



# Seismic Behavior Investigation of the Corrugated Steel Shear Walls Considering Variations of Corrugation Geometrical Characteristics

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## Abstract

The corrugated steel plate shear walls have recently been proposed to address the seismic issues associated with simple steel plate shear walls; however, stiffness, strength, and ductility of the corrugated shear walls are significantly affected by varying the corrugation geometry under seismic loading. The present study investigates steel shear walls' models with corrugated or simple infill plates subjected to monotonic and cyclic loads. The performance of the corrugated steel plate is evaluated and then compared to that of the simple steel plates by evaluating the damping ratios and energy dissipation capability. The effect of corrugation profile angle, the existence of an opening, and the corrugation subpanel length are numerically investigated after validation of the finite element modeling methodology. The results demonstrate that incorporating corrugated plates would lead to better seismic damping ratios, specifically in the case of opening existence inside of the infill plate. Specifically, the corrugation angle of 30° decreases the ultimate strength, while increasing the initial stiffness and ductility. In addition, the subpanel length of 100 mm is found to be able to improve the overall performance of shear wall by providing each subpanel appropriate support for the adjacent subpanel, leading to a sufficient buckling resistance performance.

**Keywords** Steel shear wall · Corrugated steel shear walls · Corrugation angle · Dissipated energy · Finite element analysis

## 1 Introduction

The steel plate shear wall (SPSW) is one of the most efficient means of lateral load resistance systems, specifically if the loads are induced by earthquakes. The SPSW contains a boundary frame of beams and columns connected to a steel infill plate to transfer the lateral loads to the supports. The behavior of SPSWs with unstiffened plates largely depends on the post-buckling strength of the infill plates, as the infill plates are significantly thin and would buckle at the initial

steps of loading. Therefore, the presence of surrounding frame members with a large flexural stiffness is essential for the development of a uniform tension field and for the effective use of steel.

Many numerical and experimental studies have demonstrated that SPSWs have a high initial stiffness, ductility, strength, and robustness under cyclic loading; furthermore, due to offering an effective energy dissipation source for structures, they can be effectively employed to avoid seismic resistance (Lu et al. 2016; Nooralizadeh et al. 2017; Sahoo et al. 2015; Farzampour 2016; Farzampour et al. 2017). The behavioral trends of SPSWs have been investigated in various design specifications, such as AISC 341 (2005, 2010, 2016), NEHRP recommendations (FEMA 2004), and CSA (1994, 2001, 2009), and this research direction has been supported by many numerical and experimental research projects.

Initially, SPSWs were designed to avoid out of plane buckling of the infill plates, with the instructions to stiffen the steel infill plate to locally limit the inelastic buckling between the stiffening elements (Takahashi et al. 1973). Later, it was observed that, although buckling would happen in a low lateral displacement in thin web panels, the

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steel shear walls are capable of withstanding large loads. In addition, it was also found that the post-buckling strength of the web is significant (Thorburn et al. 1983), and that the stiffening does not have an important effect on the overall strength (Guo et al. 2015).

In this respect, Thorburn et al. (1983) used a diagonal strip model according to the theory of pure diagonal tension to show the diagonal tension field effect on strength of shear walls. The strip model results were reported to be able to relatively accurately predict the tension field action. The method was further clarified and elaborated by the tests conducted by Timler and Kulak (1983), Driver et al. (1998), Berman and Bruneau (2003, 2008), Qu et al. (2008), Guo et al. (2012, 2013), Gholipour and Alinia (2016), Tian et al. (2016), and Ozelik and Clayton (2017). Furthermore, in several studies, such as Habashi and Alinia (2010), Kharrazi et al. (2008), Roberts and Sabouri-Ghomi (1991), Sabouri-Ghomi et al. (2005) and Khanoukia et al. (2016) the plate-frame interaction (PFI) theory was proposed for estimating the behavior of different steel shear walls containing thick or thin plates, stiffened or unstiffened steel plates with openings.

Previously, corrugated steel plates were introduced to replace stiffened steel plates in girders due to their high in- and out-of-plane geometric stability (Bacher and Kirkland 1986; Lee and Kennedy 1988). Some strategies were proposed to overcome the out-of-plane buckling issues by strategically removing the material from the plates leading to yielding mechanisms as the predominant mode of behavior (Farzampour and Eatherton 2017, 2018a, b). In addition, in the last two decades, the corrugated steel plates have been used in steel shear walls as a different strategy to postpone the buckling (Farzampour et al. 2015). The behavior of the corrugated steel shear walls (CSSWs) has been studied in several numerical and experimental applications, so the general behavior of these walls is now reasonably well understood. For instance, Mo and Perng (2000) found that employing the corrugated steel wall in the reinforced concrete (RC) buildings greatly improves the seismic behavior of the RC frames. Furthermore, Gholizadeh and Yadollahi (2012) compared the behavior of a flat steel plate and a corrugated plate using the finite element analysis software. In addition, the half-scale, one-story, one-bay corrugated steel shear walls with trapezoidal section were cyclically tested by Emami et al. (2013). Likewise, Vigh et al. (2013) proposed the seismic performance factors for the steel corrugated shear wall system. Another relevant study was conducted by Shimizu et al. (2013) who experimentally investigated the corrugated steel plate wall with end failure under both monotonic and cyclic loading. Similarly, Vigh et al. (2014) developed a component model for calibration of a corrugated shear wall under cyclic loading. In addition, push over trends of trapezoidally corrugated steel shear walls under cyclic and monotonic loadings were evaluated using finite element analyses by Emami and Mofid (2014). Tong and Guo (2015) both theoretically

and numerically studied the elastic buckling behavior of stiffened trapezoidal corrugated shear walls. Extensive numerical studies aiming to investigate the behavior of CSSWs with and without an opening were conducted by Farzampour and Laman (2016), Farzampour et al. (2015) and Farzampour and Yekrangnia (2014). Dou et al. (2016) presented the theory of elastic shear buckling of the sinusoidal CSSWs. Furthermore, Zhao et al. (2017) studied the cyclic behavior of CSSWs with the corrugation in the vertical and horizontal directions. Another numerical and experimental study of the corrugated steel sheathing under shear and gravity loads was conducted by Zhang et al. (2017). Finally, the interaction between local and global buckling strength of CSSW under pure shear force was recently reported in a study by Hosseinzadeh et al. (2017).

In the present study, we evaluate the energy dissipation capacity of CSSWs and propose the corresponding damping coefficients. The finite element analysis is performed on CSSWs and SPSWs' computational models to extract the push-over behavior of the models. The effect of opening existence inside of the infill plate on the damping ratio is studied. We also conduct an in-depth analysis of the corrugation angle and the corrugation subpanel length effect on strength, stiffness, and ductility of CSSWs.

## 2 Computational Study

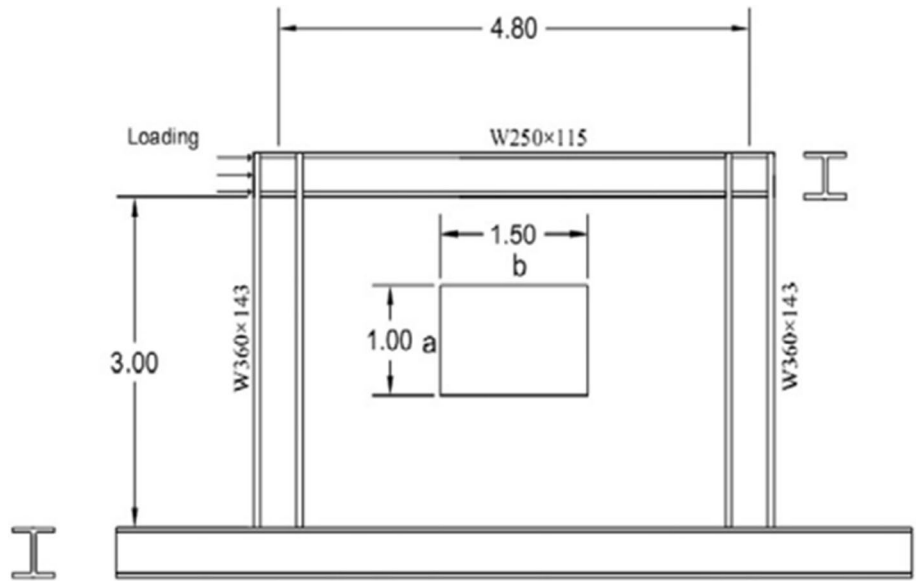
### 2.1 Finite Element Modeling

The member sizes, member orientation, applied loading condition are shown in Figs. 1 and 2. To simulate the conventional residential building, a one-story panel with the height of and the length of 4.8 m centerline to centerline was considered. The rigid frame for the girder-to-column connection was implemented. The plate frame interaction method (PFI) was used to establish the finite element (FE) models for both SPSW and CSSW with and without any opening in the panel. Therefore, the vertical load effect on the lateral resistance system was ignored.

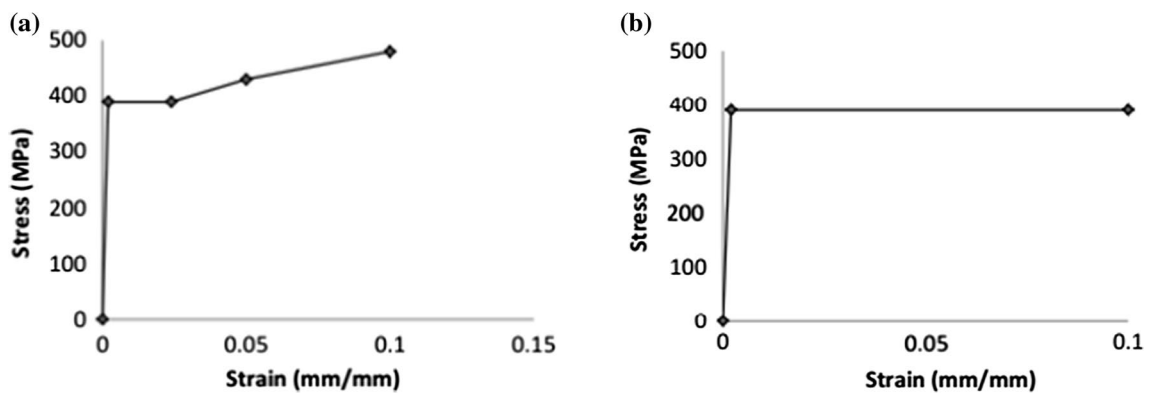
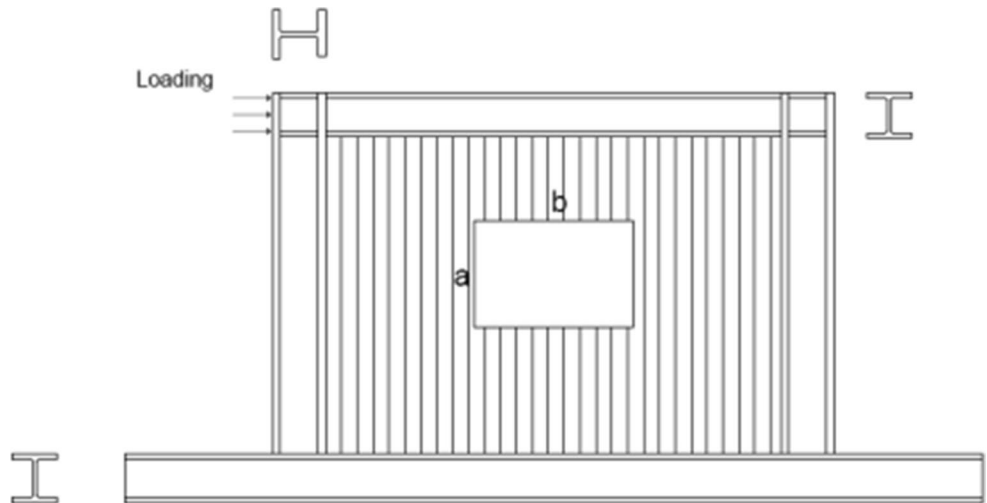
The out-of-plane movement was braced for the girder to simulate the conventional construction conditions. The commercial finite element program, ABAQUS (ABAQUS-6.14 2014), was used to extract the push-over curves and different behavior modes for all the models. The post-processing was conducted with the MATLAB package to calculate the parameters of interest associated with the shear walls.

The general four node reduced integration shell element (S4R) was used to predict the nonlinear behavior and large displacement of the models. The occurrence possibility of shear locking and hour glassing was checked and eliminated for each model. Material yield limits for panel and boundary elements were based on Habashi and Alinia (2010) (see Fig. 3).

**Fig. 1** The simple steel plate shear wall with opening



**Fig. 2** The corrugated steel plate shear wall with opening with the same boundary elements



**Fig. 3** Material model behavior. **a** The girder and column constitutive model. **b** The panel constitutive model

The intersection shell elements were connected with each other in all the beam-to-column moment resisting connections. The shear walls whether corrugated or simple were continuously tied with the columns and girders. Following Driver et al. (1998) and Emami et al.'s (2013) laboratory tests, the boundary condition for bottom support girder and the column base were fully fixed. The dynamic explicit numerical procedure was used to perform the quasi-static monotonic displacement controlled loading. The loading was applied to the upper girder and increased monotonically up to the suggested 2.5% ultimate drift ratio of the story according to ASCE7–10.

## 2.2 Verification Study

The modeling methodology in the ABAQUS finite element program was confirmed by two widely known laboratory tests. The push over curves under the monotonic loading conditions were extracted and then verified with the actual experimental tests. The failure modes and the

general behavior of the models were compared against the corresponding experimental tests. The four-story simple plate steel shear wall tested by Driver et al. (1998), as well as one-story corrugated plate steel shear wall tested by Emami et al. (2013) were constructed in the finite element software. The pushover results and the general behavior of the models yielded satisfactory precision in capturing the laboratory test results with numerical models (see Figs. 4, 5). As shown in Fig. 5, the finite elements models were able to completely capture the behavior of the shear walls system. According to Driver et al. (1998), the limit state associated with the wall was the buckling of column at the base, which was captured in the FE models. The results indicate that while, for Driver et al. (1998) tests, the accuracy of modeling methodology was within 98% under the monotonic loading condition, the modeling methodology was accurate at more than 85% of Emami et al.'s (2013) experimental test.

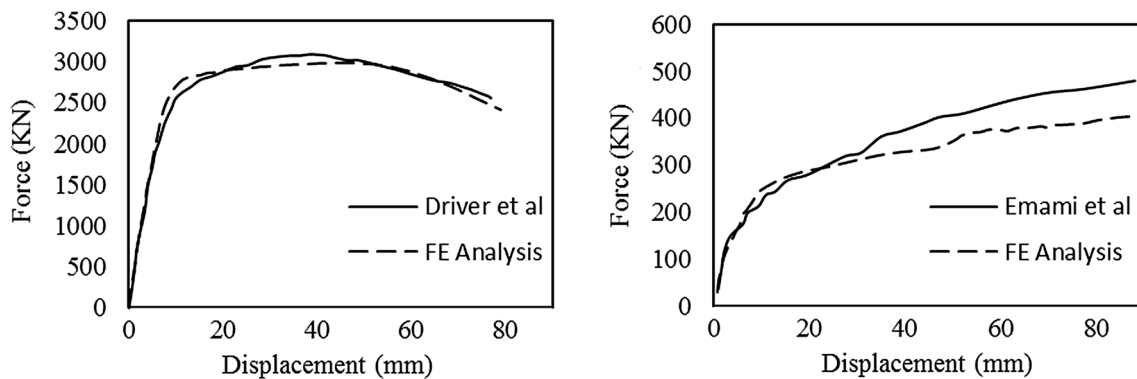


Fig. 4 The verification of the test results with the FE methodology for monotonic loading

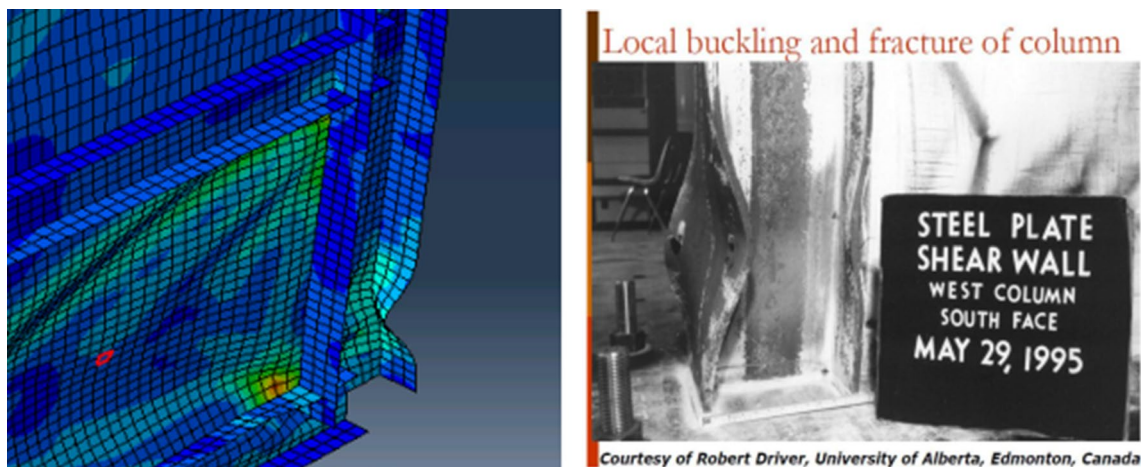


Fig. 5 The verification of the test results behavior and failure modes with the FE numerical model

### 3 Investigation of the Seismic Characteristics of the Corrugated Steel Shear Walls

Thereafter, in order to investigate the damping ability under cyclic loading, 40 different models were constructed in the FE program. The corrugation, plate thickness, and the presence of opening in the infill panel were investigated with regard to seismic response of the shear walls. The opening sizes were 70 cm × 100 cm, 100 cm × 150 cm, and 150 cm × 225 cm. The damping coefficients were calculated based on the equating the strain energy ( $E_S$ ) and the energy dissipated within a loop ( $E_L$ ) which is shown in Fig. 6. Based on the dividing of Eq. (2) over Eq. (1), the damping coefficient could be derived as shown in Eq. (3).

$$E_S = \frac{1}{2}Ku_0^2 = \frac{1}{2}mw_nu_0^2 \tag{1}$$

$$E_D = \pi u_0(Cwu_0) = \pi u_0(2\epsilon mw_n)wu_0 = 2\pi\epsilon mw_nu_0^2 \tag{2}$$

$$\epsilon = \frac{1}{4\pi} \left( \frac{1}{w/w_n} \right) \left( \frac{E_L}{E_S} \right) \tag{3}$$

where  $m$ ,  $w_n$ ,  $C$ , and  $K$  are the mass, natural frequency, damping constant, and stiffness of the system.  $w$  and  $u_0$  are the loading frequency and ultimate displacement within a hysteresis loop, respectively. The results showed that the CSSW and SPSW would have approximately similar damping coefficients in different thicknesses of the infill plates if there was no opening in the infill plate; however, CSSW with opening would show a significantly better performance compared to the corresponding SPSW with opening in all different opening sizes and infill plate thicknesses. The reason for the better performance of CSSWs with opening compared to the corresponding SPSWs is the ability of the corrugated panel in limiting the plastic strains at the edges and the corners of the opening. Of note, this difference, which is tangible for one cycle, would be even more significant if the number of cycles increased (see Table 1 for a summary).

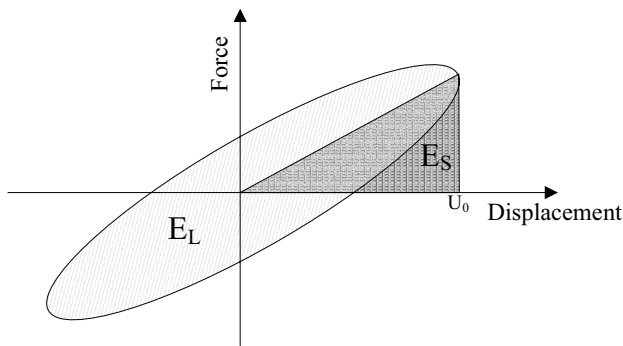


Fig. 6 The damping coefficient concept ( $\epsilon$ )

Table 1 The comparison of damping coefficient for CSSW and SPSW with or without opening

	NOOP	OP 70×100	OP 100×150	OP 150×225
<i>CSSW</i>				
1.5 mm	0.28	0.52	0.39	0.33
2 mm	0.30	0.54	0.41	0.36
2.5 mm	0.30	0.49	0.43	0.33
3.25 mm	0.29	0.47	0.44	0.34
4 mm	0.30	0.54	0.44	0.35
<i>SPSW</i>				
1.5 mm	0.30	0.50	0.37	0.30
2 mm	0.34	0.52	0.34	0.30
2.5 mm	0.35	0.44	0.34	0.31
3.25 mm	0.29	0.44	0.34	0.37
4 mm	0.29	0.45	0.35	0.38

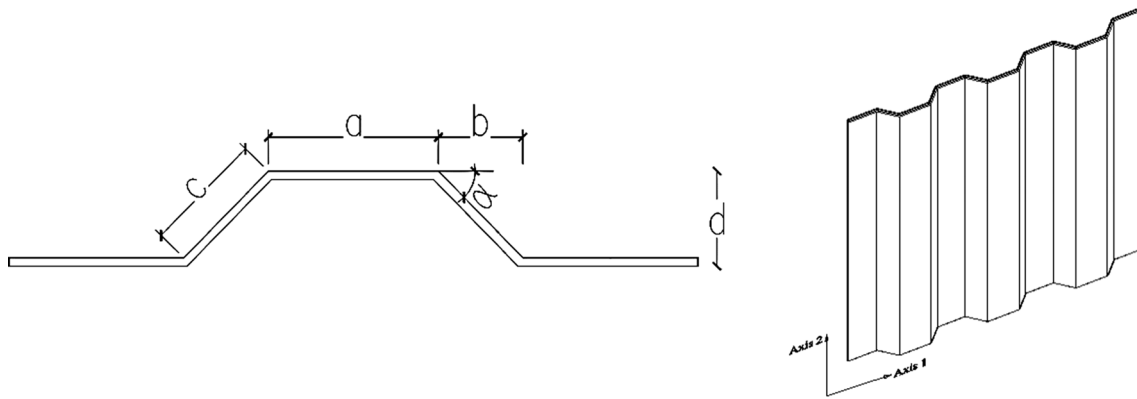
NOOP no opening in the system, OP opening within the system

### 4 Investigation of the Corrugation Profile Angle Effect on CSSW Behavior

In the next step, we investigated the general behavior of corrugated steel plate shear wall with opening considering different corrugation profile angles (Fig. 7). Specifically, we focused on three different corrugation profile angles (30°, 45°, 60°) (see Tables 2, 3, 4). Previous studies have suggested that the profile angle below 30° or over 60° would induce some instability for subpanels and would lead to immature development of interactive shear buckling capacity (e.g., Driver et al. 2006). A set of 36 CSSWs was constructed in the FE software to simultaneously investigate the effect of corrugation profile angles, opening sizes in infill plate, and plate thicknesses. Based on the definition in the literature (Farzampour et al. 2015), the parameters of the ultimate strength, initial stiffness, and ductility were calculated for all models. Subsequently, for an easier comparison, the results were normalized to the corresponding CSSW with the corrugation profile angle of 30°. The normalized results are shown in Tables 2, 3 and 4.

As can be seen the results, despite the reduction in ultimate strength, the walls having panels with the corrugation profile angle of 30° have a noticeable increase in the initial stiffness and ductility. The reason for such behavior is due to the fact that each subpanel sufficiently supported the adjacent subpanel. By having lower angles, each subpanel would have higher in-plane stiffness, while the out-of-plane stiffness would be reduced.

In addition, with an increase of the corrugation profile angle, the capacity for general shear buckling stress would be increased, thereby leading to an increase in interactive shear buckling stress capacity and, ultimately, the increase in total strength carried by the infill plate.



**Fig. 7** The corrugation profile angle ( $\alpha$ )

**Table 2** The ultimate strength ratio variation with the corrugation profile angle

Thickness	OP 70×100			OP 100×150			OP 150×225			Without OP		
	CO 60	CO 45	CO 30	CO 60	CO 45	CO 30	CO 60	CO 45	CO 30	CO 60	CO 45	CO 30
t=4	1.16	1.00	1.00	1.35	1.23	1.00	1.26	1.20	1.00	1.27	1.25	1.00
t=2.5	1.28	1.17	1.00	1.29	1.17	1.00	1.35	1.24	1.00	1.29	1.07	1.00
t=1.5	1.21	1.08	1.00	1.27	1.15	1.00	1.21	1.17	1.00	1.04	0.89	1.00

CO corrugated. CO 60, 45 and 30 show the corrugation profile angles of 60°, 45° and 30°, respectively

**Table 3** The initial stiffness ratio variation with the corrugation profile angle

Thickness	OP 70×100			OP 100×150			OP 150×225			Without OP		
	CO 60	CO 45	CO 30	CO 60	CO 45	CO 30	CO 60	CO 45	CO 30	CO 60	CO 45	CO 30
t=4	0.98	1.00	1.00	0.86	0.89	1.00	0.88	0.88	1.00	0.88	0.89	1.00
t=2.5	0.87	0.95	1.00	0.85	0.89	1.00	0.82	0.84	1.00	0.90	0.97	1.00
t=1.5	0.90	0.97	1.00	0.83	0.88	1.00	0.98	0.95	1.00	0.92	1.00	1.00

**Table 4** The ductility ratio variation with the corrugation profile angle

Thickness	OP 70×100			OP 100×150			OP 150×225			Without OP		
	CO 60	CO 45	CO 30	CO 60	CO 45	CO 30	CO 60	CO 45	CO 30	CO 60	CO 45	CO 30
t=4	0.62	0.77	1.00	0.60	0.69	1.00	0.86	0.90	1.00	1.34	0.62	1.00
t=2.5	0.71	0.85	1.00	0.69	0.80	1.00	0.80	0.88	1.00	0.70	0.91	1.00
t=1.5	0.97	0.98	1.00	0.96	0.99	1.00	0.95	1.11	1.00	0.90	0.98	1.00

## 5 Investigation of the Subpanel Length on CSSW Behavior

Although the amount of steel used to construct the shear wall would differ considerably, the subpanel length effect on ultimate strength and stiffness was significant. In order to reach to the appropriate size of subpanel, four different subpanel values are considered. Therefore, the subpanel lengths of 100, 150, 200, and 250 mm were considered

(see Tables 5, 6, 7, 8). All results were normalized to the model with the subpanel of 100 mm. A total of 48 models were constructed to simultaneously consider the effect of opening size, subpanel length, and thickness.

The results of our investigation of the effect of subpanel length on ultimate strength with or without opening are summarized in Tables 5 and 6. Furthermore, the results relative to the effect of subpanel size on initial stiffness are summarized in Tables 7 and 8. As can be concluded from the



**Table 5** The effect of the subpanel length on the ultimate strength of the shear wall

Thickness	OP 70×100				OP 100×150			
	CO 100	CO 150	CO 200	CO 250	CO 100	CO 150	CO 200	CO 250
t=4	1.00	0.95	0.84	0.90	1.00	0.97	0.89	0.98
t=2.5	1.00	0.99	0.88	0.81	1.00	0.99	0.84	0.83
t=1.5	1.00	0.92	0.91	0.90	1.00	0.98	0.94	0.93

CO corrugated. CO 100, 150, 200, and 250 = the corrugation profile lengths of 100, 150, 200, and 250 mm, respectively

**Table 6** The effect of the subpanel length on the ultimate strength of the shear wall

Thickness	OP 150×225				Without OP			
	CO 100	CO 150	CO 200	CO 250	CO 100	CO 150	CO 200	CO 250
t=4	1.00	1.00	0.98	0.99	1.00	0.92	0.83	0.86
t=2.5	1.00	0.99	0.94	0.92	1.00	0.78	0.75	0.79
t=1.5	1.00	0.96	0.91	0.90	1.00	0.73	0.71	0.75

**Table 7** The effect of subpanel length on the initial stiffness of the shear wall

Thickness	OP 70×100				OP 100×150			
	CO 100	CO 150	CO 200	CO 250	CO 100	CO 150	CO 200	CO 250
t=4	1.00	1.01	0.98	0.73	1.00	0.98	0.96	0.73
t=2.5	1.00	0.99	0.94	0.92	1.00	0.99	0.94	0.92
t=1.5	1.00	0.97	0.88	0.86	1.00	0.97	0.88	0.86

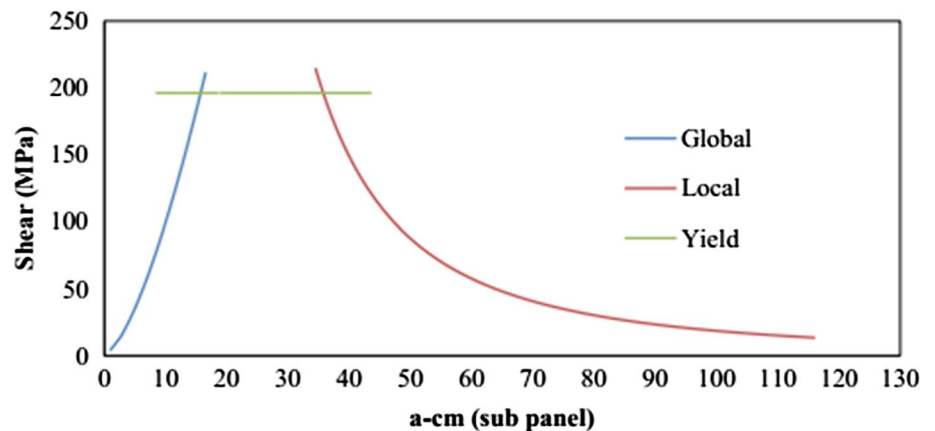
**Table 8** The effect of the subpanel length on the initial stiffness of the shear wall

Thickness	OP 150×225				Without OP			
	CO 100	CO 150	CO 200	CO 250	CO 100	CO 150	CO 200	CO 250
t=4	1.00	0.92	0.91	0.90	1.00	0.95	0.91	0.90
t=2.5	1.00	0.90	0.87	0.85	1.00	0.99	0.95	0.89
t=1.5	1.00	0.99	0.96	0.81	1.00	0.96	0.81	0.95

results, the corrugated panel 100 mm in length has a more appropriate performance compared to the panels of the remaining corrugation lengths. The difference in the corrugation length is even more obvious when the CSSW does not have any opening inside of the corrugated panel.

Furthermore, Fig. 8 shows the effect of subpanel size on the buckling mode domination for a typical corrugated plate (length, 4.8 m; height, 3.2 m; 4 mm, thickness). The limit states governing the shear buckling stress behavior are shown in Fig. 8. As can be seen in the results, by lowering

**Fig. 8** The shear buckling stress variation with subpanel length



the subpanel size, the global shear buckling stresses would govern as the major parameter contributing to the interactive shear stress, which would eventually yield to a more stable condition.

## 6 Conclusions

The present study investigated the seismic performance of the corrugated steel plate shear walls. Two well-established tests are numerically verified to validate the methodology of modeling in capturing the push-over backbone curves and the structural modes. Our results suggest that, in the case of the walls without opening inside of the infill plate, the corrugated shear walls would develop the same damping capability under seismic loading as compared to the corresponding simple plate shear walls; however, corrugated shear walls with openings were found to have a considerably better performance as compared to the corresponding simple plate shear walls.

In addition, we also studied the effect of varying corrugation geometry under the existence of an opening inside of the infill plate. The results show that increasing the profile angle would lead to an increase in ultimate strength; however, the stiffness and ductility would decrease. The effect of subpanel length on strength and stiffness of the shear walls is shown to be considerable. Our results show that the subpanel length of 100 mm has a better seismic performance as compared to the subpanels of the other corrugation lengths. Therefore, it can be concluded that, despite reduction in ultimate strength, the walls with the panels with the corrugation profile angle of  $30^\circ$  have a considerable increase in the initial stiffness and ductility.

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