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# Finite Element Analysis of Light Gauge Steel Plate Shear Walls

E.B. Machaly<sup>1</sup>, S.S. Safar<sup>2</sup> and M.A. Amer<sup>1</sup> <sup>1</sup> Structural Engineering Department Faculty of Engineering, Cairo University, Egypt <sup>2</sup> Construction and Architectural Engineering Department American University in Cairo, Egypt

## Abstract

Previous research work on steel plate shear walls, SPSWs, showed that such a system provides adequate shear strength, and dissipated a significant amount of hysteretic energy when subjected to cyclic loading. Current design rules for SPSWs were based on simplified theory reducing the in-fill plate into tension strips in the direction of principal tensile stresses after buckling, which is designated a strip model. In this paper, the behavior of SPSWs subjected to lateral loads was investigated using the finite element method in which all plate elements were modelled with shell elements. Numerical results were validated by comparison with the strip model numerical results and test results published in the literature on fourteen SPSWs with a wide variety of material and geometric configurations. The proposed finite element model can be used to investigate the effect of SPSW configurations on its stiffness, strength, hysteretic behaviour and boundary elements design requirements.

**Keywords:** tension field action, hysteresis behaviour, cyclic analysis, push over analysis, post-buckling strength.

## **1** Introduction

Previous research work on steel plate shear walls, SPSW, revealed that such a system is capable of resisting lateral earthquake and wind loads effectively due to its adequate strength, ductility and large energy dissipation capacity [1 to 6]. A conventional SPSW is composed of infill plate surrounded by beams and columns designated as boundary elements. In typical SPSW designs [7, 8], the infill plate is un-stiffened and slender and thus principal compressive stresses due to shear cause the plate to buckle and form diagonal tension folds at early stages of loading. Lateral loads applied on SPSW after buckling are transferred through the infill plate by principal tensile stresses parallel to the fold lines. The capacity of the SPSW is

achieved when the infill plate is fully yielded. On the other hand, boundary elements are proportioned to remain essentially elastic with the exception of plastic hinge formation at the ends of horizontal boundary elements, HBE. Vertical boundary element, VBE, are proportioned to support significant flexure resulting from diagonal tension forces in the infill plate together with axial compression resulting from overturning moment and gravity loads applied on the SPSW. The axial yield of VBE at the base should be avoided to allow the in-fill plate achieve its full yield capacity [9]. For intermediate HBE, diagonal tension forces from the infill plate achieve its full plate above balance much of the forces from the plate below. Therefore, intermediate HBE are designed to support gravity loads and compression force resulting from inward flexure of VBE. Unlike intermediate HBE, top HBE requires large flexural capacity to support unbalanced tension forces in the web in addition to gravity loads.

Numerous experimental testing of SPSW was conducted to investigate the wall behavior under cyclic loading. Park et al [10] tested single bay-three stories SPSW depicted in Figure 1. Test parameters were infill plate thickness,  $t_p$ , and strength and compactness of column sections. Table 1 lists the specimen designation,  $t_p$ , panel aspect ratio (i.e panel width, L, to height, H, ratio) and width-to-thickness ratio,  $L/t_p$ , and material properties. The SC series had strong I-shaped column sections with depth,  $h_w$ , of 250 mm, flange width,  $b_f$ , of 250 mm, web thickness,  $t_w$ , of 20 mm and flange thickness,  $t_f$ , of 20 mm. On the other hand, the WC series had relatively weaker column sections with identical  $h_w$  and  $b_f$  as SC series but with plate thicknesses  $t_w$  and  $t_f$  reduced to 9 mm and 12 mm respectively. All intermediate beams were also composed of I-shaped section with  $h_w$  of 200 mm,  $b_f$  of 200 mm, and  $t_f$  and  $t_w$  of 16 mm.



Figure 1: SPSW tested by Park et al [10]

		t.			Yield	Ultimate
Test	Specimen	(mm)	L/H	$L/t_p$	stress, $F_y$	strength, $F_u$
		(IIIII)			(MPa)	(MPa)
1	SC2T	2.42	1.5	619.8	351	450
2	SC4T	4.49	1.5	334.1	392	461
3	SC6T	6.50	1.5	230.8	377	509
4	WC4T	4.49	1.5	334.1	392	461
5	WC6T	6.50	1.5	230.8	377	509

Table 1: Dimensions and properties of SPSW tested by Park et al [10]

Top beam was identical to intermediate beams except that the depth  $h_w$  was doubled. All beams were rigidly connected to columns by welding. Out-of-plane supports were provided at mid-height of second and third stories to avoid out-of-plane instability (see Figure 1). Test results indicated that with relatively strong column sections, the lateral force resistance, energy dissipated and stiffness of SPSW was progressively increased with the increase in  $t_p$ , and failure was caused by plastic collapse due to moment frame action. On the other hand, SPSW with weak columns did not gain significant strength and stiffness by increasing  $t_p$  and failure was caused by local buckling near base due to cantilever action.

Choi et al [11, 12] conducted tests on three story-single bay SPSW subjected to cyclic loading. Geometric configuration and dimensions of the five tested specimens are illustrated in Table 2 and Figure 2. The in-fill plate was composed of mild steel with  $F_y$  of 299 MPa whereas boundary elements were composed of high grade steel with  $F_y$  of 365 MPa.

Specimen		Panel Dir	Column				
	L	Н	$t_p$	L/H	$L/t_p$	$H(h_w x b_f / t_w x t_f)$	
FSPW 1	1500	1000	4	1.5	375	H(150x150 / 22x22)	
FSPW 2	2200	1000	4	2.2	550	H(150x150 / 22x22)	
FSPW 3	2200	1000	4	2.2	550	H(150x150 / 8x20)	
FSPW 4	2200	1000	4	2.2	550	H(150x150 / 22x22)	
FSPW 5	2200	1000	4	2.2	550	H(150x150 / 22x22)	

Table 2: Dimensions of SPSW tested by Choi et al [11, 12]



Figure 2: SPSW tested by Choi et al [11, 12]

All intermediate beams were composed of I-shaped section H(150x100 / 12x20) whereas the top beam section is composed of I-shaped section H(250x150 / 12x20). Test results showed that increasing the aspect ratio of panels pronounced the strength and ductility of SPSW since axial force in columns due to overturning moment is reduced and failure was caused by tearing of in-fill plate. On the other hand, reducing the column inertia reduced the capacity and ductility of the wall and

failure was caused by plastic collapse of the column. When the in-fill plate was detached from column (FSPW 4), the ductility of the wall was not affected but the strength was reduced and plastic hinges were observed at the beam-to-column connections. The introduction of rectangular openings in the in-fill plate with width 500 mm and with the total height of floor reduced the shear strength of the wall dramatically since the diagonal tension folds did not achieve full yield capacity due to lack of boundary element on all boundaries of the plate, however the ductility of the wall was slightly affected.

Chao et al [13, 14] conducted full scale test on two story-single bay SPSW shown in Figures 3 and 4. The in-fill plate was composed of mild steel with  $F_y$  of 195 MPa whereas the boundary elements were composed of high grade steel with  $F_y$  ranging from 358 to 492 MPa [14]. The testing program was established to monitor the hysteresis behavior of SPSW with narrow width together with examining the effect of using reduced section in beams, RBS, and reduced section in columns, RCS, and the effect of using horizontal struts attached to VBE (see Figure 4). Table 3 lists the geometric dimensions and designation of the four tested SPSW. Results indicated that specimen N with the largest column dimensions provided the highest strength compared to other specimens. On the other hand, the use of three horizontal struts per panel in specimens RS and CY increased the base shear supported by the wall by 10% compared to specimen S without horizontal struts.



Figure 3: Specimens N and S tested by Chao et al [13, 14]



Figure 4: Specimens RS and CY tested by Chao et al [13, 14]

No	Panel Dimensions (mm)				ım)	Column	Remarks
	L	Н	$t_p$	L/H	$L/t_p$	$H(h_w x b_f / t_w x t_f)$	
Ν	1790	2875	2.6	0.62	688	H(350x350 / 12x19)	DDC
S	1840	2875	2.6	0.64	707	H(300x300 / 10x15)	KDS
RS	1840	2875	2.6	0.64	707	H(300x300 / 10x15)	Struts and RBS
CY	1840	2875	2.6	0.64	707	H(300x300 / 10x15)	Struts, RBS and RCS.

Table 3: Dimensions of SPSW tested by Chao et al [13, 14]

It was shown that the use of RBS in beams or columns accelerated the formation of plastic hinges at the reduced section, however, had minimal effect on ductility and strength of the wall. Due to narrow width of the wall in all four specimens, failure was caused by plastic collapse of the column.

Current codes of practice [7, 8] adopt the analytical approach based on the strip model developed by Thorburn et al in 1983. In this model, the infill plate in each panel is replaced by a series of strips inclined at an angle  $\alpha$  with the vertical to represent tension field action in the plate. Each strip is assigned an area equals to the product of the strip width and the plate thickness. An expression for the angle  $\alpha$  was derived using the least work approach by Timler and Kuak [2] and adopted by current codes [7, 8]. Although it was reported by several investigators that the strip model provided good estimation of the elastic behavior of SPSW [2 to 6], it was reported that it misestimates the wall stiffness and requires modifications to predict the entire push-over curve including inelastic behavior. On the other hand, the strip model was not well suited to study the behavior of SPSW subjected to cyclic loading resulting from wind or earthquake loads. Several investigators [2 to 6, 9 to 14] reported the use of finite element approach to predict both monotonic and quasistatic cyclic loading of SPSW. Although the finite element method was validated using test results, it is important to establish a finite element model that can effectively predict the behavior of SPSW with wider variety of in-fill plate aspect ratio and thickness, SPSW with RBS and SPSW with intermediate struts.

In this paper, a finite element model for SPSWs was established using the general-purpose finite element software, ANSYS [15]. Due to the highly non-linear nature of the problem, both material and geometric non-linearities were incorporated in the analysis. The finite element solution was validated by comparison to fourteen test results on SPSWs with wide range of dimensions and configurations and obtained from different sources. The finite element solution was also compared to the simplified strip model results.

## 2 Finite Element Modelling

A finite element model for SPSW was established by representing all plate elements with iso-parametric finite strain shell element, SHELL 181, built in ANSYS element library [15]. SHELL 181 is a four nodded element with six degrees of freedom per node that is well-suited for large rotation and/or large strain nonlinear applications. The spatial strain field distribution is sampled at four Gaussian integration points on the shell mid-surface. At each integration point, the flexural behavior of the element is numerically integrated through five integration points across the thickness [15]. The size of the elements did not exceed 70 mm to provide accurate numerical results [1]. Nodes at column base were restrained from translation and rotation to mimic fixed support. The model is supported in the out-of-plane at the positions of lateral supports (see Figs 1, 2 and 3) to match testing condition. The material model is assumed to be elastic perfectly plastic, however, a small non-zero tangent modulus was assumed to avoid numerical instability. Von-Mises yield criterion is adopted in the analysis presented herein.

The analysis conducted is essentially non-linear static analysis at which loading is applied as lateral displacement, i.e. displacement controlled, to enhance convergence of numerical solution. In monotonic push-over analysis, nodal displacement at the level of top beam was applied incrementally and the converged solution after each displacement increment was obtained by iterations using the Full Newton-Raphson technique [15]. On the other hand, quasi-static cyclic loading was applied in accordance to test program described in literature [10 to 14] at which the model was subjected to loading cycles with progressively increasing displacement magnitude. The solution was terminated at the first limit load or when the maximum specified displacement was achieved.

Due to the highly non-linear nature of the problem, severe convergence difficulties can occur due to large displacements at each load increment. Therefore a special nonlinear stabilization technique was used by adding an artificial damper at each node with a small damping factor of  $5 \times 10^{-6}$  [1, 15]. The force in the damper is proportional to nodal displacement per load increment. Therefore the node that tends to be unstable has large displacement increment causing large damping force that reduces the displacement and stabilize the model. On the other hand, stable nodes with small displacement increment, the effect of damping forces are minimal [15].

### **3** Verification of Finite Element Model Results

The finite element model established herein using shell elements was verified by comparing push-over curve obtained from push-over loading and hysteretic curve obtained from quasi-static cyclic loading to test results of fourteen SPSWs published in literature [10, 11, 12 & 14] (see Sec 1). The analysis of some of the tested SPSWs was repeated herein using the strip model [2, 3] and double strip model proposed by Bruneau et al [5] to assess the use of conventional numerical methods in the analysis of SPSW. In the strip and double strip model, all boundary elements were modelled with the beam element, Beam188, whereas tension strips were modelled by the tension-only link element, Link180, in ANSYS element library [1, 15].

#### **3.1** SPSW tests conducted by Park et al [10]

The shell element model and double strip model results computed by ANSYS using quasi-static cyclic analysis of specimen SC4T were compared to test [10]. Figure 5 shows the contour plot of lateral displacements at the last loading cycle obtained from the two numerical models. Unlike the shell element model, the double strip model was unable to predict folds in the in-fill plate [5, 15]. Comparison of numerical hysteresis loops of top displacement versus total base shear with test [10] revealed that the shell element model provided good estimate of stiffness and shear capacity of the wall compared to the double strip model that underestimated stiffness and strength of the wall (see Figure 6). Figure 7 shows that the cumulative energy dissipated during cyclic loading was well predicted by the shell element model compared to the double strip model.

Examination of the Von-Mises stresses obtained from shell element model at peak lateral displacement (see Figure 8) revealed that the numerical solution predicted formation of plastic hinge at the 3<sup>rd</sup> storey beam-to-column connection as per test [10]. Unlike, conventional design assumption, the obtained Von-Mises stress pattern was non-uniform and did not achieve yield strength at all locations due to the complex effect of folds that provided stiffening effect to the plate when lateral displacements were reversed. Hence, the double strip model underestimated the stiffness and energy dissipated by the wall

![](_page_8_Figure_0.jpeg)

Figure 5: Contour plot of lateral displacement (mm) of SC4T

![](_page_8_Figure_2.jpeg)

Figure 6: Comparison of Hysteresis curve with test results of SC4T

Push-over load-displacement curves obtained from shell element model and conventional strip model (see Fig. 9) were compared to test results in Fig. 10. Unlike the strip model, the shell element model provided good estimation of the wall strength and stiffness.

![](_page_9_Figure_0.jpeg)

![](_page_9_Figure_1.jpeg)

![](_page_9_Figure_2.jpeg)

![](_page_9_Figure_3.jpeg)

000

120

![](_page_9_Figure_4.jpeg)

Figure 9: Contour plot of lateral displacement (Strip Model), SC4T

![](_page_9_Figure_6.jpeg)

Figures 11 & 12 illustrate the distribution of tension forces in the in-fill plate on boundary elements at maximum deflection computed from push-over analysis. Unlike theoretical assumption of uniform tensile yield stresses [7, 8], the shell element model revealed that tensile stresses did not reach the specified minimum yield stress,  $F_{y}$ , at connections due to limited tensile strains in the plate when it is bound by rigidly connected boundary elements. Numerical results, however, were closer to theoretical solution assuming that the ratio of expected yield stress to  $F_{y}$ ,  $R_{y}$ , equals to unity. The strip model, however, indicated significant reduction in tensile forces at corner connections compared to shell element model.

![](_page_10_Figure_1.jpeg)

Figure 11: Distribution of resultant tensile forces in-fill plate on columns, SC4T

The push-over curve was compared to hysteresis loop of SC2T, SC6T, WC4T and WC6T in Figure 13. Although the numerical solution predicted the wall stiffness, it underestimated the wall strength when  $t_p$  increased to 6 mm in SC6T. When column section was reduced, the shell element model captured local buckling failure in columns and estimated the reduced stiffness and strength of the wall. The strip model underestimated the stiffness and strength of the wall for all tested specimens.

![](_page_11_Figure_0.jpeg)

Figure 12: Distribution of resultant tensile forces induced by in-fill plate on top beam, SC4T

![](_page_11_Figure_2.jpeg)

Figure 13: Hysteresis and Push over curves of SPSW tested by Park et al [10]

#### **3.2** SPSW tests conducted by Choi et al [11, 12]

Figure 14 illustrates that the hysteresis loops, stiffness and strength of specimen FSPW1 (see Figure 2) was adequately predicted by the shell element model. The double strip model provided good estimation of wall strength; however, the hysteresis loops determined from the numerical solution did not match test results (see Figure 14b). Since the shell element model captured the formation of folds in the in-fill plate during cyclic loading, it predicted the cumulative energy dissipated adequately compared to the double strip model (see Figure 15). The Von-Mises stresses distribution obtained from the shell element model (see Figure 16) showed that failure was caused by plastic hinge formation at column base as per test results. On the other hand, the in-fill plate did not achieve yield stress at all locations due to the complex effect of folds that develop after buckling.

![](_page_12_Figure_2.jpeg)

a) Shell Elements Model

b) Double Strip Model

Figure 14: Hysteresis curves of FSPW1 obtained by ANSYS and test [11]

![](_page_12_Figure_6.jpeg)

re 15: Cumulative Energy Dissipate FSPW1

Figure 16: Von-Mises stresses (MPa), FSPW1

The push-over curve computed by the shell elements model was compared to hysteresis loops determined by test on specimens FSPW2 and FSPW3 in Figure 17. It was shown that the numerical solution determined the initial stiffness and shear strength of the wall adequately. In specimen FSPW3, the slope of the numerical push-over curve after limit load was reduced (see Figure 17b) due to the formation of plastic hinge in the column (see Figures 18 and 19) similar to pinching of experimental hysteretic loops.

![](_page_13_Figure_1.jpeg)

a) Specimen FSPW2

b) Specimen FSPW3

Figure 17: Hysteresis and Push over curves of SPSW tested by Choi et al [11]

Von-Mises stresses at limit load revealed that failure was caused by plastic hinge in the column below the first floor beam (see Figures 18 and 19) for specimen FSPW3. The deformed shape obtained by test resembled the numerical solution as depicted in Figure 19. Due to plastic hinge formation in the column, the in-fill plate did not achieve full yield capacity thus the wall shear strength was reduced from 1800 KN in specimen FSPW2 to 1500 KN in specimen FSPW3 when a weaker column section was used.

In specimen FSPW4, the in-fill plate was connected to beams only therefore the yield strength of the in-fill plate was not achieved and out-of-plane deformations were pronounced as depicted in Figure 20. On the other hand, internal forces in columns were dramatically reduced. A similar behavior was determined when vertical openings were introduced in the in-fill plate in specimen FSPW5 as shown by the Von-Mises stresses at maximum lateral displacement in Figure 21.

![](_page_14_Figure_0.jpeg)

at limit load, FSPW3

Figure 18: Von-Mises stresses (MPa) a) Finit

![](_page_14_Picture_2.jpeg)

a) Finite Element b) Test [11] Figure 19: Plastic hinge in column, FSPW3

![](_page_14_Figure_4.jpeg)

107.1 54.954 2.8045 Min

: FSPW5 quivalent Stress ype: Equivalent

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627.35 Max

420 367.85 315.7 263.55

211.4

159.25

ent (von-Mi

Figure 20: Von-Mises stresses (MPa) at maximum displacement, FSPW4

Figure 21: Von-Mises stresses (MPa) at maximum displacement, FSPW5

Since the yield tensile strength of the in-fill plate was not fully utilized in specimens FSPW4 and FSPW5 as shown in Figures 20 and 21, the shear strength of the wall was reduced compared to specimen FSPW2 with identical dimensions but in-fill plate was welded to columns and beams with no openings. The push-over curve determined from shell elements model predicted the stiffness and strength of specimens FSPW4 and FSPW5 and enveloped the hysteretic loops obtained by test as shown in Figure 22. The push-over curve, however, did not predict the pinching effect in hysteretic loops of FSPW4. The analysis was terminated at maximum displacement achieved in test with no signs of plastic hinge formation in beams or columns.

![](_page_15_Figure_0.jpeg)

a) Specimen FSPW4

b) Specimen FSPW5

Figure 22: Hysteresis and Push over curves of SPSW tested by Choi et al [12]

#### **3.3** SPSW tests conducted by Chao et al [13, 14]

The finite element model using shell elements was further verified by solving narrow SPSW with reduced beam and/or column sections and narrow SPSW stiffened with horizontal struts (see Sec 1) that were tested by Chao et al [13, 14]. Similar to test results, the push-over analysis of specimens N and S with RBS revealed that the SPSW reached its shear capacity due to the formation of plastic hinge in columns as depicted by Von-Mises stresses at limit load in Figures 23 to 26. Comparison of Von-Mises stresses contour plots shown in Figures 23 and 25 revealed that when the column section was reduced in specimen S, the plastic hinge at the base was shifted upwards away from base connection and tensile yield strength of in-fill plate was not fully utilized. Therefore specimen S recorded shear strength less than that of specimen N.

When horizontal struts were introduced in specimens RS and CY with reduced column section identical to that of specimen S, the Von-Mises stresses distribution (see Figures 27 and 28) at limit load revealed that the tensile strength of in-fill plate was fully utilized. However the shear strength of the wall was not significantly enhanced because failure was controlled by plastic collapse due to formation of plastic hinges at beam-to-column connections. On the other hand, the use of RCS at top beam in specimen CY caused the plastic hinge to develop in the column rather than the top beam but did not significantly increase the shear strength of the wall (see Figure 28). Comparison of the push-over curves obtained from the shell element model to hysteretic loops obtained by testing the four specimens revealed that the numerical solution predicted adequately the wall stiffness and enveloped the hysteretic loops determined by test. The shear strength determined by the finite element analysis was in good agreement with test results in specimens RS and CY, however, the numerical solution slightly underestimated the shear strength of specimens N and S as depicted in Figure 29.

![](_page_16_Figure_0.jpeg)

![](_page_16_Picture_1.jpeg)

Figure 23: Von-Mises stresses (MPa) at Limit load, Specimen N

a) Finite Element b) Test [13] Figure 24: Plastic hinge, Specimen N

![](_page_16_Figure_4.jpeg)

![](_page_16_Figure_5.jpeg)

Figure 25: Von-Mises stresses (MPa) at Limit load, Specimen S

a) Finite Element b) Test [13] Figure 26: Plastic hinge, Specimen S

## 4 Summary and Conclusions

In this paper, a finite element model using shell elements was established for SPSWs using the general-purpose finite element program, ANSYS. The finite element model was validated by comparing quasi-static cyclic analysis results and monotonic push-over analysis results to hysteresis curves obtained by test results published in literature on fourteen SPSWs. The tested specimens were characterized with wide variety of yield strength, in-fill plate thickness ( $t_p = 2.6$  to 6.5 mm), in-fill plate width-to-thickness ratio ( $L/t_p = 230$  to 707), panel aspect ratio (L/H = 0.62 to 2.2),

![](_page_17_Figure_0.jpeg)

![](_page_17_Figure_1.jpeg)

Figure 27: Von-Mises stresses (MPa) at limit load, Specimen RS

Figure 28: Von-Mises stresses (MPa) at limit load, Specimen CY

and various geometric configurations including: openings in the in-fill plate, use of reduced section in HBE and/or VBE and use of horizontal struts. The use of the conventional strip model to compute push-over curves and the double strip model to compute the hysteresis behavior of SPSW was also assessed by comparison to test results and numerical results obtained from the shell elements model.

Based on the above-mentioned work the following was concluded:

- Although the problem of SPSW is rather complex due to geometric and material non-linearities, the proposed finite element model using shell elements predicted adequately the hysteretic behavior, stiffness, strength, energy dissipated and mode of failure of tested SPSWs subjected to cyclic loading. Moreover, the push-over curve computed by the proposed finite element model was sufficient to predict the SPSW stiffness and strength.
- The push-over curve computed by strip model underestimated the wall stiffness and shear strength.
- The double strip model did not predict the hysteretic behavior of SPSW accurately and underestimated the energy dissipated by the wall since it does not include the effect of folds that develop in the in-fill plate after buckling.
- Although the numerical solution using shell elements requires larger storage and run-time requirements compared to simplified strip model and/or double strip model, the strip model(s) require further research work to enhance the predicted stiffness, strength, hysteresis behavior, energy dissipated during cyclic loading, and to account for non-conventional SPSW configurations such as: horizontal struts and/or openings in the in-fill plate.
- The finite element model proposed herein may be used as a powerful numerical tool to investigate the effect of SPSW material properties and geometric configuration on wall stiffness, strength, hysteresis behavior and design requirements of boundary elements.

![](_page_18_Figure_0.jpeg)

Figure 29: Hysteresis and push-over curves of SPSWs tested by Chao et al [13, 14]

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