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Experimental and theoretical studies of steel shear walls with and without stiffeners

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ABSTRACT

Steel plate shear walls are lateral load resisting systems, especially against earthquake excitation. They are constructed with or without stiffeners. In contrary to stiffened steel plate shear walls, there are many theoretical and experimental studies on these systems without stiffeners and different analytical methods have been presented for them which are mostly applicable to very thin steel plates shear walls. In this research, two one story similar steel plate shear walls with and without stiffeners and one of their surrounding frames were tested and the behavior of them was studied. The results showed that, installation of stiffeners improved the behavior of the steel plate shear walls. It caused 26% increase in energy dissipation capacity and 51.1% increase in the shear stiffness of steel plate while its effect on the steel plate shear strength was minor. In addition, the Plate-Frame interaction theory (PFI) was verified by using the experimental results and the test results showed that, this theory has good capability for predicting the shear load – displacement curve behavior of steel shear walls with or without stiffeners.

1. Introduction

Because of high capability of steel plate shear walls against lateral loads and the energy dissipation capacity of earthquake, using of steel plate shear walls has developed in many countries [1]. In thin steel plate shear walls without stiffeners, one of the reasons for reducing in energy dissipation capacity is shear buckling of steel plate under low shear force. Concrete coverage [2] or installation of stiffeners can be used for improving steel plate buckling behavior. To date, different theories for thin steel plate shear walls without stiffeners have been presented. Trueborn, Kulak, Montgomery and Timler, Kulak presented a strip model [3,4] for the analysis of thin steel plate shear walls. In this method, steel plate is divided in equal inclined strips. Then, steel plate strips are substituted with steel truss bars which are capable of carrying tension forces only. The analytic outcome of this method has been compared with experimental results by different researchers [5–8] and several methods like modification of stress-strain curves [6] and correction in angle of strips [8] have been recommended for modification of the strip model.

In addition, Berman and Bruneau developed equations for determining the shear strength of steel plate shear walls by using plastic analysis [9] of the strip model together with considering surrounding frame.

Plate-Frame interaction theory (PFI) was presented for predicting the linear and nonlinear behavior of different steel plate shear walls configurations including thin or thick steel plates, with or without stiffeners and opening by Roberts and Sabouri-Ghomi on 1991 [10]. In addition, this theory provides understanding of how the different components of the system interact and has capability to represent the system overall hysteretic characteristics properly [10–14].

The objective of this research is to study the effect of stiffeners on the behavior of steel plate shear walls and to verify the accuracy of PFI theory for these systems with or without stiffeners. For this purpose, two experimental specimens of steel plate shear walls with and without stiffeners and one experimental specimens of surrounding frame were tested.

2. Specimens descriptions of tests set up

Three experimental specimens were used for this research. Two one-third scale specimens were steel plate shear walls and another one was surrounding frame of them. They were designed base on PFI theory [10–14].

The steel plate shear wall specimen with stiffeners was called "DS-SPSW-0%" and consisted of steel plate, stiffeners and its surrounding frame (columns and top beam), as shown in Fig. 1. The other steel plate shear wall specimen was called "DS-PSW" which its specifications were similar to "DS-SPSW-0%" specimen, except it was without stiffeners, as shown in Fig. 2. The surrounding frame of the steel plate shear wall specimens (without steel plate) was tested and called "Frame" specimen and its specifications were similar to the surrounding frame of DS-SPSW-0% and DS-PSW specimens as shown in Figs. 1 and 2. The width, height and thickness of the steel plate in the specimens were 1410, 960 and 2 mm, respectively, as shown in Figs. 1 and 2.

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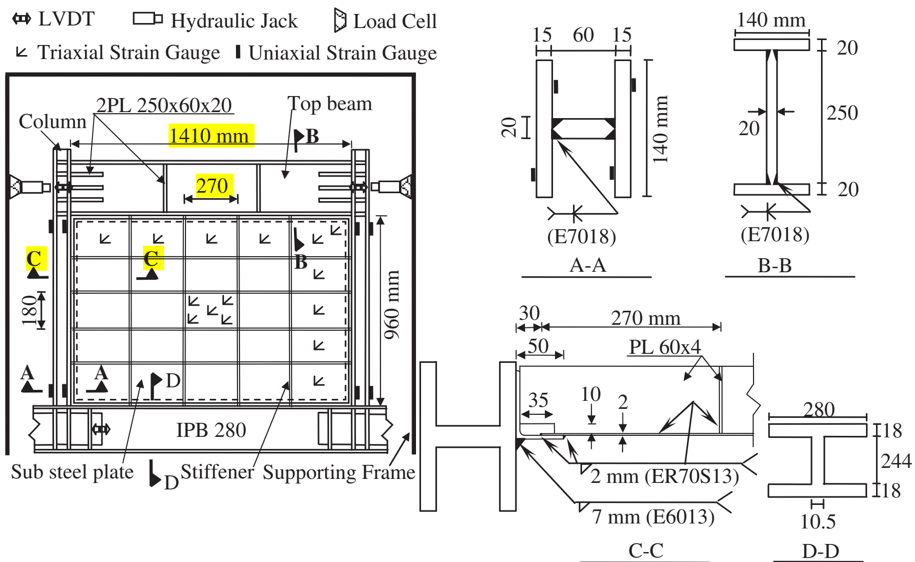

Fig. 1. Specifications of DS-SPSW-0% specimen.

In steel plate shear walls, steel plate is usually connected to surrounding frame with fish plates. In these specimens, angle of 60x60x6 mm was used for connecting the steel plate to the surrounding frame. For accurate welding between the steel plate and the angle, one side of the angle was milled. In DS-SPSW-0%, the width and thickness of the stiffeners were 60 mm and 4 mm, respectively. They installed on the one side of the steel plate and their both ends were welded to the angle as shown in Fig. 1. The stiffeners divided the steel plates to some sub steel plates as shown in Fig. 1. By using eigenvalue buckling finite element method, the size of the stiffeners so designed that to prevent the global shear buckling of the steel plate. For this purpose the moment inertia of the stiffeners increased gradually until finite element results showed that local shear buckling happened in the sub steel plates. By using equations for the shear buckling of plate [15], the dimensions of sub steel plates were so designed that to avoid local shear buckling of the sub steel plates before yielding.

In DS-PSW specimen, four small stiffeners were installed at four corners of the steel plate on the one side of it to avoid plate zipping effect which their specifications are shown in Fig. 2. The width and thickness of the columns flanges were 140 and 15 mm, respectively, and the width and thickness of the columns webs were 60 and 20 mm, respectively. The width and thickness of the top beam flanges were 140 mm and 20 mm, respectively, and its web width and thickness were 250 mm and 20 mm, respectively, as shown in Figs. 1 and 2. The section of beam and columns were built up by using steel plates, as shown in Fig. 1.

Material which was used for the steel plate was low strength steel and the material of the top beam and columns were of high strength steel. The yield and ultimate stresses of the steel materials were obtained by coupon tests and indicated in Table 1.

The specimens were connected to the laboratory strong floor with IPB 280 section profile (DIN Germany standard) and placed in a supporting frame, as shown in Fig. 1. Lateral shear loads were applied by two horizontal hydraulic jacks as shown in Fig. 1. Four horizontal LVDTs (Linear Variable Differential Transformer) and two load cells were located at the positions of jacks for measuring the story shear displacements. The average horizontal displacement of these LVDTs was used for measuring the story shear displacements of the specimens. Some triaxial and uniaxial strain gauges installed on the steel plate and columns. Their locations are shown in Figs. 1 and 2.

3. Tests descriptions

Lateral shear load was applied by two horizontal hydraulic jacks on the both sides of the top beam, according to ATC-24 [16] provision; the lateral shear loads and the story shear displacements were recorded.

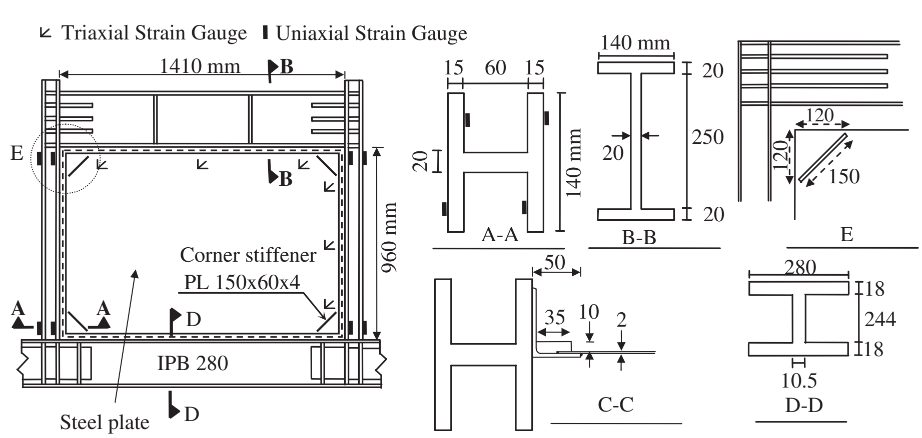

Fig. 2. Specifications of DS-PSW specimen.

Table 1
Results of tensile coupon tests.

	Yielding stress (MPa)	Ultimate stress (MPa)	Steel grade in DIN Germany standard
Plate	192.4	277.2	St14
Column and beam	414.9	551.8	St52
Stiffeners	258.3	390.4	St37

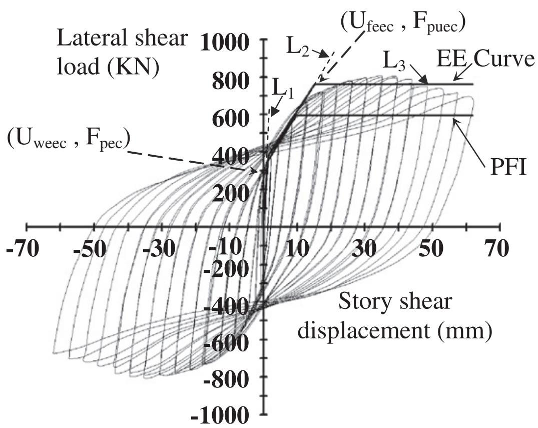

Fig. 3. Hysteresis, EE curve and PFI curves of DS-SPSW-0% specimen.

During the first five cycles of loading, the load was gradually increased and no significant yielding was observed in the specimens. In the sixth cycle of loading, installed strain gauges on the steel plates of DS-SPSW-0% and DS-PSW specimens and the column flanges in Frame specimen, showed significant yielding. Then, by considering this yield point, tests continued with displacement control according to ATC-24 [16] guidelines. In DS-SPSW-0% and Frame specimens, the story shear displacement steps were constant up to the end of the tests, but in DS-PSW specimen, the story shear displacement steps were constant up to the twenty second cycles and increased to twice for the next steps.

3.1. Test description of DS-SPSW-0% specimen

In the sixth cycle of the loading, the installed strain gauges on the steel plate showed that the first significant yielding happened at the story shear displacement of 1.58 mm (0.16% drift). The hysteresis curve of the specimen due to the cyclic loading is shown in Fig. 3. The maximum load carrying capacity of the specimen was 808 KN that happened at the story shear displacement of 34.05 mm (3.55% drift). The maximum drift of the specimen was 6.44%.

The first sub steel plate local buckling happened at the story shear displacement of 2.7 mm (0.28% drift), then by increasing the story shear displacement, post buckling field developed in all of them. In this specimen, the first steel plate tearing occurred as a very small tear in the middle of the one sub steel plates at 21.6 mm the story shear displacement (2.25% drift) and by increasing the story shear displacement, it was observed in the other sub plates. The steel plate tearing happened because of excessive curvature due to the post buckling waves. The excessive curvature of the waves caused reduction in the steel plate thickness on the crest of the waves and the tearing occurred during the cyclic loading. By increasing the story shear displacement, the tearing in the middle of the sub steel plates increased. No zipping happened between the steel plate and the surrounding frame in the specimen. The tearing gradually developed in the middle of the sub steel plates. The dimensions of the tears were small and the steel plate kept their continuity (as shown in Fig. 4). So they had no significant effect on the shear strength of the specimen. By increasing the story shear displacement, the middle tears of the sub steel plates developed. In the end of the test, the steel plate shear strength decreased when the sub steel plates almost lost their continuity. At this stage, no local or global buckling in the columns and no zipping between the steel plate and the surrounding frame happened. The situation of the specimen at the end of the test is shown in Fig. 5.

The strain gauges installed on the columns flanges showed that plastic hinges formed at the top and bottom of the columns between 8.6 mm to 12.9 mm the story shear displacement (0.89% to 1.34% drift), as shown in Fig. 6.

3.2. Test description of DS-PSW specimen

In the first five cycles of loading, the lateral shear load was gradually increased and almost no significant yielding happened. In the sixth cycle of loading, installed strain gauges on the steel plate showed that the first significant yielding happened at the story shear displacement of 1.7 mm (0.18% drift). The hysteresis curve of the specimen due to cyclic loading is shown in Fig. 7. The maximum load carrying capacity of the specimen was 789.6 KN that happened at the story shear displacement of 39 mm (4.06% drift). The maximum drift of the specimen was 5.34%. The steel plate buckled in the beginning of the loading.

The experimental results of two unstiffened thin steel plate shear wall specimens without the corner stiffeners, that were tested by Sabouri-Ghomi and Gholhaki [17,18], showed that destruction of the specimens was mostly because of sudden starting and developing tears as zipping at the steel plate corners edges. So in this research, in

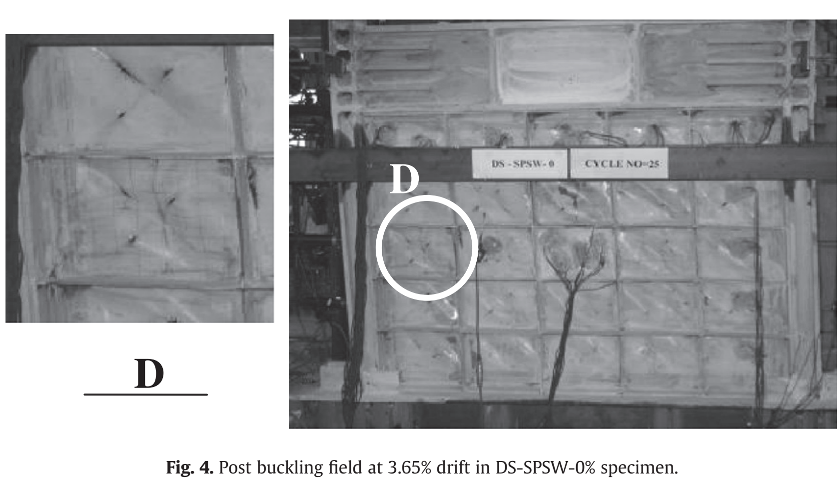

Fig. 4. Post buckling field at 3.65% drift in DS-SPSW-0% specimen.

Fig. 5. Deformation of DS-SPSW-0% specimen at 6.44% drift.

order to avoid zipping effect, four small stiffeners were installed on the corners of the specimen as shown in Fig. 2. The small tears started on the steel plate from one corner of the stiffeners at the story shear displacement of 9 mm (0.94% drift) and by increasing the lateral shear load, tearing developed firstly inward internal part of triangular steel plate surrounded by the stiffener and the surrounding frame. Then it very gradually developed toward external part of triangular steel plate at the steel plate edge as shown in Fig. 8.

Comparing the tears development in DS-PSW specimen with the corner stiffeners which was tested in this research, with specimens without it which were tested by Sabouri-Ghomi and Gholhaki [17,18] showed that using corner stiffeners could prevent sudden zipping between the steel plate and surrounding frame.

The end of the test and the destruction of the specimen was because of continues large tears between the steel plate and angle at the corner locations. Up to the end of the test, no global or local buckling observed in the columns. Deformation of the specimen at the end of the test is shown in Fig. 9. The strain gauges installed on the columns flanges showed that plastic hinges were formed in the top and bottom of the columns between 8.4 mm to 13 mm story shear displacement (0.88% to 1.35% drift), as shown in Fig. 6.

3.3. Test description of frame specimen

In the first five cycles of the loading, the lateral shear load was gradually increased and almost no significant yielding happened. During the sixth cycle of the loading, installed strain gauges on the columns flanges showed that the first significant yielding happened at the story shear displacement of 8.5 mm (0.89% drift). The hysteresis curve of the specimen due to the cyclic loading is shown in Fig. 10. The maximum load carrying capacity of the specimen was 405 KN that happened at the story shear displacement of 59.6 mm (6.21% drift). The maximum drift of the specimen was 6.68%. The strain gauges installed on the columns flanges showed that plastic hinges were formed at the top and bottom of the columns between 8.5 mm to 15.7 mm the story shear displacement (0.89% to 1.64% drift), as shown in Fig. 6.

The end of the test was because of the columns fracture and up to this stage, no local buckling was observed in the web and flange of the columns.

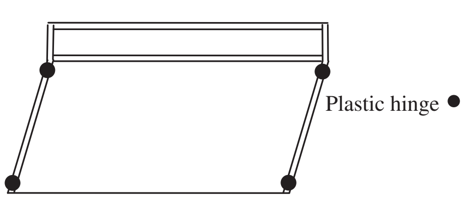

Fig. 6. Plastic hinges forming in the experimental specimens.

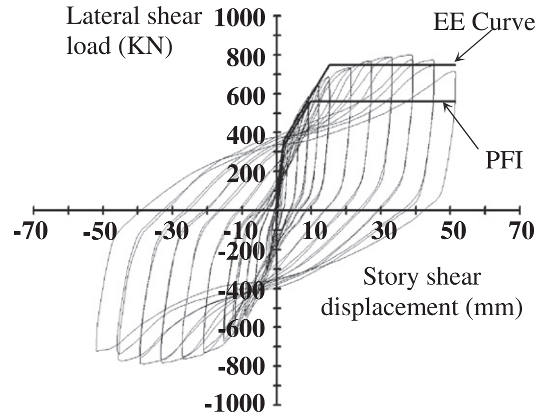

Fig. 7. Hysteresis, EE curve and PFI curves of DS-PSW specimen.

4. Equal energy curves (EE curves)

For determining theoretical parameters like the shear strength, stiffness and yield displacements of the specimens, equal energy curves (EE Curves) define by using of hysteresis curves envelopes.

EE Curves are obtained by equating the enclosed area under hysteresis curves envelopes and ideal curves. In the steel shear wall specimens, the results obtained from the strain gauges showed that, yielding occurred in two stages, the steel plate yielding and the plastic hinges forming in the columns. For this reason, ideal EE Curves are traced in three lines which represent these phenomena, as shown in Figs. 3 and 7.

For tracing ideal EE Curves, hysteresis curve envelope is traced firstly, then by using this curve and results of the strain gauges, the three L_1, L_2 and L_3 lines are traced as shown in Fig. 3. L_1 line is traced from the origin of coordinate system to the point where the strain gauges showed the steel plate yielding. L_2 line is traced between the steel plate yielding and the column flanges yielding limits. L3 line is so traced that area under envelope curve and EE Curve are equal.

By using three above mentioned lines, EE Curves are traced as shown in Figs. 3 and 7. In these figures, U_{weec} and F_{pec} are the story shear yield displacement of the steel plate and the related shear force of the specimen, respectively, U_{feec} and F_{puec} are the story shear displacement of forming plastic hinges in the columns and the shear strength of the steel plate shear wall, respectively, which are obtained from EE Curves.

In Frame specimen, since the specimen only yielded at the columns plastic hinges, based on the same method, ideal EE Curve is traced in the form of double lines by using L_1 and L_3 lines, as shown in Fig. 10. In this figure, F_{fuec} is the shear strength of Frame specimen.

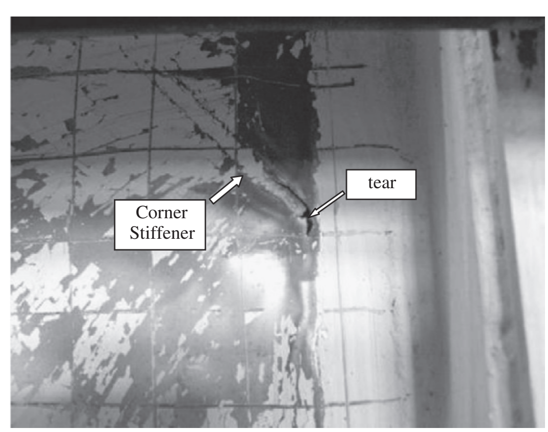

Fig. 8. Plate tearing from stiffener edge at 2.71% drift in DS-PSW specimen.

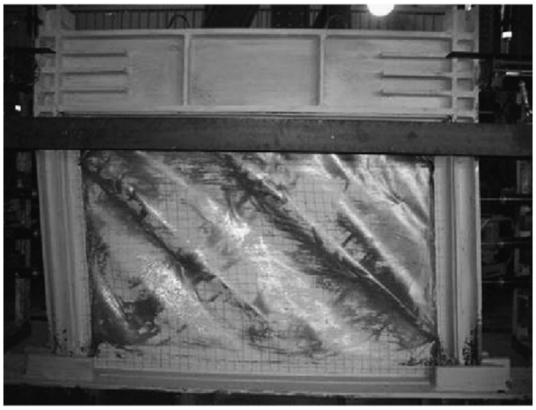

Fig. 9. Deformation of DS-PSW specimen at 5.34% drift.

5. Plate-Frame Interaction Theory (PFI)

In this theory [10–14], a typical isolated story of a multistory structure steel plate shear walls is represented as shown in Fig. 11. According to PFI theory, the shear load – displacement curve for the steel plate and for the surrounding frame is obtained separately. Then, by adding the two curves, the shear load – displacement curves of steel plate shear wall (the specimens) is obtained that is shown in Fig. 12.

In this figure, U_{we} and F_{wu} are the shear yield displacement and shear strength of the steel plate, respectively. is the shear displacement of forming plastic hinges in the columns. F_{fu} is the surrounding frame shear strength and F_{pu} is the shear strength of the steel plate shear wall and obtained from:

$$F_{pu} = F_{wu} + F_{fu} \tag{1}$$

By using Fig. 12, the shear stiffness of the steel plate shear wall (K_p) is obtained:

$$K_p = K_w + K_f \tag{2}$$

In Eq. (2), $K_w(=F_{wu}/U_{we})$ is the shear stiffness of the steel plate and $K_f(=F_{fu}/U_{fe})$ is the shear stiffness of the surrounding frame.

For DS-PSW specimen which is an unstiffened thin steel plate shear wall and the steel plate buckled at the beginning of the loading, PFI theory [10–14] equations are obtained as follows:

$$F_{wu} = 0.5bt\sigma_y \sin 2\theta \tag{3}$$

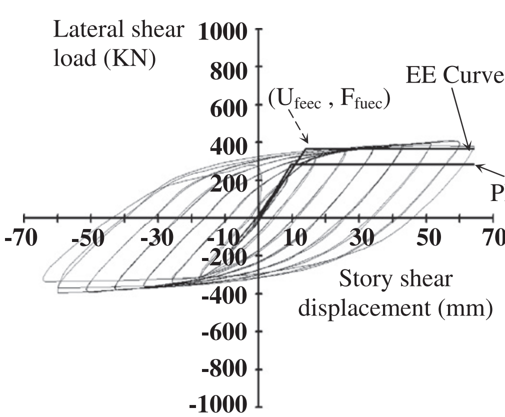

Fig. 10. Hysteresis, EE curve and PFI curves of frame specimen.

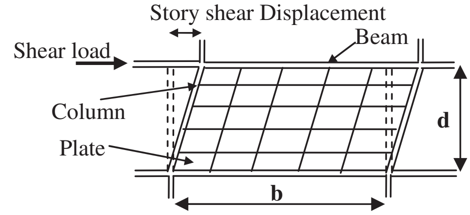

Fig. 11. Isolated steel plate shear wall story.

$$U_{we} = \frac{2\sigma_y}{E \sin 2\theta} d \tag{4}$$

$$K_w = \frac{0.25Ebt}{d} \sin^2 2\theta \tag{5}$$

In Eqs. (3) to (5), b is the steel plate width, d is the steel plate height as shown in Fig. 11, t is the steel plate thickness, E is the steel plate elasticity modulus, σ_y is the uniaxial yield stress of the steel plate and θ is the angel of tension field inclined respect to the horizontal line as shown in Fig. 13.

For DS-SPSW-0% specimen which is a stiffened steel plate shear wall and steel plate buckled after the steel plate yielding, PFI theory [10–14] equations are obtained as follows:

$$F_{wu} = \sigma_y bt/\sqrt{3} \tag{6}$$

$$U_{we} = \frac{\sigma_y}{\sqrt{3}G} d \tag{7}$$

$$K_w = Gbt/d \tag{8}$$

In these equations, $G\ (=E/2(1+\nu))$ is the shear elasticity modulus of the steel plate and $\nu(=0.3)$ is the steel plate Poisson's ratio.

In DS-PSW, DS-SPSW-0% and Frame specimens, Plastic hinges formed in the top and bottom of the columns as shown in Fig. 6, therefore PFI theory [10–14] equations are obtained as follows:

$$F_{fu} = 4M_{fp}/d \tag{9}$$

$$U_{fe} = \left(M_{fp}d^2\right)/\left(6E_f I_f\right) \tag{10}$$

$$K_f = 24E_f I_f/d^3 \tag{11}$$

In these equations, M_{fp} and I_f are the plastic moment and moment of inertia of the columns cross section, respectively, and E_f is the column elasticity modulus.

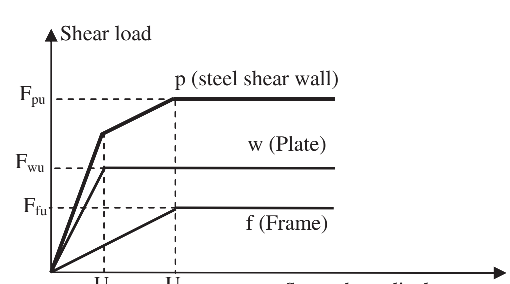

Fig. 12. Components of plate-frame interaction theory in steel plate shear wall.

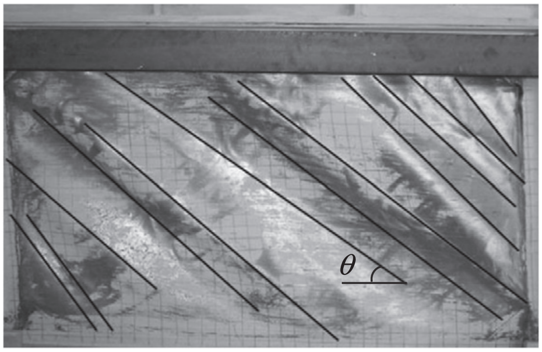

Fig. 13. Tension field orientations in DS-PSW specimen.

6. Determining the tension field angle (θ) in post-buckling field

The steel plate in DS-PSW specimen buckled in elastic region in the beginning of the loading and post buckling tension field and related waves formed as shown in Fig. 13. In DS-SPSW-0% specimen, the steel plate in the sub steel plates buckled after the steel plate yielding. The post buckling tension field of this specimen is shown in Fig. 14. The post buckling waves have θ angle respect to horizontal line which is shown in Figs. 13 and 14. By using Fig. 13 in DS-PSW specimen, the angel values of post-buckling waves are between 40 to 55° which average of them is approximately, 48°. By using Fig. 14 in DS-SPSW-0% specimen, the angel values of post-buckling waves are 35 to 54° which average of them is approximately, 42°.

In PFI theory, according to the Eqs. (6) to (8) the values of F_{wu}, U_{we} and K_w are independent from θ angel for DS-SPSW-0% specimen. In DS-PSW specimen, the values of F_{wu}, U_{we} and K_w (Eqs. (3) to (5)) are dependent on the θ angel. Comparing between the values of Eqs. (3) to (5) that obtained by using the upper and lower values of obtained θ from Fig. 13 relative to using of angel $\theta = 45°$ are negligible. This is also recorded before by Robert and Sabouri-Ghomi [10], Sabouri-Ghomi [12] and Sabouri-Ghomi, Ventura et al. [14].

7. Comparison of experimental results with PFI Theory

Results obtained from Eqs. (1) and (2) for DS-PSW and DS-SPSW-0% specimens are shown in Figs. 3 and 7, which represent the total behavior (plate and frame) of the specimens. In addition, results are obtained from Eqs. (9) to (11) for Frame specimen is shown in Fig. 10. As it can be seen in these figures, there are good agreements between the experimental and theoretical results. The shear stiffness of the steel plate shear walls by using EE Curves (K_{pec}) is obtained as follows:

$$K_{pec} = F_{pec}/U_{weec} \tag{12}$$

Fig. 14. Tension field orientations in DS-SPSW-0% specimen.

Table 2
Comparison of the PFI Theory shear stiffness respect to the EE curve shear stiffness for DS-PSW and DS-SPSW-0% specimens.

	DS-PSW	DS-SPSW-0%
K_p (KN/m)	181298.8	260385.4
K_{pec} (KN/m)	175762.4	252228.9
PFI respect to EE curve	3.1%	3.2%

The shear stiffness of Frame specimen by using EE Curves (K_{fec}) is obtained as follows:

$$K_{fec} = F_{fuec}/U_{feec} \tag{13}$$

The differences of the shear stiffness of DS-PSW and DS-SPSW-0% specimens which are obtained from PFI theory (K_p) and EE Curves (K_{pec}) are given in Table 2. According to this table, the differences of them are negligible.

The differences of the shear strength of DS-PSW and DS-SPSW-0% which are obtained from PFI theory (F_{pu}) and EE Curves (F_{puec}) are given in Table 3. According to this table, the shear strengths of PFI theory are less than 25.3% and 22.1% values respect to the shear strengths of EE Curves, for DS-PSW and DS-SPSW-0%, respectively, which are in favor of safety. The differences are because of the effect of material strain hardening which is not considered in PFI theory but can be seen in EE Curves. In addition in PFI theory the behavior of steel plate and frame are assumed elastic-perfectly plastic (Table 3).

8. Effect of stiffeners on the behavior of the steel plate

For determining the shear strength and shear stiffness of the steel plate, both the EE Curve of Frame specimen and the EE Curves of DS-PSW and DS-SPSW-0% specimens, are used. By using Fig. 12, the following equations are obtainable:

$$K_{wec} = K_{pec} - K_{fec} \tag{14}$$

$$F_{wuec} = K_{wec} \cdot U_{weec} \tag{15}$$

In the above equations, K_{wec} and F_{wuec} are the shear stiffness and shear strength of the steel plate, respectively, which are obtained from EE Curves.

The difference in the shear strength, shear stiffness and story shear yield displacements of the steel plate obtained from the tests and the theory for DS-SPSW-0% specimen comparing with DS-PSW specimen are given in Tables 4 and 5.

According to this table, installations of stiffeners have little effect on the shear strength of the steel plate (F_{wuec}), but they have significant effect on the steel plate shear yield displacement and shear stiffness (U_{weec} and K_{wec}). According to Table 4, the experimental results of DS-SPSW-0% and DS-PSW specimens showed that, the installation of stiffeners on the steel plate increased the steel plate shear stiffness (K_{wec}) by 51.1% and decreased the steel plate story shear yield displacement (U_{weec}) by −25.4%, which are in good agreement with PFI theory (Table 5).

Table 3
Comparison of the PFI theory shear strength respect to the EE curve shear strength for DS-PSW and DS-SPSW-0% specimens.

	DS-PSW	DS-SPSW-0%
F_{pu} (KN)	574.68	616.65
F_{puec} (KN)	769.54	791.54
PFI respect to EE curve	−25.3%	−22.1%

Table 4
Comparison of steel plate results for DS-SPSW-0% specimen respect to DS-PSW specimen which obtained from the EE curve.

	K_{wec}	F_{wuec}	U_{weec}
EE curve	51.1%	12.8%	−25.4%

9. Effect of component of PFI Theory on the behavior of the steel plate shear wall

According to PFI theory, the ratio of shear strength and stiffness of the steel plate and surrounding frame to steel plate shear walls are given in Table 6. As could be seen in this table, the ratio of shear strength of steel plate to steel plate shear wall is 0.47 to 0.51. These ratios mean that both of steel plate and surrounding frame almost had the same contribution in the shear strength of steel plate shear wall. According to Table 6, the ratio of shear stiffness of steel plate to steel plate shear wall is 0.81 to 0.87 which means that the steel plate was principal member in shear stiffness and surrounding frame had a minor effect.

10. Energy dissipation capacity

Energy dissipation Capacity corresponding to the story shear displacement is defined the area under cyclic curve as shown in Fig. 15. The energy dissipation curves of the experimental specimens are shown in Fig. 16. According to this figure, the maximum energy dissipation capacity of DS-SPSW-0% specimen comparing with DS-PSW specimen increased 26%. According to Fig. 16, the maximum energy dissipation capacity of Frame specimen are 50% and 67% less than DS-SPSW-0% and DS-PSW specimens, respectively. As it can be seen in Fig. 3 comparing with Fig. 7, the installation of stiffeners on thin steel plate cased the shape of hysteresis curve changed from “S” shape to “Spindle” shape in the stiffened steel plate that caused increase in the energy dissipation capacity of system.

According to Eqs. (4), (7) and (10), by using of low strength steel in the steel plate and high strength steel in the beam and columns, the difference between U_{we} and U_{fe} increases so according to Fig. 13, it could be expected that the steel plate becomes non-linear in smaller shear displacement and absorbs more energy. In this situation, the columns as main vertical load bearing members, commonly remains safer.

11. Conclusion

In this research, three experimental specimens were used for considering experimentally and theoretically of the effect of the stiffeners installation on the behavior of thin steel plate shear walls.

The results of the tests showed that the installation of stiffeners on the steel plate had minor effect on the shear strength of the steel plate but its effect on the shear stiffness and the shear yield displacement of the steel plate was significant. The results showed that the installation of stiffeners decreased the shear yield displacement of the stiffened thin steel plate by 25.4% and increased the shear stiffness of the steel plate by 51.1% comparing with the unstiffened steel plate. In addition, it increased the value of the steel plate shear wall energy dissipation capacity by 26%.

Comparing the EE Curves (experimental results) with the PFI theory shear load-displacement curves showed that there are good agreement between them for both the stiffened and the unstiffened specimens. In the stiffened steel plate relative to the unstiffened steel plate, PFI theory also predicts the increase in the shear strength and the shear stiffness of the steel plate and decrease in the steel plate shear yield displacement with very close values to the experimental results. This agreement also shows the ability of PFI theory for providing a good understanding of how the different components of the system interact on the behavior of it.

Table 5
Comparison of steel plate results for DS-SPSW-0% specimen respect to DS-PSW specimen which obtained from PFI theory.

	K_w	F_{wu}	U_{we}
PFI	49.8%	13.4%	−24.3%

Table 6
Comparison the shear strength and stiffness of steel plate and surrounding frame respect to steel plate shear wall in PFI Theory.

	F_{wu}/F_{pu}	F_{fu}/F_{pu}	K_w/K_p	K_f/K_p
DS-PSW	0.47	0.53	0.81	0.19
DS-SPSW-0%	0.51	0.49	0.87	0.13

12. Notation

The following symbols are used in this paper:

b	steel plate width;
d	steel plate height;
E	steel plate elasticity modulus;
E_f	column elasticity modulus;
F_{fu}	frame shear strength which obtained from PFI theory;
F_{fuec}	frame shear strength which obtained from EE Curve;
F_{pec}	shear force at the steel plate yielding which obtained from EE Curve;
F_{pu}	shear strength of steel plate shear wall which obtained from PFI theory;
F_{puec}	shear strength of steel plate shear wall which obtained from EE Curve;
F_{wu}	shear strength of steel plate which obtained from PFI theory;
F_{wuec}	shear strength of steel plate which obtained from EE Curve;
G	steel plate shear elasticity modulus;

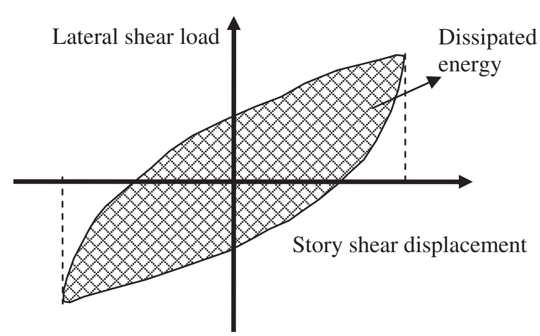

Fig. 15. Energy dissipation capacity corresponding hysteresis curve.

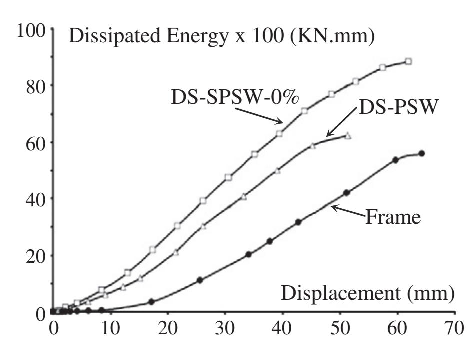

Fig. 16. Energy dissipation capacity in the specimens.

I_f column moment of inertia;
K_f shear stiffness of surrounding frame which obtained from PFI theory;
K_{fec} shear stiffness of surrounding frame which obtained from EE Curve;
K_p shear stiffness of steel plate shear wall which obtained from PFI theory;
K_{pec} shear stiffness of steel plate shear wall which obtained from EE Curve;
K_w shear stiffness of steel plate which obtained from PFI theory;
K_{wec} shear stiffness of steel plate which obtained from EE Curve;
M_{fp} column plastic moment;
t steel plate thickness;
U_{fe} shear displacement of forming plastic hinges in columns which obtained from PFI theory;
U_{feec} story shear displacement of forming plastic hinges in columns which obtained from EE Curve;
U_{we} shear yield displacement of steel plate which obtained from PFI theory;
U_{weec} story shear yield displacement of steel plate which obtained from EE Curve;
θ inclination of tension field;
ν steel plate Poisson`s ratio;
σ_y steel plate uniaxial yield stress;

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