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Experimental study of low-yield-point steel plate shear wall under in-plane load

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ABSTRACT

This paper describes the study of the low-yield-point (LYP) steel plate shear walls under in-plane load. In the LYP steel plate shear wall system, LYP steel was selected for the steel plate wall while the boundary frame was constructed by the high strength structural steel. A series of experimental studies examined the inelastic shear buckling behavior of the LYP steel plate wall under monotonic in-plane load. The effects of width-to-thickness ratio on the shear buckling of LYP steel plates were examined. The stiffness, strength, deformation, and energy dissipation characteristics were investigated by performing cyclic loading tests on the multistorey LYP steel plate shear walls. Excellent deformation and energy dissipation capacity were obtained for all specimens tested. The LYP steel plate shear wall system is able to exceed 5% of storey drift angle under lateral force.

1. Introduction

In the current seismic resistant design, building structures are allowed to exceed their elastic limit under severe earthquake excitation. However, brittle collapse of a building should be prevented. Besides strength requirements, stiffness is another concern in a structural design. With high strength and high stiffness, the steel plate shear wall has drawn many engineers' attention. Research works have been carried out on the steel plate shear walls. Experimental studies have been carried out on the thin steel plate shear walls by Caccese et al. [1], Driver et al. [2] and Lubell et al. [3]. Analytical studies on the shear buckling behavior of steel plate wall and the behavior of a multi-storey steel wall system were conducted by Elgaaly et al. [4,5], Driver et al. [6], Berman et al. [7] and Sabouri-Ghomi et al. [8]. Design rules of the thin steel plate shear wall are also specified in the design specifications, such as AISC [9] and CSA [10].

Steel plate shear wall systems have been applied in building construction. The steel plate used in the shear wall is usually very small in thickness due to the high strength steel used. The elastic shear buckling of the thin plate steel shear wall usually results in reduced strength, stiffness and energy dissipation capacity. Although the tension field action is able to provide the post-buckling strength, however if the shear buckling occurred in the early stage, out-of-plane permanent deformation may affect the serviceability of the thin plate shear wall under small or moderate earthquake. To defer the shear buckling and increase the energy dissipation capacity, stiffening devices can be used for the plate wall. These can be done by adding steel stiffeners which is quite common in Japan. A composite shear wall that adopts reinforced concrete to restrain the steel plate wall was also reported with good seismic resistance by Zhao and Astaneh-Asl [11].

A new type of structural steel, Low-Yield-Point Steel (LYP steel), has been developed and applied in seismic resistant design [12]. The LYP steel possesses extremely low yield strength and high elongation capacity. The yielding stress of this type steel can be as low as 100 MPa (LYP-100 steel), which is about one-fourth of the conventional structural steel such as ASTM A572-Gr. 50 steel. In this study, LYP-100 steel was selected for the plate wall and ASTM A572 Gr. 50 steel was used for the boundary frame. Fig. 1 shows the stress–strain relationships of LYP and A572 steel used in this research. It is shown that the LYP steel has a superior elongation capacity and significant strain hardening in the post-yield stage. The elongation capacity of LYP steel is more than 50%, which is much larger than that of conventional structural steel. The LYP steel also possesses low yield ratio, the ratio between the yield stress and the ultimate stress (F_y/F_u), which is only 0.34. With low yield ratio, the structure that utilizes LYP steel is able to redistribute the inelastic stress easily and provides a larger plastic zone. Due to its superior deformation capacity, LYP steel was also used in the steel dampers to dissipate earthquake energy as reported by Saeki et al. [12], Chen and Kuo [13] and Knodo et al. [14]. LYP steel can be used in the steel shear wall as well. The hysteresis behavior of LYP steel plate has been examined and a two-force strip model was proposed to predict its in-plane strength by Chen and Jhang [15]. In this reported research, a series of experimental studies was conducted to examine the inelastic shear buckling behavior of the LYP steel plate wall unit under

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Nomenclature

F_{cr}	Shear buckling stress
E	Elastic Young's modulus
μ	Poisson's ratio
t	Thickness of the steel plate
η	Plastic reduction factor ($\eta = \sqrt{E_t/E}$)
E_t	Tangent modulus
k_v	Plate buckling coefficient
a, b	Widths of the panel that surrounded by the stiffeners or boundary members, $a > b$.

monotonic in-plane load. The stiffness, strength, deformation, and energy dissipation characteristics of the multi-storey LYP steel plate shear walls were investigated by performing cyclic loading. Based on these studies, design recommendations are proposed for the LYP steel shear wall.

2. Shear buckling of LYP steel shear wall

The shear buckling strength of the steel plate is governed by its width-to-thickness ratio, boundary condition and tangent modulus of the steel. The following equation has been suggested to predict the inelastic buckling stress of a thin plate subjected to in-plane load by Galambos [16],

$$F_{cr} = \frac{k_v \cdot \pi^2 \cdot \eta \cdot E}{12(1-\mu^2)} \cdot (t/b)^2. \quad (1)$$

In which, F_{cr} is the shear buckling stress, E is the elastic Young's modulus, μ is the Poisson's ratio, t is the thickness of the steel plate, η is the plastic reduction factor ($\eta = \sqrt{E_t/E}$), E_t is the tangent modulus, and k_v is the plate buckling coefficient. The plate buckling coefficient k_v can be obtained by the simplified equations as follows:

$$\text{simply support: } k_v = 5 + \frac{5}{(a/b)^2}, \quad (a > b) \quad (2)$$

$$\text{fixed support: } k_v = 8.98 + \frac{5.6}{(a/b)^2}, \quad (a > b). \quad (3)$$

The parameters a and b in the above equations are the widths of the panel that surrounded by the stiffeners or boundary members. It should be noted that LYP steel has ample strain hardening after yield, a plastic reduction factor η is used to reflect the post-yield strength of the LYP steel plate wall. An iteration process is needed to calculate the buckling stress F_{cr}. This can be done by assuming an η, and by an iterative process to calculate the critical stress F_{cr} until a converged value is obtained as shown in Fig. 2.

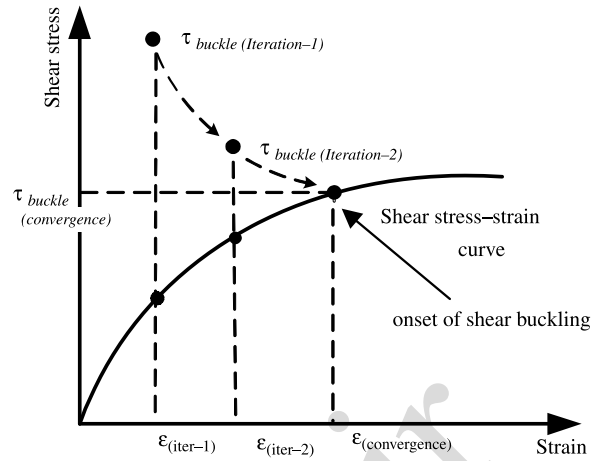
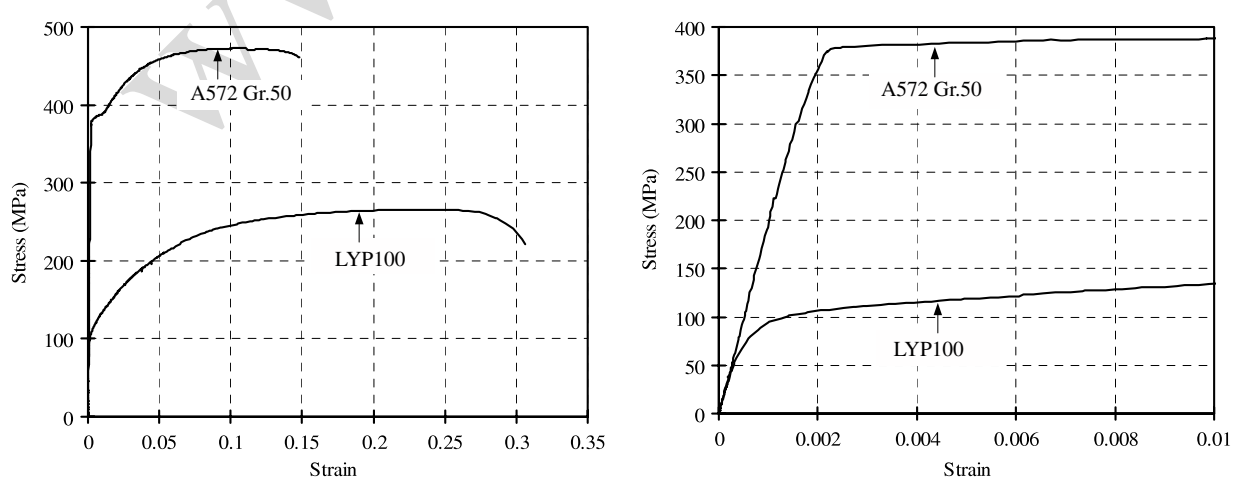


Fig. 2. Iterative calculation of shear buckling strength.

3. LYP steel shear walls under monotonic loads

A series of experimental studies is conducted to examine the in-plane buckling behavior of LYP steel shear wall. Table 1 lists the mechanical properties of the steel used. Fig. 3 shows the design of the specimen. The stiffeners used for the shear wall are 60 mm in width, 6 mm in thickness and spaced at 400 mm. Fig. 4 shows the test set-ups. In this series of studies, the LYP steel plate panels were subject to in-plane monotonic load. The parameter in this study was the width-to-thickness ratio of the LYP steel shear panel. Smaller width-to-thickness ratio of the panel is able to enhance the stiffness, strength and deformation capacity of the steel panel. Plate type stiffeners were used to reduce the width-to-thickness ratio of the panel. However, adding the stiffeners will increase the cost of the construction. In the design of the stiffener of the shear wall specimen, it was aimed to let the steel panel reach its plastic shear strength and dissipate energy effectively.

Four specimens were selected with the width-to-thickness ratios from 50 to 150, as listed in Table 2. It should be noted that there was no stiffener for Specimen Nos. 3 and 4. For these



(a) Complete stress–strain curve.

(b) Close-up around the yield point.

Fig. 1. Typical stress–strain curve of steel used.

Table 1
Mechanical properties of steel used.

Specimen	Plate thickness (mm)	F_y (MPa)	F_u (MPa)	F_y/F_u
A 572 Gr. 50	6	444.852	555.813	0.800
	9	425.206	550.452	0.772
	14	419.309	535.128	0.784
LYP-100	3.5	94.931	279.208	0.340
	6	93.768	274.961	0.341
	8	92.844	272.154	0.341
	15	85.433	258.426	0.331

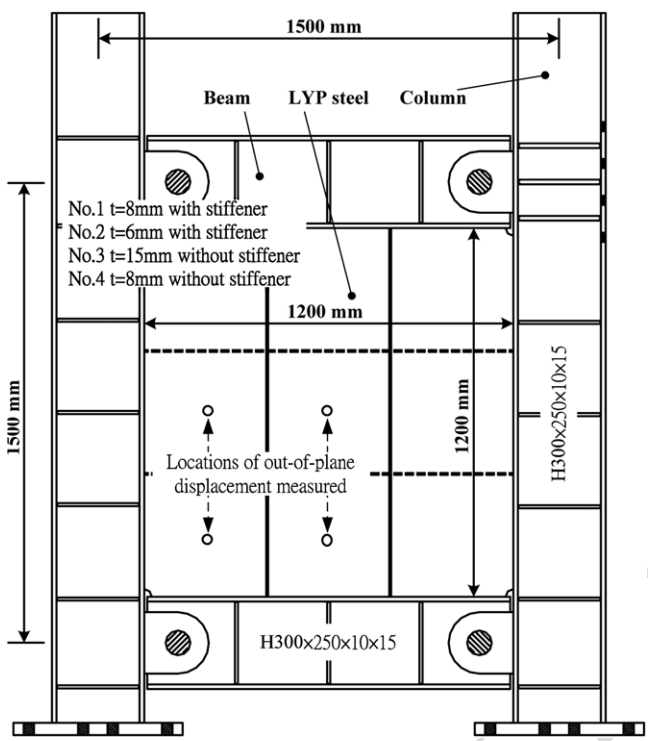


Fig. 3. Design of Specimen Nos. 1–4.

two specimens, their boundary beams and columns serve as the stiffeners of the panel and these boundary members provided much better restraints as compared with that of a single plate type stiffener. In the calculation of the buckling strength, Eq. (2) was selected for Specimen Nos. 1 and 2 due to their boundaries being similar to the simply supported condition. While Eq. (3) was selected for Specimen Nos. 3 and 4 since their boundaries were similar to the fixed condition. Table 2 lists the summary of the design parameters and the experimental results.

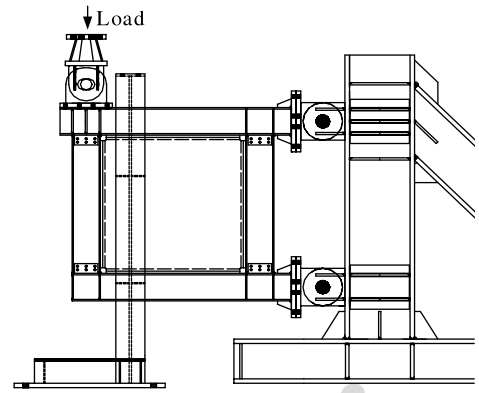


Fig. 4. Test set-ups for Specimen Nos. 1–4.

3.1. Deformation capacity

Fig. 5 shows the out-of-plane displacements of the tested specimens. The locations of the measurements are shown in Fig. 3. The rapid increase in displacement was due to the onset of the shear buckling. Fig. 6 shows the load and in-plane displacements (storey drift angles) of the specimens. All of the specimens reached drift angles between 0.04 and 0.05 rad without any decay on their strengths. These amounts of storey drift angles were considered satisfactory in the seismic design. The tests ceased after reaching the limitation of the stroke of the actuator. As shown in Fig. 6, significant plastic behavior was observed on Specimen Nos. 1–3 at the onset of shear buckling. The storey drift angles at the onset of buckling were approximately 0.02–0.03 rad for Specimen Nos. 1–3. Specimen No. 4 had the largest width-to-thickness ratio (150) and its drift angle was about 0.01 rad when shear buckling occurred. This study showed that adding stiffeners could defer the shear buckling and reduce the out-of-plane deformation of the shear plate wall. The experimental results also showed that by using LYP steel and keeping the width-to-thickness ratio less than 80 (Specimen Nos. 1–3), there was no visible out-of-plane deformation on the steel shear wall even the storey drift angle was as large as 0.02 rad. Based on these findings, it is suggested to keep the width-to-thickness ratio less than 80 in the design of the LYP steel wall for better performance.

3.2. Shear buckling strength

As described in the previous section, stiffeners were provided for Specimen Nos. 1 and 2. In the calculation of the ultimate strength of these steel panels, the boundary conditions were simplified as the simply supported edges. Eq. (1) was used to calculate their strength. The width-to-thickness ratios of Specimen Nos. 3 and 4 were 80 and 150 respectively. No stiffener was used in

Table 2
Summary of experimental and analytical results.

Specimen	No. 1	No. 2	No. 3	No. 4
Shear plate thickness (mm)	8	6	15	8
Width-to-thickness ratio	50	65	80	150
Plastic reduction factor	0.112	0.167	0.167	0.488
Plate buckling coefficient	$k_v = 5 + 5/(a/b)^2$		$k_v = 8.98 + 5.6/(a/b)^2$	
	10	10	14.58	14.58
Buckling strength (kN)	883.826	559.685	1355.242	558.918
Drift angle of shear buckling (rad)	0.029	0.019	0.022	0.009
Analytical buckling stress (MPa)	81.113	68.248	69.126	57.051
Experimental buckling stress (MPa)	92.063	77.744	75.287	58.222
Comparison of buckling stress (experiment/analysis)	1.135	1.139	1.089	1.021

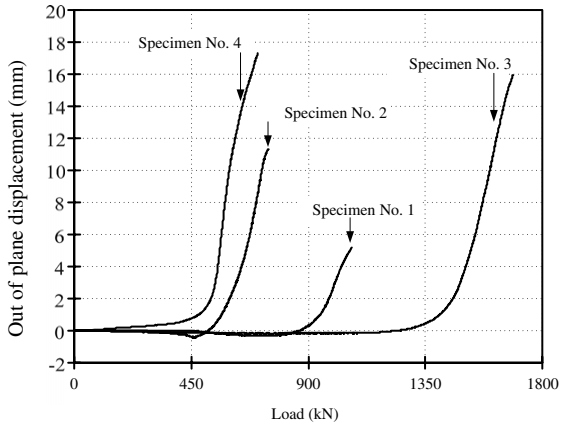


Fig. 5. Out of plane displacement.

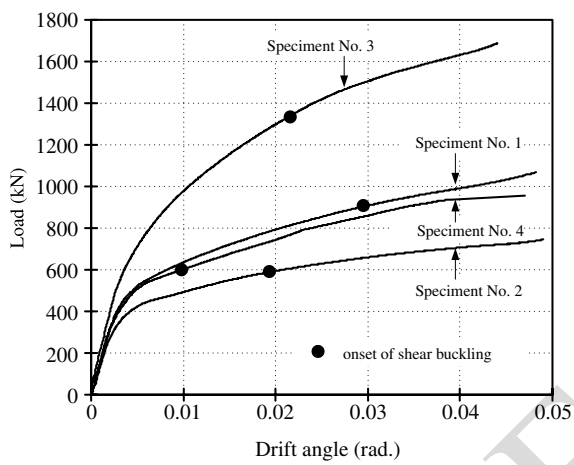


Fig. 6. Load–drift angle relationships.

these two specimens. The steel plates were connected to boundary beams and columns, which provided much stronger restraints as compared with that of conventional plate type stiffeners. The boundary conditions of Specimen Nos. 3 and 4 were classified as fixed and their strength were calculated based on Eq. (3).

Using Eqs. (2) and (3), the buckling strengths of the LYP steel shear wall were obtained and listed in Table 2. It was found that the difference between experimental results and those from Eqs. (2) and (3) were less than 14%. It should be noticed that the boundary

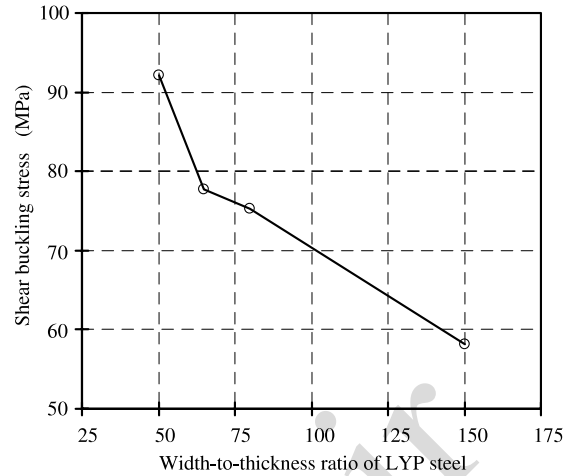


Fig. 8. Relationship between shear buckling stress and width-to-thickness ratio of LYP panel.

conditions for the LYP steel panels were different. For example, in Specimen Nos. 1 and 2, the boundary members around the panel unit varied because parts of the boundary were steel plate stiffeners and parts were the surrounding beams or columns. The restrains provided by the boundary members of the shear panel were between simple supported and fixed conditions. This was the main reason that the calculated strengths of Specimen Nos. 1 and 2 had larger variations with experimental results. However, even if Eq. (2) was used for the plate buckling coefficient, the calculated strength was on the conservative side. Therefore, it is suggested to use Eq. (2) if the boundaries of panel units consist of a plate type stiffener. If the panel is surrounded by larger members, such as beams or columns, Eq. (3) is suggested for the calculation of buckling strength of the panel. However, if the shear wall unit is surrounded by a combination of beams, columns, and plate type stiffeners, it is suggested to use Eq. (2) to ensure a conservative estimation of the restrain from the boundary members. With a proper plate buckling coefficient, Eq. (1) can be used to predict the shear buckling strength of the LYP steel panel with good accuracy.

Fig. 7 shows the shear buckling of Specimen Nos. 2 and 4. The relationships between the shear buckling strength and the width-to-thickness ratio of LYP steel panel are shown in Fig. 8. The specimens with smaller width-to-thickness ratio possess higher shear buckling strength. To enhance the shear buckling strength and energy dissipation capacity of the steel shear wall, the design of the LYP shear wall is aimed to exceed its yielding strength. From Fig. 8, it is found if the width-to-thickness ratio is less than 150, the

(a) Specimen No. 2. (b) Specimen No. 4.

Fig. 7. Shear buckling of Specimen Nos. 2 and 4.

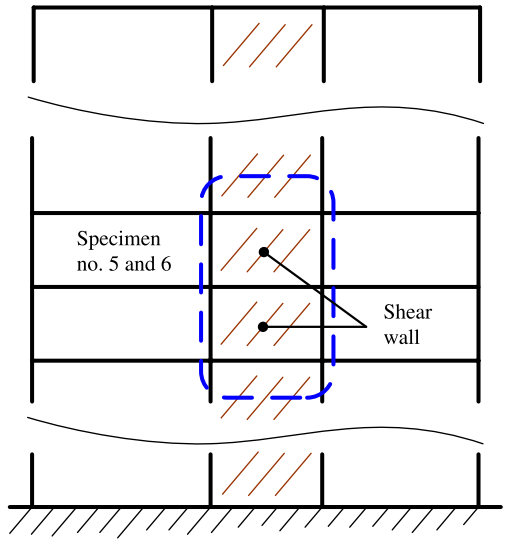


Fig. 9. Relative location of Specimen Nos. 5 and 6 in a steel shear wall building.

shear wall is able to reach the inelastic shear buckling stress which is $\tau_{cr} \geq \tau_y = 0.6F_y$. However, if the width-to-thickness ratio of the shear wall is further reduced to 80, the onset of shear buckling can be avoided up to 0.02 rad of storey drift angle, as shown in Fig. 6.

4. LYP steel shear walls under cyclic loads

Two specimens (Specimen Nos. 5 and 6) were designed to examine the behavior of multistorey LYP steel shear wall under cyclic load simulating the recursion of seismic excitation. The scale factor of these specimens was about 1/3–1/4. Fig. 9 shows the relative schematic locations of the specimens in a steel shear wall system. The low yield point steel plate LYP-100 was used for the shear wall, and conventional steel (ASTM A572 GR. 50) was selected for the boundary beams and columns. The LYP steel shear wall specimens were two stories with extension parts to simulate the lower stories and upper storeys in a multi storey building. One horizontal and three vertical stiffeners were added in both specimens so that the width-to-thickness ratio of the shear panel is 71. The design of Specimen Nos. 5 and 6 were the same, except that Specimen No. 5 adopted the rigid moment connections for its beam-to-column connections, while Specimen No. 6 adopted simple shear connections. These variations were aimed to examine the effects of rigid and simple beam-to-column connections of the boundary frame. The test setups and the detail design of the specimens are shown in Fig. 10 and Table 3.

The compressive stress due to gravity usually accelerates the buckling of a thin plate shear wall. It is suggested to connect the wall panel to the bottom flange of the upper beam first. After the erection works on the upper stories are completed, the plate wall is then connected to the surrounding columns and the top flange of the bottom beam. By this arrangement, the compressive force due to gravity load in the shear wall can be minimized. To simplify the experimental protocol, only one actuator was mounted on top of the shear wall specimen to simulate the seismic lateral force. Although the seismic lateral forces were different along the height of the building, it was assumed that the variation in two adjacent stories was not significant enough to affect the structural characteristic in the study of the shear wall system. The loading protocol applied in the experimental study is shown in Table 4. The loading rate (displacement rate) was kept under 0.7 mm/s. The specimens were subjected to cyclic load with increasing displacement amplitudes. The structural tests were stopped when either the floor storey drift angle exceeded 6%, or if the strength of the specimen degraded to a value less than 80% of its ultimate strength.

The stiffness, strength, deformation, and energy dissipation capacity are the major characteristics affecting the seismic performance of the steel shear wall system. The load–displacement relationships of the specimens tested are shown in Fig. 11. Both specimens showed significant inelastic tension field action with large storey drift angles at their ultimate strengths. Experimental

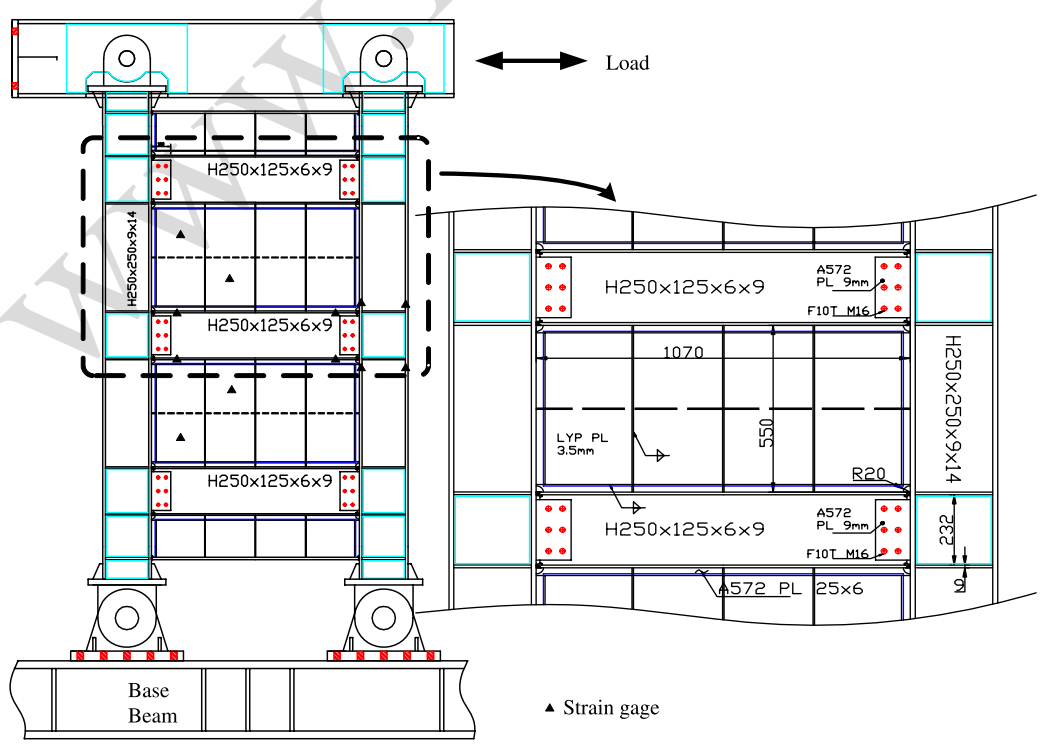


Fig. 10. Test setups of Specimen Nos. 5 and 6.

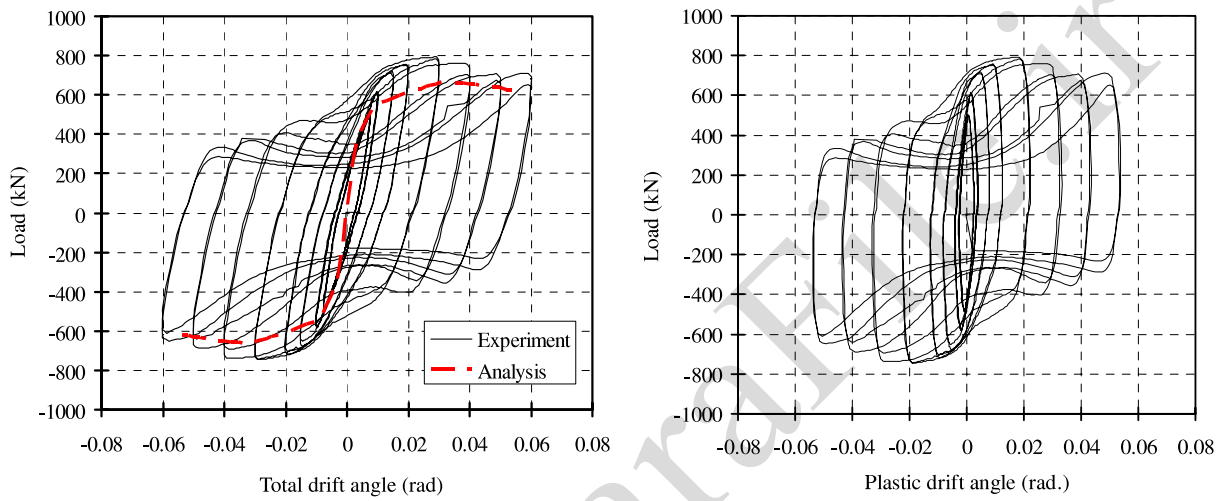
Table 3
Design of Specimen Nos. 5 and 6.

Specimen	Beam	Column	LYP plate	b/t	h/t	Beam-to-column connection
No. 5	H250 × 125 × 6 × 9	H250 × 250 × 9 × 14	3.5	71	71	Rigid
No. 6	H250 × 125 × 6 × 9	H250 × 250 × 9 × 14	3.5	71	71	Simple

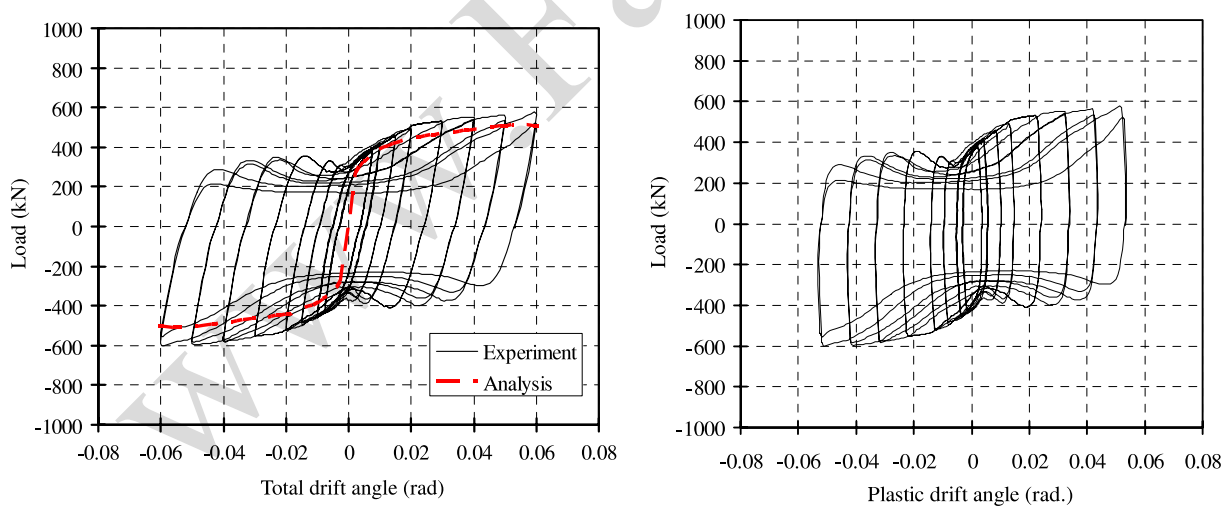
b/t: Width-to-thickness ratio of LYP plate.
h/t: Height-to-thickness ratio of LYP plate.

Table 4
Loading protocol of Specimen Nos. 5 and 6.

Drift angle (%rad)	0.375	0.5	0.75	1.00	1.5	2.0	3.0	4.0	5.0	6.0
Number of cycles	3	3	3	3	2	2	2	2	2	2



(a) Specimen No. 5.



(b) Specimen No. 6.

Fig. 11. Hysteresis behaviors of Specimen Nos. 5 and 6.

results are summarized in Table 5. Fig. 11 also shows the results from analytical studies by using a modified two-force strip model suggested by Chen and Jhang [15]. Good correlations were found between the experimental and the analytical results. The discussions of these studies are as follows.

4.1. Stiffness

Fig. 12 shows the decay in stiffness of both specimens. The stiffness is calculated based on the tangent stiffness of the last cycle of each displacement increment. It was found that due to the moment connections used for Specimen No. 5, its stiffness was larger than that of Specimen No. 6, which adopts simple connections for the beam-to-column connections. However, since the in-plane stiffness of the steel wall contributed to the major portion of the lateral stiffness of the steel wall system, the difference of elastic stiffness of the moment frame and simple frame was only about 9%. After the onset of yielding on the steel wall, the difference of stiffness between Specimen Nos. 5 and 6 was increased. For example, at the storey drift of 0.01 rad, the stiffness

Table 5
Summary of experimental results of Specimen Nos. 5 and 6.

Specimen	No. 5	No. 6
Yield strength (kN)	371.171	355.334
Ultimate strength (kN)	716.522	561.923
Yielding storey drift angle	0.005	0.005
Max. storey drift angle (total deformation)	0.06 rad	0.06 rad
Max. storey drift angle (plastic deformation)	0.05 rad	0.05 rad
Ductility ratio	12	12
Cumulated energy (kN m)	1681.714	1422.583

Note: Ductility ratio = maximum displacement/yielding displacement.

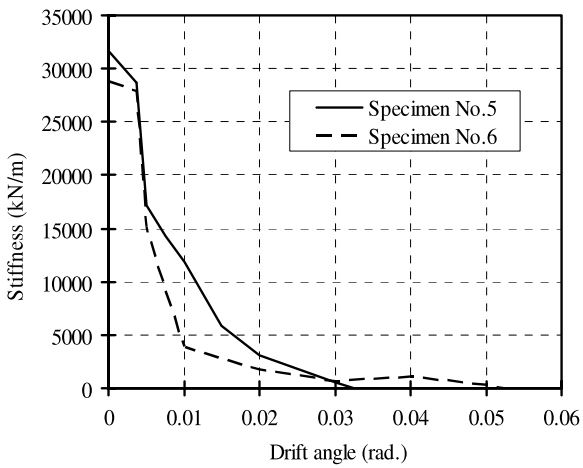


Fig. 12. Decay of stiffness of Specimen Nos. 5 and 6.

of Specimen No. 5 was about 30% larger than that of Specimen No. 6. After the beam-to-column connection of Specimen No. 5 was loaded into the plastic range, the stiffness of both specimens gradually became almost the same, as shown when the storey drift angle exceeded 0.02 rad (Fig. 12). This also showed that the stiffness of the shear wall system was mainly contributed by the LYP steel plate after the plastic hinges formed at beam-to-column connections.

4.2. Inelastic behavior and strength

In the design of a shear wall system, it is suggested to let the system remain elastic or minor plastic under the events of moderate earthquakes. During severe earthquakes, both the steel shear wall and the boundary frame are allowed to reach plastic stage, and a large amount of seismic energy can be dissipated primarily by the inelastic deformation of the steel shear wall. Due to high in-plane stiffness and low strength of the LYP steel shear wall, the shear wall yields prior to their boundary frame. The boundary columns and beams are designed to keep their elasticity as long as possible. This arrangement allows the system to maintain its stability and dissipate energy even after the shear buckling of the shear panel occurs. Fig. 13 shows the load–strain relationships at the LYP steel plate, column and beam-to-column connection. From the strain gage readings at the LYP steel panel and the beam-to-column connection, it was found that the LYP steel plate yielded at a storey drift angle of 0.0033 rad, while the beam-to-column connection yielded at a storey drift angle of 0.0127 rad. This also shows that the LYP steel plate yielded earlier than the frame.

The experimental results also showed that the plastic strain of LYP steel panel was much larger than that of the boundary frame under cyclic loading. When the beam-to-column connection yielded, the LYP steel plate had already yielded and even formed the tension field action. This also demonstrated that the LYP steel

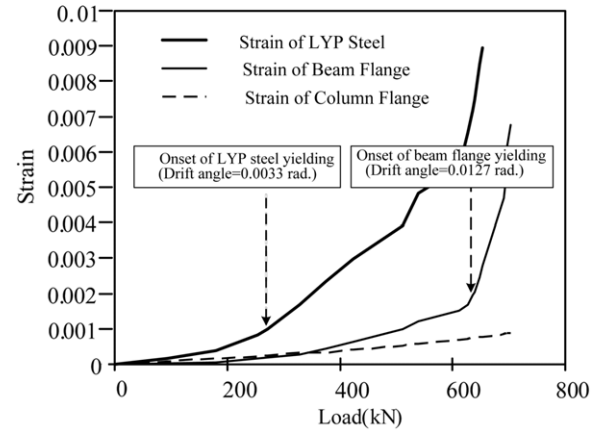


Fig. 13. Inelastic behavior on LYP steel panel, beam and column of Specimen No. 5.

shear wall yielded first and dissipated seismic input energy. In this study, Specimen No. 5 adopted the reduced beam section (RBS) type connections on its boundary frame suggested by Chen et al. [17] as shown in Fig. 14, while Specimen No. 6 adopted simple beam-to-column connections. With moment connections on the boundary frame, the ultimate strength of Specimen No. 5 is 28% larger than that of Specimen No. 6 as shown in Table 5. The enhanced strength of Specimen No. 5 was contributed from its moment connections on the beam-to-column joints. Table 5 also shows the energy dissipation capacity of Specimen No. 5 that with a moment connection is about 18% larger than that of the Specimen No. 6, which used the simple connection. Therefore adopting the ductile beam-to-column moment connection on the boundary frame can achieve higher strength and better energy dissipation capacity for the steel shear wall system. In this study, the moment connections on the boundary frame enhanced its strength and energy dissipation capacity by 28% and 18%, respectively, as compared with that of the simple connections on the boundary frame.

Although the strength of Specimen No. 5 with moment connections was greater than Specimen No. 6 with a simple connection, the storey drift angles of both specimens reached 0.06 rad. Since the simple beam-to-column connection was not able to contribute the lateral strength of steel shear wall system effectively, the strength of Specimen No. 6 is governed mainly by the LYP steel shear panel. Due to the strain hardening of LYP steel, a stable performance of strength in Specimen No. 6 was also obtained as shown in Fig. 11. This study also shown that both the rigid and simple connections on the boundary frame of LYP steel wall were able to have good seismic performance for LYP steel shear wall. However, if the strength of shear wall is a major concern in the design, it is suggested to use the moment frame instead of simple frame.

4.3. Deformation and energy dissipation capacity

Fig. 15 shows the deformation of Specimen Nos. 5 and 6 at the drift angel of 0.06 rad. The skeleton of hysteresis curves are shown in Fig. 16. Both specimens were able to reach storey drift angles of 0.06 rad without significant decay on their strengths. This shows that the LYP shear wall system is able to provide an excellent deformation capacity. Furthermore, the ductility ratios of both specimens are able to reach 12, as shown in Table 5.

Fig. 17 shows the cumulated energy versus drift angles. The total energy dissipation of Specimen No. 5 is 18% more than that of Specimen No. 5 as can be obtained from Table 5. It is believed that the rigid beam-to-column connection used in Specimen No. 5 increases its strength and energy dissipation capacity. Besides,

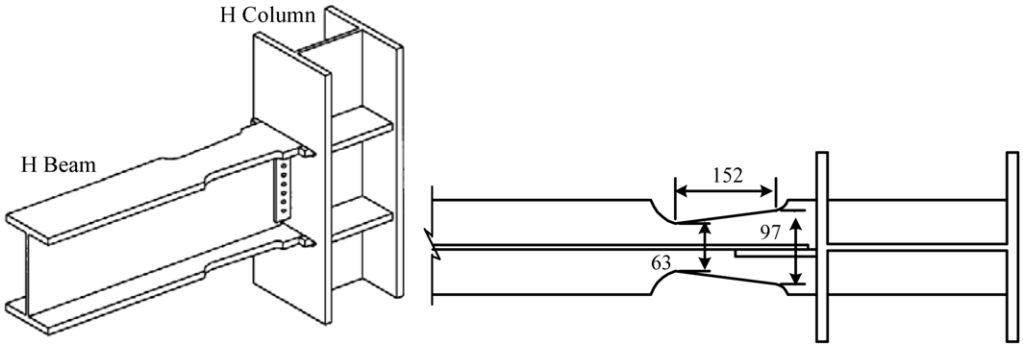


Fig. 14. Ductile beam-to-column connection.

(a) Specimen No. 5. (b) Specimen No. 6.

Fig. 15. Deformation of Specimen Nos. 5 and 6.

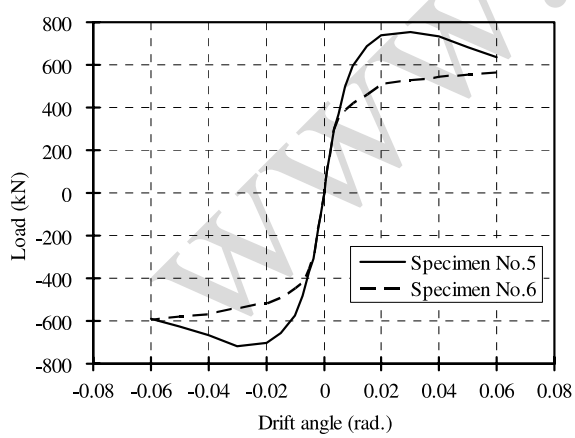


Fig. 16. Skeleton of hysteresis curves of Specimen Nos. 5 and 6.

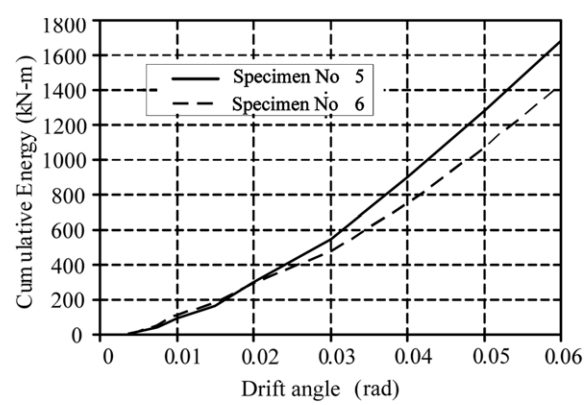


Fig. 17. Cumulated energy of Specimen Nos. 5 and 6.

from the hysteresis behavior under recursion of load shown in Fig. 11, it was found that the characteristics of energy dissipation of the LYP shear wall system were very stable and reliable. The pinching behavior that is usually observed in the conventional thin steel plate shear wall system under in-plane load is alleviated as shown in Fig. 11. Due to the low yield ratio, F_y/F_u, of LYP steel the shear wall is able to provide better capacity of stress re-distribution. With low yield ratio, the stiffness of the shear wall decays gradually after yielding and keeps ample stiffness to avoid soft storey due to yielding of the shear wall. However, it is suggested to check the ultimate strength of the wall by considering the strain hardening of the LYP steel to avoid excessive strength of the shear wall.

5. Summary and conclusions

The structural behaviors of low yield point (LYP) steel plate shear walls under in-plane load were examined in this reported

research. The experimental results showed that by using LYP steel and keeping the width-to-thickness ratio of the steel shear wall less than 80, there was no visible out-of-plane deformation on the steel shear wall even the storey drift angle was as large as 0.02 rad. It is suggested to keep the width-to-thickness ratio less than 80 in the design of the LYP steel wall for better performance. Experimental studies also showed that the LYP steel shear wall system was able to deform stably up to a storey drift angle of 0.05 rad. It was also found that an LYP steel shear wall with simple beam-to-column connection or rigid beam-to-column connection were able to provide good seismic resistance. However, the performance of steel wall with moment connections on its boundary frame was better than the system that with shear connections. The differences on the strength and energy dissipation capacity between these two systems were 28% and 18% respectively. It is suggested to use moment connections on the boundary frame of the LYP steel shear wall system.

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