Characteristics of Steel Plate Shear Walls With Infill Plates Strengthened by GFRP Laminates: Experimental Investigation

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SUMMARY:
In composite steel plate shear walls system, steel web plates can be strengthened by adding a number of layers of fiber reinforced polymer laminate or concrete on one or both sides of the web plate. In this paper, nonlinear behavior of composite steel plate shear wall systems, in which steel infill plate is strengthened by fiber reinforced polymer (FRP) layers, are experimentally investigated. Experimental models are scaled one-story steel shear panel model, with hinge type connections of boundary elements at four corners. In the first test, unstiffened steel infill plate is used for test. In the next four tests, strengthened steel infill plates are being used with different number and orientation of GFRP layers. Each test was performed under fully reversed cyclic quasi-static loading in the elastic and inelastic response zones of the specimens, in compliance with ATC-24 (1992) test protocol. The experimental results indicate that by strengthening infill steel plate yield strength, ultimate shear strength and cumulative dissipated energy can be significantly increased.

Keywords: Steel plate shear wall; Composite; GFRP; Fiber orientation, Stiffness

1. INTRODUCTION

Steel plate shear wall (SPSW) systems have significant advantage over many other systems in term of cost, primarily, substantial ductility, high initial stiffness, fast pace of construction, and the reduction in seismic mass. SPSW system can be used in different configurations, such as stiffened, un-stiffened thin steel plate, and composite steel plate. Unstiffened steel plate shear wall is the basis for SPSW systems. This type of web plate has negligible compression strength and shear buckling occurs at low levels of loading. Lateral load are resisted through diagonal tension in the web plate. Stiffened web may also be used to increase shear buckling strength. In this type, the strength is a combination of shear buckling strength and additional strength from diagonal tension action. In composite steel plate shear walls (CSPSWs) system, steel web plates can be stiffened by adding concrete on one or both sides of the web plate. Concrete layers can improve load carrying capacity of SPSWs by permitting utilization of the full yield strength of the infill plate. In addition, shear strength of the concrete is effective to increase capacity of system. Steel infill plate can be strengthened by adding number of layers of fiber reinforced polymer laminate in both sides. In this type of CSPSW, like unstiffened SPSW systems, strengthened steel plate has negligible compression strength and shear buckling occurs at low levels of loading. FRP laminate layers are effective to increase post buckling strength, initial and secant stiffness of the system.

During the four last decades many experimental and numerical research on seismic performance of un-stiffened and stiffened SPSW have been carried out and these researches lead to better understanding of this lateral load resistant system. Wagner is the first researcher who used a complete and uniform tension fields to determine the shear strength of a panel with rigid flanges and very thin web, and inferred that the shear buckling of a thin aluminum plate supported adequately on its edges does not constitute failure. Other researches were also conducted based on this idea to develop an analytical method for modeling of thin SPSW. Thorburn et al. developed a simple analytical method to evaluate the shear strength of unstiffened SPSWs with thin steel plates and introduced the strip model to represent the tension field action of a thin steel wall subjected to shear forces. Timler and Kulak modified the formula for the angle of strips inclination with the column by the tests. Elaghy experimentally investigated behavior of SPSW and proposed an analytical model to determine the behavior of thin steel plate shear walls. Berman and Bruneau presented plastic analysis method based on the strip models as an alternative for the design of SPSW. This method has been implemented into the Canadian design codes for steel structure (CAN/CSA 2001) and the AISC (2005b) seismic design specifications. Sabouri-Gohomi et al. proposed Plate-Frame-Interaction (PFI) method to predict the shear behavior of the SPSWs. Kharrazi et al. presented modified plate frame interaction (M-PFI) method for use in the design of steel plate wall systems. Khazaei-Poul and Nategh-Alahi proposed an analytical model, the Composite-Plate Frame Interaction (C-PFI) method, to predict the shear behavior of the strengthened steel plate shear walls by FRP Laminate, and they showed C-PFI method is able to properly predict the shear behavior of the C-SPSW systems. They showed fiber orientation is an important variable on the seismic behavior of the C-SPSW. They reported if principal orientation of GFRP laminates is oriented in the direction of tension fields, the shear strength and stiffness of C-SPSW will reach the maximum possible value

Astaneh-Asl performed experimental tests on the two specimens of three-story C-SPSW under cyclic loads. They showed the concrete layer produces a better distribution of stress in the steel plate and developing tension field lines in a wider region. Rahai and Hatami experimentally and numerically investigated the effects of shear studs spacing variation, middle beam rigidity and the method of beam to column connection on the C-SPSW behavior.

Lubell et al. tested two single and one 4-story thin SPSWS under cyclic loading and compared the experimental results with the simplified tension field analytical models and found that the models can predict post-yield strength of the specimens well, with less satisfactory in the elastic stiffness results. Caccese et al. tested five one-fourth scale models of three-story into the effects of panel slender ratio and type of beam-to-column connection. They reported as the plate thickness increased, the failure mode was governed by column instability and the difference between simple and moment-resisting beam-to- column connection was small. Driver et al. tested a 4-story large-scale steel plate shear wall specimen with unstiffened panels under cyclic loading to determine its behavior under an idealized severe earthquake event. Robert and Sabouri-Gohomi conducted a series of 16 quasi-static loading tests on unstiffened steel plate shear panel with central opening. Vian et al. performed test on special perforated SPSW with reduced beam section anchor beams under cyclic loading and reported the perforated panel reduced the elastic stiffness and overall strength of the specimen by 15% as compared with the solid panel specimen.

Alinia and Dastfan studied the effect of surrounding members on the overall behavior of thin steel plate shear walls. His results show that the flexural stiffness of the surrounding members has no significant effect on elastic shear buckling or the post-buckling behavior of the shear walls.

In this paper, nonlinear behaviour of composite steel plate shear walls has been experimentally investigated. In this regard, five experiments have been conducted. Experimental models are scaled one-story steel shear panel models, with hinge type connections of boundary elements at four corners. Steel infill plates are strengthened by number of GFRP laminate layers with different orientation and all specimens are subjected to quasi-static loading.

2. SHEAR LOAD–DISPLACEMENT DIAGRAM OF C-SPSW

In SPSWs, steel infill plate can be strengthened by number of FRP laminate layers with different orientations, (see Figure 1). In Figure 1, (X-Y) coordinate system is general coordinate system and (1, 2) is the principal material coordinates of a typical FRP laminate layer. (X-Y) coordinate system in the Figure 1 dose not coincides with the principal material coordinate system. The 1-axis is oriented at an angle of +γ counterclockwise from the X-axis (see Figure 1). In (X′- Y′) coordinate system, X′-axis is oriented along the direction of tension field and at an angle of +θ counterclockwise from the X-axis. FRP layers can be attached to steel infill plate by adhesive. If a perfect bond is considered at the bond between adhesive and steel infill plate interface and between FRP layers and also it is assumed that delamination does not occur in the FRP layers, shear load–displacement diagram of composite steel plate shear wall can be approximately obtained by superimposing the shear load–displacement diagrams of frame and composite steel plate as shown in Figure 2. In C-SPSW after yielding of the steel infill plate, system will have a secant stiffness that it is provided by FRP laminate layers.

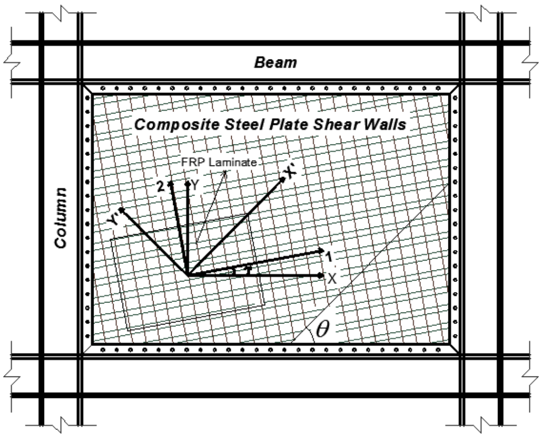


Figure 1. Composite steel plate shear wall

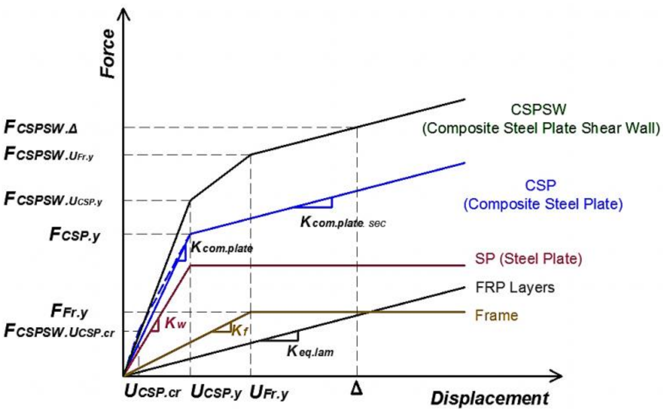


Figure 2. Shear load–displacement diagram of C-SPSW (Khazaei-Poul and Nateghi-Alahi 2012)

Notations:

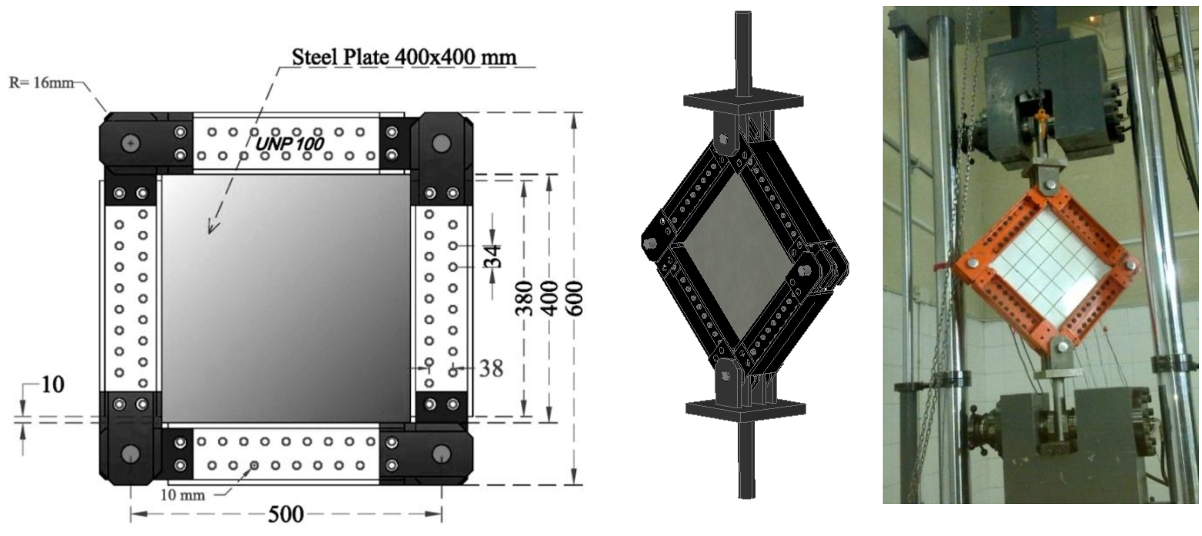
$U_{CSP.cr}$	Limiting elastic displacement of composite steel plate
$U_{CSP.cr}$	The limiting elastic shear displacement
U_{wpb}	Shear displacement of the post-buckled component
U_{Fr_y}	The limiting elastic shear displacement of the frame
△	Displacements larger yielding point of the frame and composite steel plate
$F_{CSPSW.U_{CSP.cr}}$	Total shear force of the CSPSWs at $U_{CSP.cr}$ (correspond to the buckling limit)
$F_{CSPSW.U_{CSP.y}}$	Total shear force of the CSPSWs at $U_{CSP.y}$ (corresponds to the yielding point of the steel plate in composite steel plate)
$F_{CSPSW.U_{Fr.y}}$	Total shear force of the CSPSWs at $U_{fr.y}$ (corresponds to the yielding point of the frame)
$F_{CSPSW.△}$	Total shear force of the CSPSWs at △ (corresponds to displacements larger yielding point of the frame and composite steel plate
F_{CSP_y}	Limiting elastic shear force of composite steel plate
$F_{Fr.y}$	The shear strength of the frame
F_{wpb}	Shear strength of the composite steel plate due to formation of tension field lines
$K_{comp.plate.sec}$	Equivalent secant stiffness of the composite steel plate
K_{fr}	Stiffness of frame
$K_{comp.plate}$	Stiffness of the composite steel plate
K_w	Stiffness of steel plate
$K_{eq.lam}$	Equivalent stiffness of FRP laminate layers

3. EXPERIMENTAL STUDY

To investigate and to evaluate the effect of glass fiber polymer on the seismic behaviour of steel plate shear wall, several tests were performed on scaled models of steel plate shear panel. For this purpose, five tests have been conducted. Experimental models are scaled one-story steel shear panel models, with hinge type connections of boundary elements at four corners. In the first test unstiffened steel infill plate are used for test. In the next tests, steel infill plates are strengthened by numbers of GFRP layers with different orientations. In this study, infill plate is strengthened by four methods of arranging the FRP laminate on the infill steel plate. Each test was performed under fully reversed cyclic quasi-static loading in the elastic and inelastic response zones of the specimens, in compliance with ATC-24 (1992) test protocol by means of a hydraulic jack with 600 kN capacity.

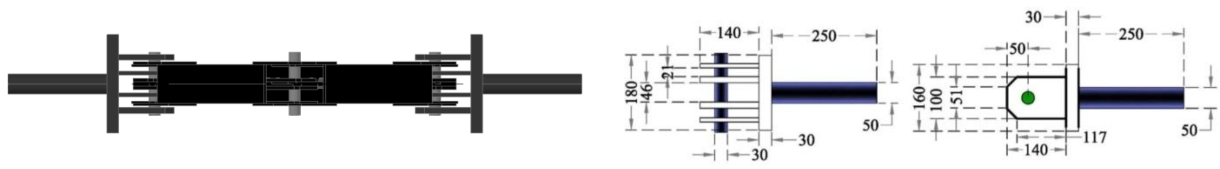
3.1. Test Set-Up and Experimental Models

For experimental study, the number of cyclic loading tests on small-scale, unstiffened steel plate shear panel (SPSP) and composite steel plate shear panel (CSPSP) were conducted. A scaled one-story steel shear panel model, with hinge type connections of boundary elements at four corners, is selected. Details of the test experimental specimens are presented in Figure 3. The edges of the steel and composite plate were clamped between pairs of rigid frame members by means of two rows of high tensile bolts. Bolts with a diameter of 10 mm are used for connections of infill steel plate to surrounding frame. The boundary elements of the specimen are similar, while the infill steel plate thickness is 0.9 mm. Specimen consisted of the standard profile double section UNP100, as boundary elements. The boundary elements were also such designed to meet the preliminary requirements of steel plate shear walls and AISC 341-05 provisions. The all specimen's depth and width are equal to 600 mm while depth and width of infill plates are equal to 400 mm.



a: Dimension of the experimental model

b: View of test set-up



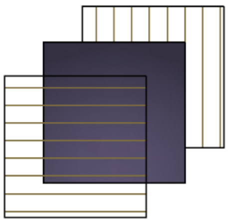
c: View of specimen and support details

Figure 3. Test set-up and specimen detail (dimension in mm)

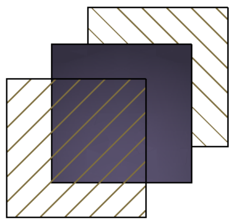
Details of the all experimental models are presented in Table 1. In the first experimental specimen (SPSP1) unstiffened steel plate with thickness of 0.9 mm is selected for infill plate. In the next models steel infill plate has been stiffened by numbers of GFRP layers with different orientation of GFRP laminated layers that are showed in Figure 4. In the CSPSP2 and CSPSP3 specimen, composite steel plate with approximately total thickness of 1.916 mm is selected for infill plate. Composite steel plate in the CSPSP2 and CSPSP3 specimens are consisted of steel infill plate whit thickness of 0.9 mm that is strengthened by one layer of GFRP laminate in each side. In the CSPSP2 specimen principal orientation of GFRP laminate (the direction that laminate have maximum amount of strength and young modules) are oriented a +45 and -45 inclination with angle with tension fields (α= 0 and 90) Figure 4-a . In the CSPSP3 specimen principal orientation of GFRP laminate are oriented in direction of tension fields (α= 0 and 90) Figure 4-b. In the CSPSP4 and CSPSP5 specimen, composite steel plate with approximately total thickness of 2.932 mm is selected for infill plate. Composite steel plate in the CSPSP4 and CSPSP5 specimens are consisted of steel infill plate whit thickness of 0.9 mm that is strengthened by two layer of GFRP laminate in each side. In the CSPSP4 specimen principal orientation of GFRP laminate are oriented a +45 and -45 inclination with angle with tension fields (α= 0 and 90) Figure 4-c. In the CSPSP5 specimen principal orientation of GFRP laminate are oriented in direction of tension fields (α= 0 and 90) Figure 4-d.

Table 1. Details of the Experimental Models

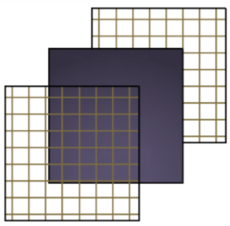
Experimental models	Number of layers in composite infill plate (mm)		Thickness of laminate and steel plate (mm)		total thickness of infill plate (mm)	Orientation of GFRP	GFRP Type
	Steel plate	GFRP layer	Steel plate	GFRP layer			
SPSP1	1	0	0.9 mm	-	0.9	-	-
CSPSP2	1	2	0.9 mm	0.508	1.916	0 # 90	SikaWrap® Hex 430G
CSPSP3	1	2	0.9 mm	0.508	1.916	45 # -45	SikaWrap® Hex 430G
CSPSP4	1	4	0.9 mm	0.508	2.932	0 & 90 # 0 & 90	SikaWrap® Hex 430G
CSPSP5	1	4	0.9 mm	0.508	2.932	45 & -45 # 45 & -45	SikaWrap® Hex 430G



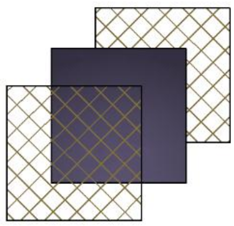
a: CSPSP2: one layer GFRP laminate in each side of infill plate; α= 0 & 90



b: CSPSP3: one layer GFRP laminate in each side of infill plate ; α= +45 & -45



c: CSPSP4: two layers GFRP laminate in each side of infill plate ; α= 0 & 90



d: CSPSP5: two layers GFRP laminate in each side of infill plate ; α= +45 & -45

Figure 4. Different types of strengthening of the infill steel plate by GFRP laminates

The SikaWrap-Hex-430G is applied to strengthen steel plates in all experimental specimens. SikaWrap-Hex-430G is a unidirectional glass fiber fabric. The structural adhesive Sikadur-330 was used to bond the composite overlays. The method of application using a grooved roller ensured that the adhesive dispersed uniformly through the fibers. Consequently, a thin layer of epoxy adhesive was formed at the steel and GFRP laminate interface and they worked together with the epoxy in-between the fiber laminates. Total thickness of each GFRP laminate used adhesive is 0.508 mm based on technical data provided by the manufacturer.

3.2. Material Properties

Tests were conducted to determine the stress- strain relationship of the infill steel plate and boundary element materials used in the experimental models. All tests were conducted on specimen dimensioned as proposed in ASTMA370-05. Yield stress and young module of infill steel plate based on the mean of static tests are equal to 180 MPa and 204 GPa, respectively and for boundary element (UNP100) those are equal to 310 MPa and 203 GPa, respectively. The failure strains were approximately 32% for steel infill plate and 19% for UNP100.

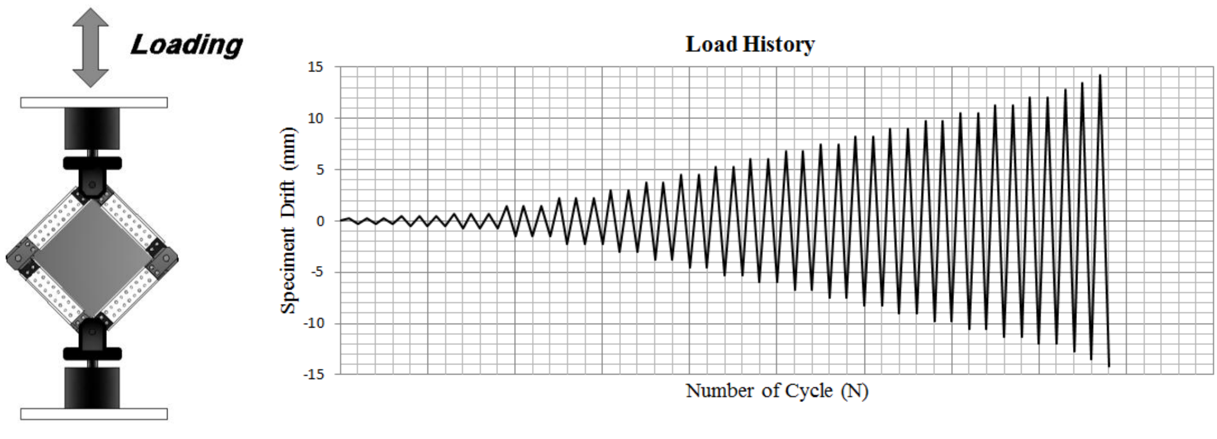
Steel infill plates were strengthened by SikaWrap-Hex-430G. Unidirectional GFRP laminate have almost linear behaviour the final failure. Based on the technical data provided by the manufacturer, the longitudinal modulus (Ex) and the longitudinal tensile strength (Tx) of the cured SikaWrap-Hex-430G Laminate with Sikadur 330 Epoxy, which is dominated by the properties of the fiber, is equal to 26.49 GPa and 537 MPa, respectively. The transverse modulus (Ey) and the transverse tensile strength (Ty) perpendicular are equal to 7.07 GPa and 23 MPa. The longitudinal and transverse failure strains of the GFRP laminate approximately are equal to 2.2% and 0.46%, respectively. Mechanical property of GFRP laminate is presented in Table 2. The Young’s moduli and Tensile Strength of Sikadur-330 are 3489 MPa and 60.6 MPa, respectively.

Table 2. Mechanical Property of The GFRP Laminate

GFRP Material	Tensile Modulus		Tensile Strength	
	Ex (GPa)	Ey (GPa)	Tx (MPa)	Ty(MPa)
SikaWrap® Hex 430G	6.49	7.07	537	23

3.3. Loading

The specimens are subjected to quasi-static cyclic loading along the diagonal axes of specimens, which began with very small values of overall drift and increased gradually until failure of the specimen, in compliance with ATC-24 (1992) test protocol. In Figure 5-a load direction is shown on the specimen. Forces are applied by using a Schenk 600 kN, servo hydraulic, dynamic testing machine. A similar load application system was used for all experimental models. In Figure 5-b load pattern that applied on the all specimen are provided. The ultimate displacement limit is considered to occur at a drift angle of 2.2% per ASCE 7-05.

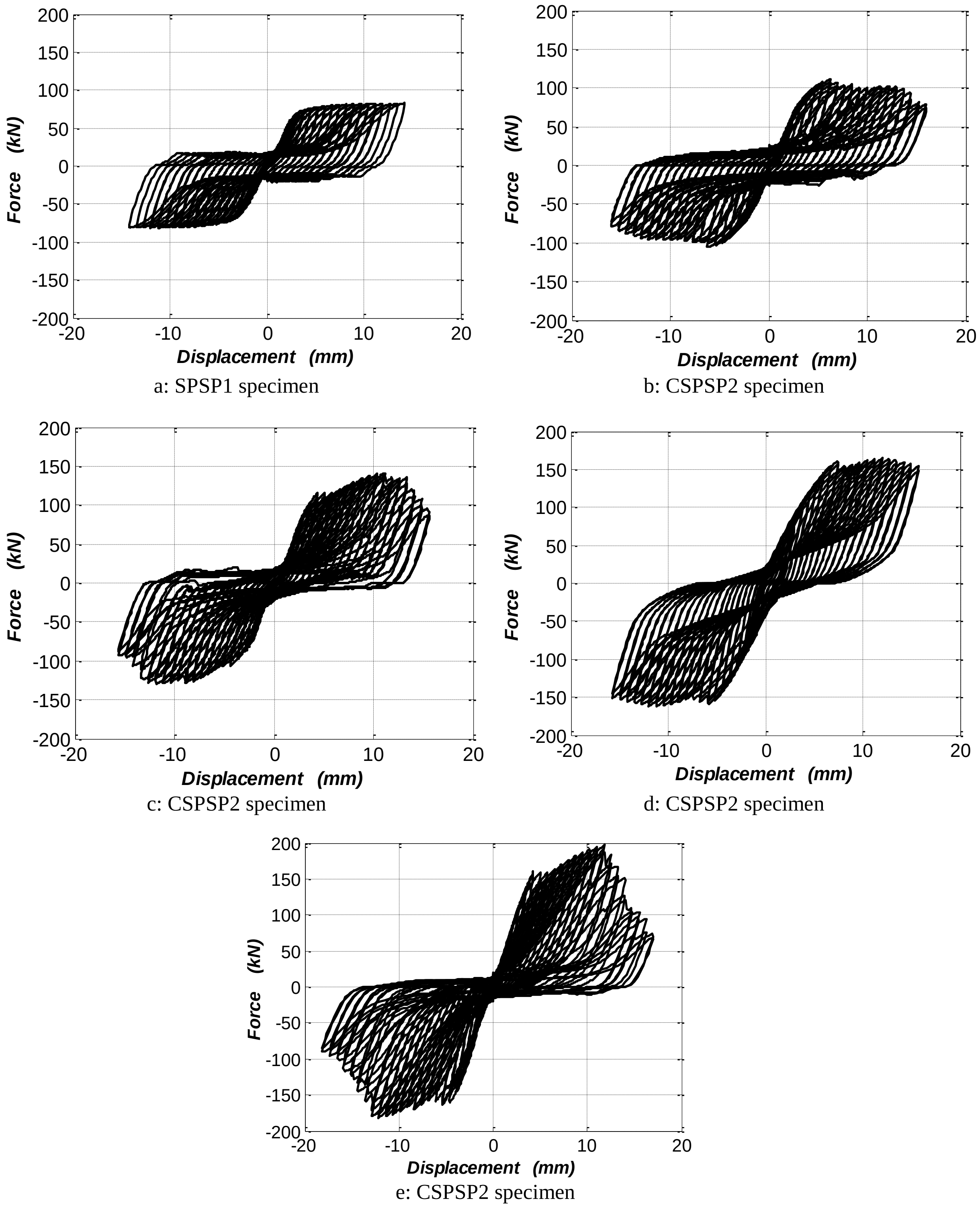


a: Load direction b: Load pattern based on ATC-24 (1992)

Figure 5. Load pattern and load direction of the experimental specimen

3.4. Discussion of Analytical Results

Hysteretic load-displacement curves for the specimens are presented in Figure 6. In Table 3, the amount of stiffness, yield strength, and ultimate strength of the specimens based on experimental hysteresis curve are presented. These results show that adding GFRP layers can be increased initial stiffness of specimens.



a: SPSP1 specimen

b: CSPSP2 specimen

c: CSPSP2 specimen

d: CSPSP2 specimen

e: CSPSP2 specimen

Figure 6. Hysteretic curve of the experimental specimens

Stiffness and strength are the major characteristics of the lateral resistant systems to resist earthquake loads. In Table 3, stiffness, yield strength, and ultimate strength of the all specimens based on experimental hysteresis curves are presented. In this study, component stiffness is calculated based on the secant stiffness to yield level forces. Yield strength is effective yield strength and is calculated based on FEMA 356 standard. These results show that by strengthening steel infill plate with the GFRP laminate layers, initial stiffness of specimen were increased up to 59%. In the CSPSP2 and CSPSP3 specimens that steel infill plates are strengthened by two layer of GFRP, initial stiffness have been increased 10% and 39%, respectively. In the CSPSP4 and CSPSP5 specimens that steel infill plates are strengthened by four layer of GFRP, initial stiffness have been increased 34% and 59%, respectively. These results show that orientation of GFRP laminates have a significant effect on the stiffness of the specimen. Accordingly, if principal orientation of GFRP laminates is oriented in the direction of tension fields, the stiffness in specimens reaches the maximum possible value. In addition, Changes in the direction of the GFRP laminates can change up to 25% in the stiffness of the specimen.

Yield and ultimate strength of the all specimen are shown in Table 3. Results show both yield and ultimate strength in the all strengthened specimens are significantly increased. Ultimate strength in the CSPSP2, CSPSP3, CSPSP4, and CSPSP5 specimens have been increased 24%, 71%, 101%, and 120%, respectively. Therefore, fiber orientation is an important variable in the ultimate strength of the C-SPSW. The structural capacity of FRP laminate can be tailored and maximized by aligning fibers along the optimal orientation. For C-SPSW, it is well established that fibers should be aligned along the direction of tension field. In this state, maximum stiffness and strength are provided by FRP laminate.

Table 3. Comparison of The Stiffness and Strength of Experimental Models

Specimen	Stiffness (K)	K.rel	Yield strength (Fy)	Fy.rel	Ultimate strength (Fu)	Fu.rel
	kN/m	(kN/m)/(kN/m)	kN	kN/kN	kN	kN/kN
SPSP1	26155.0	1.00	72.88	1	82.58	1
CSPSP2	28666.7	1.10	100.5	1.38	102.3	1.24
CSPSP3	36325.3	1.39	115	1.58	140.8	1.71
CSPSP4	34981.7	1.34	160.7	2.20	165.6	2.01
CSPSP5	41652.4	1.59	160	2.20	182	2.20

Cumulative hysteretic dissipated energy is the summation of dissipated energy experienced by the specimen during the test. In Figure 7, the comparison in terms of cumulative (hysteretic) dissipated energy versus drift of all the specimens is provided. This parameter is one of the most important characteristics affecting the seismic performance of the C-SPSW system. In the all Specimens, with increasing drift, cumulative dissipated energy of the specimens was increased. Cumulative dissipated energy in the all strengthened specimens is larger than un-strengthened specimen (SPSP1). At the 2% drift, cumulative dissipated energy of the CSPSP2, CSPSP3, CSPSP4, and CSPSP5 specimens have been increased 24%, 28%, 48%, and 50%, respectively. As it can be observed, the amount of absorbed energy in the CSPSP2 and CSPSP3 and also in the CSPSP4 and CSPSP5 are close to each other. However the cumulative dissipated energy in specimens will be a little more, if the principal orientation of GFRP laminates is oriented in the direction of tension fields. Therefore, according to the results, fiber orientation is not a substantial variable in the cumulative dissipated energy of the C-SPSW.

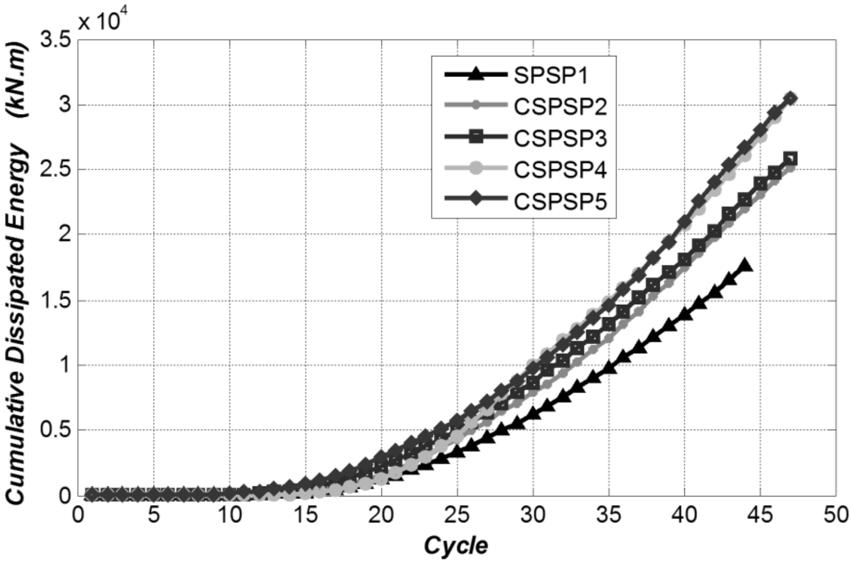


Figure 7. Cumulative dissipated energy of the specimens

4. CONCLUSION

In this paper, nonlinear behavior of composite steel plate shear wall systems, in which steel infill plate is strengthened by fiber reinforced polymer (FRP) layers, are experimentally investigated. The main results of this experimental study can be summarized as follows:

1- If steel infill plate strengthens by GFRP layers, yield and ultimate strength of the C-SPSW will significantly increase. Fiber orientation is an important variable in the shear strength. If principal orientation of GFRP laminates is oriented in the direction of tension fields, the shear strength will increase.

2- If steel infill plate strengthens by GFRP layers, initial and secant stiffness of the C-SPSW will significantly increase. And also the initial and secant stiffness will increase, if principal orientation of GFRP laminates is oriented in the direction of tension fields.

3- Cumulative dissipated energy in the all strengthened specimens is larger than un-strengthened specimen. Fiber orientation is not a substantial variable in the cumulative dissipated energy of the C-SPSW. However if the principal orientation of GFRP laminates is oriented in the direction of tension fields, the cumulative dissipated energy in specimens are a little more.

4- Fiber orientation is an important variable on the behaviour of the C-SPSW.

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REFERENCES

Astaneh-Asl, A. (2001). Seismic behaviour and design of steel shear walls. Steel TIPS Report, Structural Steel Educational Council, Moraga, CA.

Sabelli, R., Bruneau, M. (2007). Design guide 20: steel plate shear walls, American Institute of Steel Construction. Chicago, IL, USA.

Haghani, R. (2010). Analysis of adhesive joints used to bond FRP laminates to steel members-a numerical and

experimental study. *Construction and building materials* **Vol 24; Number 11**, pages 2243-2251
Zhao X-L, Zhang L. (2007). State-of-the-art review on FRP strengthened steel structures. *Engineering Structures* **Vol. 29(8)**:1808–23.
Photiou, NK., Hollaway, LC., Chryssanthopoulos, MK. (2002). Strengthening of an artificially degraded steel beam utilising a carbon/glass composites system. *2nd international conference on advances polymer composites for structural applications in construction. Guildford, UK: Woodhead Publishing Limited*.
Khazaei-Poul M, Natghi-Alahi F.(2011). Behavior of strengthened steel plate shear wall by FRP laminate. M.Sc. Thesis. International Institute Of Earthquake Engineering and Seismology, Iran.
Nateghi-Alahi, F., Khazaei-Poul, M. (2012). Analytical study on the strengthened steel plate shear walls by FRP laminate. *The 2nd International Conference on Rehabilitation and Maintenance in Civil Engineering*. (ICRMCE-2), **B32-105**, Solo, Indonesia.
Wagner, H. (1931). Flat sheet metal girders with very thin webs. Part i. General theories and assumptions. Tech Memo. National Advisory Committee for Aeronautics, Washington D. C. No. 604.
Thorburn LJ, Kulak GL, Montgomery CJ. (1983). Analysis of steel plate shear walls. Structural engineering report 107. Edmonton (Alberta, Canada).
Timler PA, Kulak GL. (1983). Experimental study of steel plate shear walls. Structural Engineering Report, University of Alberta, Canada, , No. 114.
Elgaaly, M. (1998). Thin steel plate shear walls behavior and analysis. *Thin walled Structures* **Vol. 32**,151-180
Berman, JW., Bruneau, M. (2003). Plastic analysis and design of steel plate shear walls. *Journal of Structural Engineering* **Vol. 129,11**, 1148–1156.
CAN/CSA S16-2001. Limit state design of steel structures. Canadian Standards Association, Willowdale, ON., Canada.
AISC (2005b). Specifications for structural steel buildings, American Institute of Steel Construction, Chicago, IL., USA.
Sabouri-Ghomi, S., Ventura, C., Kharrazi, HK. (2005). Shear analysis and design of ductile steel plate walls. *Journal of Structural Engineering*, **Vol. 131, No. 6, June 1.**
Kharrazi, MHK. (2005). Rational method for analysis and design of steel plate walls. Ph.D. dissertation. Vancouver (Canada): University of British Columbia.
Khazaei-Poul, M., Nateghi-Alahi, F. (2012). Theoretical and numerical study on the strengthened steel plate shear walls by FRP laminates, *International Journal of Engineering, IJE TRANSACTIONS C: ASPECTS*, **Vol. 25, No. 1**, p. 25-37.
Astaneh-Asl A. (2000). Seismic studies of innovative and traditional composite shear walls. Research project in-progress, Dept of Civil and Env Engineering: Univ. of California, Berkeley.
Astaneh-Asl A. (2001). Cyclic tests of steel shear walls. Research project. Berkeley: Dept of Civil and Env Engineering, Univ of California.
Rahai, A., Hatami, F. (2009). Evaluation of composite shear wall behavior under cyclic loadings. *Journal of Constructional Steel* **Vol. 65(7)**:1528–37.
Lubell, AS., Prion, HGL., Ventura, CE., Rezai, M. (2000). Un-stiffened steel plate shear wall performance under cyclic loading. *Journal of Structural Engineering*, **Vol. 126, No. 4**, p. 453-460.
Caccese, V., Elgaaly, M., Chen, R. (1993). Experimental study of thin steel-plate shear walls under cyclic load. *ASCE Journal of Structural Engineering*, **Vol. 119**, **No. 2**, p. 573-587.
Driver, RG., Kulak, GL., Kennedy, DJL. (1998). Cyclic tests of four-story steel plate shear wall. *ASCE Journal of Structural Engineering*, **Vol. 124**, **No. 2**, p. 112-120.
Sabouri-Ghomi, S., Roberts, TM. (1992). Nonlinear dynamic analysis of steel plate shear walls including shear and bending deformations, *Engineering Structures*, **Vol. 14**, **no.5**, pp. 309 317.
Vian, D., Bruneau, M., Tsai, KC., Lin, YC. (2009). Special perforated steel plate shear walls with reduced beam section anchor beams. I: Experimental investigation. *Journal of Structural Engineering* **Vol. 135, No. 3**.
Alinia, MM. (2005). A study into optimization of stiffeners in plates subjected to in- plane shear loads. *Thin-Walled Structures* **Vol. 43(4)**:845-60.
Alinia, MM., Dastfan, M. (2006). Behaviour of thin steel plate shear walls regarding frame members. *Journal of Constructional Steel Research* **Vol. 62**:730-8.
ATC. (1992). Guidelines for cyclic seismic testing of components of steel structures for buildings. Rep. No. ATC-24, Applied Technology Council, Redwood City, Calif.
ASTM, A-370- 03. (2003). Standard Test Methods and Definitions for Mechanical Testing of Steel Products. ASTM International, 100 Barr Harbor Drive, PO Box C700, West Conshohocken, PA 19428-2959, United States.