{V_{\mathrm{d}}} = \dfrac{818.7}{130} = 6.3$

For SPSW2： $k = \dfrac{V_{\mathrm{u}}}{V_{\mathrm{d}}} = \dfrac{501}{184.6} = 2.71$

Where, V_{u} refers to the base shear of structure at the ultimate failure state.

In general, a recognized strength assurance coefficient should be about 3.0. For SPSW1, the assurance coefficient is 6.3, which is overlyconservative. Note that the strength assurance coefficient of SPSW2 is 2.71, which illustrates that the design of the thin SPSW is more reasonable compared to the thick SPSW.

3.6.3 Design drift angle

The total height of both specimens is 2550 mm. In light of roof displacement deformation, the roof displacement angle of SPSW1 when it reaches the designed load 130kN can be expressed by the following equation.

$$\theta = \frac{\Delta_d}{H} = \frac{1.91}{2550} = \frac{1}{1335}$$

When the horizontal force of SPSW2 reaches the design load of 184.6kN, the roof displacement angle is as follows.

$$\theta = \frac{\Delta_d}{H} = \frac{6.26}{2550} = \frac{1}{407}$$

The maximum elastic inter-story displacement angle limit given by the “Code for the seismic design of buildings” is 1/300 under the action of frequent earthquakes. The roof drift angle of the thick specimen SPSW1 is only 1/1335, which is far less than the limit required by the code. While the roof drift angle of thin specimen SPSW2 is 1/407, which is still slightly less than the limit requirement in the specification. It is clear that the thin specimen SPSW2 meets the design requirements and has superior advantages of being more reasonable and economical in design.

4. Conclusions

Based on two different design methods given in “Technical specification for steel structure of tall buildings” and the shear bearing capacity method for infilled shear plates, two designed specimens of the steel plate shear wall have been built and tested under the static monotonic pushover and low horizontal cyclic loading conditions, in order to investigate the structural behaviors of the shear wall when exposed to a severe earthquake. The comparative analysis of different mechanical seismic performance has been carried out and the following conclusions are obtained.

Firstly, both specimens have satisfactory seismic performance in terms of overall drift angle. They can continue to carry loads after the structure behaves beyond the elasto-plastic displacement limit specified by the national standards. However, the load carrying by the thick SPSW is always greater than that of the thin SPSW. Under the test conditions, the design of the thick SPSW appears to be overly conservative, whilst the design of the thin SPSW is rather economical and reasonable.

Secondly, the lateral stiffness of the thick SPSW is higher than that of the thin SPSW because the thickness of infilled shear plate is larger. The first principal stresses along the horizontal direction of both infilled shear plates are relatively close. It is suggested that the inclined angle of tension field of shear panels reasonably be taken as 45° for the design of SPSW structures.

Thirdly, by design, the horizontal force shall be mainly carried by the shear wall and overturning moment should be mainly taken by the steel frame for the overall SPSW structures. The thick specimen SPSW1 complies well with the design method "shear wall bears most of the shear force" for SPSW structures. On the other hand, the thin specimen SPSW2 complies withthe design method "frame bears most of the overturning moment". However, it performs better than the thick specimen SPSW1.

Fourthly, the seismic elastic design of both specimens can be evaluated by three performance indices including over-strength factor, strength assurance coefficient and design drift angle. For the thick specimen SPSW1 designed by the domestic codes, the over-strength factor is 4.98, strength safety coefficient is 6.3 and the design drift angle is 1/1335. However, for the thin specimen SPSW2 designed according to the shear bearing capacity method for infilled shear plates, three performance indexes are 2.21, 2.71 and 1/407, respectively. It is noted that the generally accepted over-strength factor should be between 2 and 3, the reasonable strength assurance coefficient is about 3.0, and the maximum elastic inter-story displacement angle limit given in the codes is 1/300 under the action of frequent earthquakes. Therefore, it is apparent that the thin SPSW can satisfy all of the performance indices and enable better economic benefit. Lastly, based on the comprehensive analyses of the performance indices, the thick SPSW, which is strictly designed in accordance with the specification in Chinese code together with the principle of yielding in shear before buckling for the infilled plates, is overly conservative and not economical. This method unnecessarily increase the amount of steel required, which is not attractive for the application and promotion of the structural steel system. In contrast, the thin SPSW designed according to the shear capacity method for the infilled plates, which allows the shear plate buckling under the design load, can be more economical.

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