International Journal of Engineering and Technology Innovation, vol. 9, no. 3, 2019, pp. 228-239

Performance of Steel-Concrete Shear Walls with Two-Sided Reinforced Concrete

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Received 03 November 2018; received in revised form 03 December 2018; accepted 02 January 2019

Abstract

This paper deals with the performance of Steel-Concrete Shear Walls (SCSWs) which have reinforced concrete on both sides of the steel plate subjected to cyclic loads. Finite element software ABAQUS is applied to analyze the SCSWs. Accuracy of the finite element modeling is verified by comparison of the theoretical results with those obtained experimentally. Then, various variables are studied in order to evaluate their effects on the performance of the SCSWs. These variables include thickness of concrete, steel plate thickness, number of bolts, gap size between reinforced concrete and steel frame, the percentage of reinforcement in reinforced concrete, and beam and column profiles of the steel frame. It is concluded that the change of the variables influences the ultimate load capacity, ductility, and energy dissipation of the SCSWs. Moreover, buckling of the walls is discussed.

Keywords: steel-concrete shear wall, cyclic load, finite element method, concrete thickness

1. Introduction

Reinforced concrete shear walls have widely been utilized in structures to resist lateral loads. However, studies have been conducted on Steel Shear Walls (SSWs) in the past 30 years, which resulted in improving the use of these walls. One of the problems associated with these SSWs is out-of-plane buckling of steel plate that causes diagonal lines in the steel plate. If these lines are distributed more uniformly, the shear capacity is enhanced. This point can be obtained by the use of reinforced concrete that is attached to the steel plate by bolts, which finally leads to Steel-Concrete Shear Walls (SCSWs).

The SCSWs include the walls with and without a gap between the reinforced concrete and steel frame. Concrete fails faster and under lower loads in the type of the SCSWs without a gap. Nevertheless, concrete is not subjected to the effect of lateral loads in the SCSWs with a gap, because concrete is not involved with the steel frame and its task is only to delay the steel plate buckling. Concrete then fails under larger loads.

Takanashi et al. [1] tested one-story and two-story specimens of SSWs. Different experimental tests were conducted on SSWs without stiffener under uniform and cyclic loads [2-4]. Zhao and Astaneh-Asl [5] presented an innovative composite shear wall with a gap between the reinforced concrete and steel frame. Arabzadeh et al. [6] experimentally studied behavior of one-story and three-story specimens. Sabouri-Ghomi and Sajjadi [7] did experimental and numerical investigations of SSWs with and without stiffeners. Bhowmick et al. [8] carried out a seismic analysis of SSWs with a plate having an opening. Guo and Yuan [9] assessed SSWs including a steel plate with a precast concrete panel. Rahnavard et al. [10] numerically evaluated some parameters of SCSWs. Kioumarsi et al. [11] analyzed the effect of increasing the height over the behavior of SSWs. Hao et al. [12] performed an experimental investigation on the axial compression behavior of SCSWs. Wang et al. [13] experimentally studied the seismic behavior of SCSWs.

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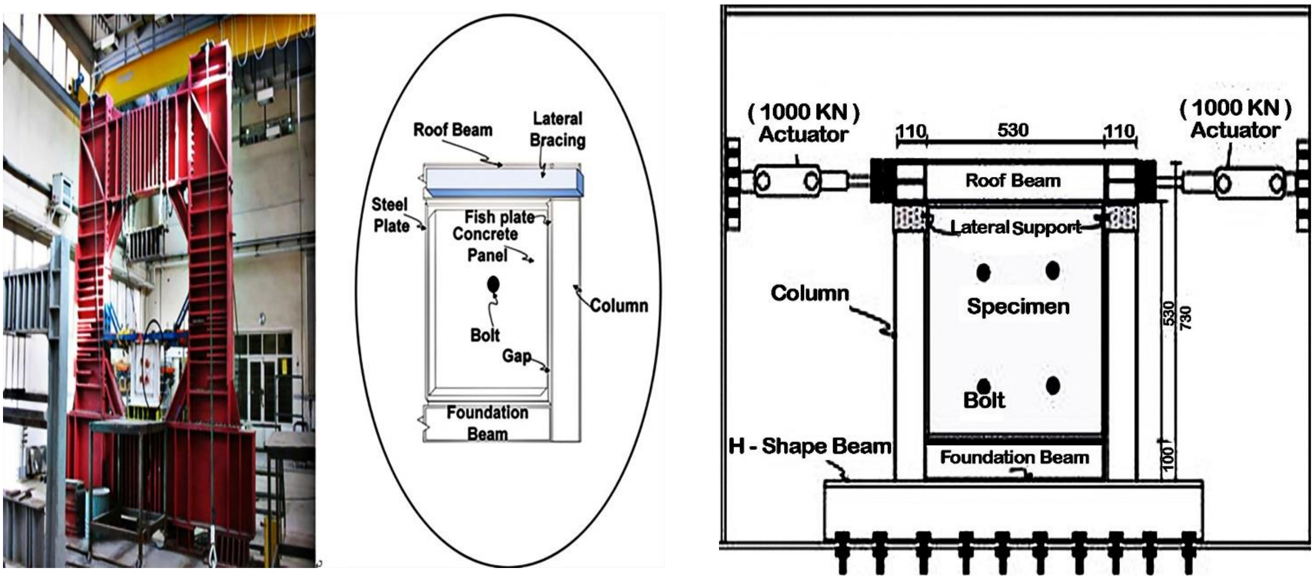
The purpose of this paper is to investigate the performance of the SCSWs with reinforced concrete on both sides of the steel plate. The SCSWs have a gap between reinforced concrete and steel frame. In order to perform this investigation, ABAQUS software [14] is used to achieve nonlinear analyses. Two experimental tests [6] are modeled herein to do the modeling verification. Comparisons of the modeling results with the experimental tests results uncover the accuracy of the model. Then, different variables are considered for the parametric study of the SCSWs models. Variables include (1) thicknesses of concrete (30 mm, 60 mm, and 100 mm), (2) thicknesses of steel plate (2 mm, 4 mm, and 8 mm), (3) gap sizes between reinforced concrete and steel frame (5.625 mm, 11.25 mm, and 22.5 mm), (4) number of bolts (4, 8, and 12), and (5) percentages of reinforcements in reinforced concrete (0.25%, 0.5%, and 1%) and (6) beams and columns profiles of steel frame (IPE100 and IPE140). Thereafter, the effects of the variables on the performance of the SCSWs are assessed. Buckling of the walls is evaluated, too.

2. Experimental Testing of SCSWs

Experimental tests of SCSWs [6] have been chosen for the nonlinear modeling in this study. The tested SCSWs comprises a steel frame (beam and column profiles), steel plate, fish plate, concrete, reinforcement, and bolts. The reinforced concrete is connected to one or both sides of the steel plate of the SCSWs by bolts. The connection between the beams and columns in the steel frame is rigid. The fish plate has connected the steel plate to the steel frame. The bolts have attached the reinforced concrete to the steel plate. A bottom beam of the steel frame is fixed and roof beam of the steel frame has lateral support to prevent out-of-plane displacement of the frame. The steel frame is connected to the floor using pins. Fig. 1 illustrates the setup of the experimental tests, the details of the wall, and the schematic view.

Material properties of the steel are presented in Table 1 and the material properties of the concrete and steel bar are summarized in Table 2. Table 3 lists the specifications of the experimental tests. The tested modulus of elasticity of the concrete and steel are 21 GPa and 210 GPa, respectively.



(a) Setup (b) Details of wall (c) Schematic view

Fig. 1 Experimental tests [6]

Table 1 Steel properties

Section type	Yield stress, f_y (MPa)	Ultimate strength, f_u (MPa)
IPE100 beam flange	308	479
IPE100 beam web	285	446
Fish plate	297	406
Steel plate	268	415
Bolt	900	1000

Table 2 Concrete and steel bar properties

Property	Value (MPa)
Cylinder compressive strength, f'_c	72.5
Cube compressive strength, f_{cu}	79
Yield stress, f_y	336
Ultimate strength, f_u	492
Young's modulus, E_c	21000

Table 3 Specifications of components of experimental tests

Component	Specification
Columns (mm)	2IPE100+2Pl100×5
Beams (mm)	2IPE100
Steel plate thickness (mm)	2
Fish plate (mm)	40×5
Number of bolts	4
Bolt diameter (mm)	6
Rebar diameter (mm)	3
Reinforcement ratio	1
Concrete thickness (mm)	30 (one or both sides of steel plate)
Gap size (mm)	11.25

3. Finite Element Modeling

3.1. Material properties and constitutive models

In this research, concrete was modeled as solid using the concrete damaged plasticity model. The following formula suggested by Carreira and Chu [15] has been used to calculate the compression strain curve of the concrete [10].

$$\sigma_c = \frac{f_c'\gamma(\frac{\varepsilon_c}{\varepsilon_c'})}{\gamma - 1 + (\frac{\varepsilon_c}{\varepsilon_c'})\gamma} \tag{1}$$

where σ_c, ε_c, and f_c' are compressive stress, strain, and cylinder compressive strength of the concrete respectively, and ε_c' is strain corresponding to f_c', and γ is calculated by:

$$\gamma = [\frac{f_c'}{32.4}]^3 + 1.55 \tag{2}$$

(a) Concrete (b) Steel

Fig. 2 Cyclic behavior [10]

The strain ε_c' was chosen as 0.002. The stress-strain behavior of the concrete in compression was assumed to be linearly elastic up to 0.4 f_c'. The plastic strain was considered beyond this region to define the stress-strain relationship of the concrete in modeling. Fig. 2(a) indicates the cyclic behavior of the concrete.

A steel constitutive model was used for the cyclic behavior of the steel. Fig. 2(b) shows the cyclic behavior of the steel under strain-controlled loading schemes. In order to account for progressive hardening and softening effects, the steel was considered to have bilinear kinematic hardening behavior [10, 16]. Yielding of the steel is independent of the equivalent stress because the center of the yield surface moves in the stress space along with the expansion and the contraction of the yield surface range [17,18,19].

3.2. Accuracy of modeling

Two experimental tests of the SCSWs were chosen to demonstrate the accuracy of the modeling, one SCSW with reinforced concrete on one side of the steel plate and the other SCSW with reinforced concrete on both sides of the steel plate. To simulate the SCSWs, all of their specifications were introduced utilizing the finite element software ABAQUS.

4-node shell element S4R was utilized for the steel frame, steel plate, and fish plate. 8-node solid element C3D8R was used for the concrete. The element T3D2 was applied for the reinforcements which are a 2-noded truss element with 3 degrees of freedom at each node. The element B31 was used for the bolts that are a three-dimensional first-degree element with 2 nodes benefiting from a linear interpolation function which has 6 degrees of freedom at each node. The contact surface between components of the SCSWs was defined as Tie. This constraint allows combining two areas with different meshes. However, Embedded Region was considered for the contact surface between the reinforcements and concrete. The displacement method was used for loading. The amount of displacement was applied to the shear walls, according to the loading code [20], as illustrated in Fig. 3. The support conditions of the experimental tests were also simulated for the specimens (Fig. 4).

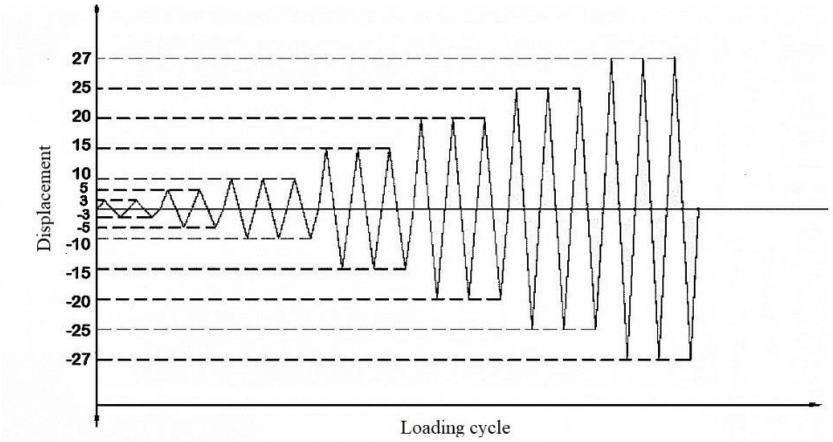


Fig. 3 Displacement history for walls

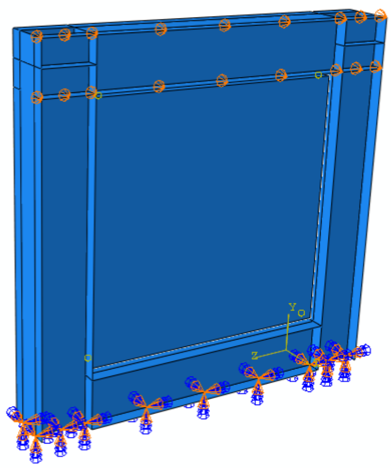
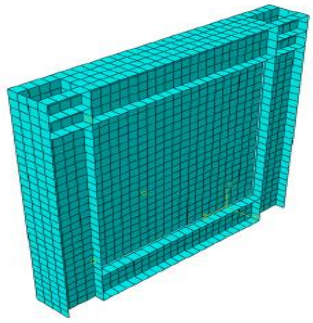


Fig. 4 Simulated SCSW with support conditions

(a) SCSW1



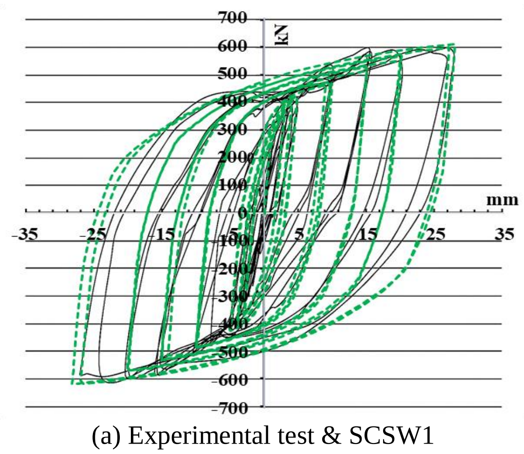
(b) SCSW2

Fig. 5 Simulated models after meshing

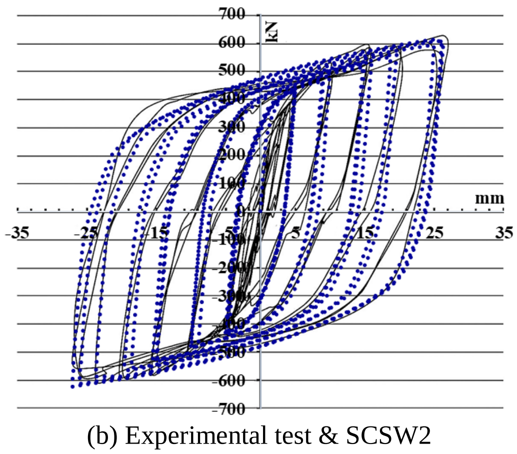
Different finite element mesh sizes were examined for the SCSW1 (wall with reinforced concrete on one side of the steel plate) and SCSW2 (wall with reinforced concrete on both sides of the steel plate) to find reasonable mesh sizes, which could obtain results that were more accurate. Fig. 5 presents the simulated models after meshing which finally led to good results.

Comparisons of the hysteresis curves of the numerical modeling results of SCSW1 and SCSW2 with their corresponding experimental tests results concluded that the obtained ultimate load capacities for the numerical models of SCSW1 and SCSW2 are 606 kN and 608 kN respectively, while they are 595 kN and 630 kN respectively for their corresponding experimental tests (Fig. 6). Accordingly, the differences between the ultimate load capacities of the numerical models and their corresponding experimental tests are only 1.8% and 3.5% respectively for SCSW1 and SCSW2.

In addition, comparing the diagrams obtained for SCSW1 and SCSW2 models with those for their corresponding experimental tests demonstrates that the numerical and experimental diagrams are similar to each other from the behavioral view (Fig. 6).



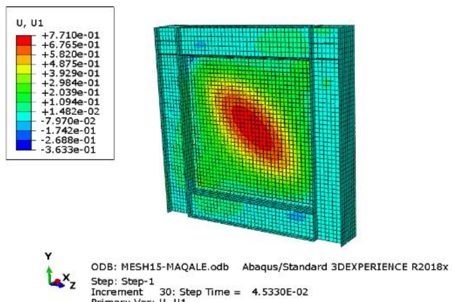
(a) Experimental test & SCSW1



(b) Experimental test & SCSW2

Fig. 6 Hysteresis curves

(a) Experimental test [6]



(b) SCSW1 model

Fig. 7 Failure modes

On the other hand, Fig. 7 illustrates the failure modes of SCSW1 model and its corresponding experimental test. It can be seen from the figure that as the load increased, local buckling in the steel plate occurred which the maximum local buckling was about the center of the walls. Therefore, the figure shows the similarity of the failure modes in the numerical model with its corresponding experimental test.

The aforementioned descriptions regarding the comparisons of the hysteresis curves (ultimate load capacities and behaviors) and failure modes of the numerical models with their corresponding experimental tests uncover the accuracy of the modeling. As a result, the accurate prediction of the performance of the SCSWs is absolutely possible by the proposed three-dimensional finite element modeling in this study.

4. Parametric Study

It was revealed that the proposed modeling was accurate to predict the performance of the SCSWs, consequently, the method was applied for the nonlinear analyses of the SCSW2s with the same size as those experimentally tested. Different variables were adopted to study their effects on the performance of the shear walls. Table 4 summarizes the features of the models based on these variables. In the table, the letters following SCSW2 are the differences in variables of the walls compared with the SCSW2. These letters, like CT, PT, NB, GS, R, BP, and CP designate the variables as concrete thickness, plate thickness, number of bolts, gap size between reinforced concrete and steel frame, the reinforcement percentage in reinforced concrete, and beam and column profiles of the steel frame, respectively.

5. Results and Discussions

Table 4 lists obtained ultimate load capacities of the analyzed SCSW2s. The effects of each variable on the performance of the SCSW2s are also discussed below.

Table 4 Features and obtained ultimate load capacities of the walls

No.	Name	Concrete thickness (mm)	Plate thickness (mm)	Number of bolts	Gap size (mm)	Reinforcement %	Beam profile (mm)	Column profile (mm)	F_{max} (kN)
1	SCSW2	30	2	4	11.25	1	IPE100	IPE100	608
2	SCSW2-CT60	60	2	4	11.25	1	IPE100	IPE100	668
3	SCSW2-CT100	100	2	4	11.25	1	IPE100	IPE100	699
4	SCSW2-PT4	30	4	4	11.25	1	IPE100	IPE100	834
5	SCSW2-PT8	30	8	4	11.25	1	IPE100	IPE100	935
6	SCSW2-NB8	30	2	8	11.25	1	IPE100	IPE100	621
7	SCSW2-NB12	30	2	12	11.25	1	IPE100	IPE100	624
8	SCSW2-GS5	30	2	4	5.625	1	IPE100	IPE100	711
9	SCSW2-GS22	30	2	4	22.5	1	IPE100	IPE100	548
10	SCSW2-R0.50	30	2	4	11.25	0.5	IPE100	IPE100	608
11	SCSW2-R0.25	30	2	4	11.25	0.25	IPE100	IPE100	605
12	SCSW2-BP140-CP140	30	2	4	11.25	1	IPE140	IPE140	1192
13	SCSW2-CP140	30	2	4	11.25	1	IPE100	IPE140	1135
14	SCSW2-BP140	30	2	4	11.25	1	IPE140	IPE100	866

5.1. Effect of concrete thickness

SCSW2s with different concrete thicknesses (30 mm, 60 mm, and 100 mm) were modeled to investigate the effect of the concrete thickness on their performance. Results illustrate that the enhancement of the concrete thickness of SCSW2 from 30 mm to 60 mm (SCSW2-CT60) increases the ultimate load capacity for 9.9%. Also, the increase of the concrete thickness of SCSW2-CT60 from 60 mm to 100 mm (SCSW2-CT100) enhances the ultimate load capacity for 4.6%. Consequently, as the concrete gets thicker, its influence on increasing the ultimate load capacity of the SCSW2s reduces. Obtained hysteresis curves of the SCSW2 models are compared in Fig. 8. As it can be observed from the figure, they have slight differences and similar behavior. Since the areas of the curves are almost the same, it can be concluded that the enhancement of the concrete thickness has little effect on the ductility and energy dissipation of the SCSW2s. Because the major task of the concrete in this kind of shear walls with a gap between the reinforced concrete and steel frame is to delay the out-of-plane buckling of the steel plate in order for the steel plate uses its ultimate strength against lateral loads. The concrete has no significant effect on carrying the lateral loads of the walls. Consequently, the minimum thickness of the concrete can suffice for delaying the buckling of the steel plate.

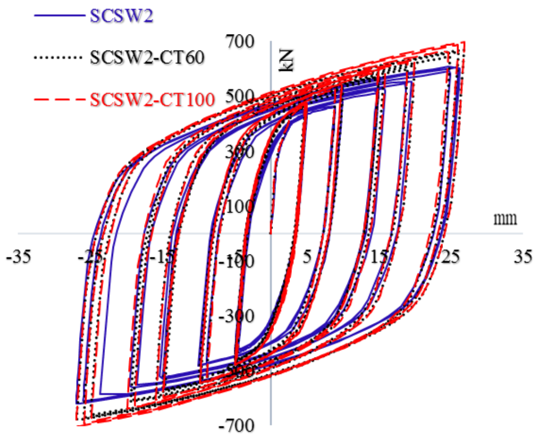


Fig. 8 Effect of concrete thickness

5.2. *Effect of steel plate thickness*

Steel plate thicknesses (2 mm, 4 mm, and 8 mm) have been considered as one of the variables for SCSW2s. According to Table 4, the obtained results from the analyses of the SCSW2s indicate that the increase of the steel plate thickness from 2 mm (SCSW2) to 4 mm (SCSW2-PT4) results in 37.2% enhancement of their ultimate load capacity. However, this thickness increase causes a 4.41 kg increase in the steel plate. If the steel plate thickness is enhanced from 4 mm (SCSW2-PT4) to 8 mm (SCSW2-PT8), the ultimate load capacity of the SCSW2s is increased 12.1% having the increase of the steel plate weight as 8.82 kg.

Fig. 9 illustrates that the increase of the steel plate thickness improves the areas of their load-displacement hysteresis curves. Therefore, it can be concluded that increasing the steel plate thickness leads to the enhancement of the ductility and energy dissipation of the SCSW2s. Since the steel plate has an important role in the ductile behavior of the SCSW2s, increasing the steel plate thickness makes the walls behave more ductile and absorb more energy. Accordingly, the ductility and energy dissipation of the walls are enhanced. Also, using the steel plate thickness of 4 mm results in the optimum value of the ultimate load capacity of the SCSW2s, while more increase of the steel plate thickness mostly increases the weight and cost of the walls.

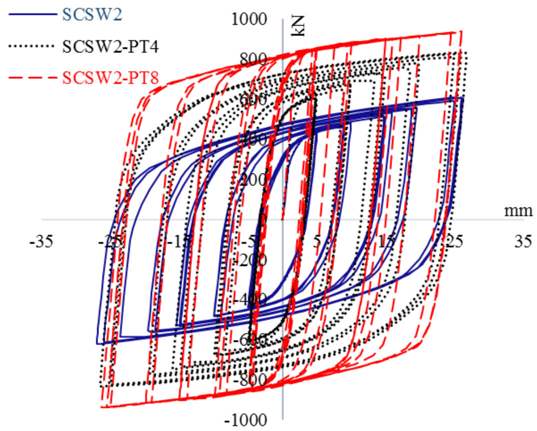


Fig. 9 Effect of steel plate thickness

5.3. Effect of number of bolts

Bolts connecting the reinforced concrete to both sides of the steel plate are one of the studied variables. A number of bolts considered as 4, 8, and 12 in the analyses. Results in Table 4 indicate that enhancing the bolts number of SCSW2 from 4 to 8 (SCSW2-NB8) increases the ultimate load capacity for 2.1%. However, the increase of the bolts number of SCSW2-NB8 from 8 to 12 (SCSW2-NB12) slightly enhances their ultimate load capacity for 0.5%. This inconsiderable effect of increasing the bolts number on the enhancement of the ultimate load capacity is due to the point that because the concrete has no significant role in carrying the large lateral load and its major task is to delay the buckling of the steel plate, then, the minimum number of bolts can be enough because the bolts only attach the concrete to the steel plate.

Fig. 10 shows that obtained curves of the SCSW2s are similar to each other and the areas of their load-displacement hysteresis curves do not significantly change with the increase of the bolts number. As a consequence, it can be concluded that the change of the bolts number does not considerably affect the ductility and energy dissipation of the SCSW2s.

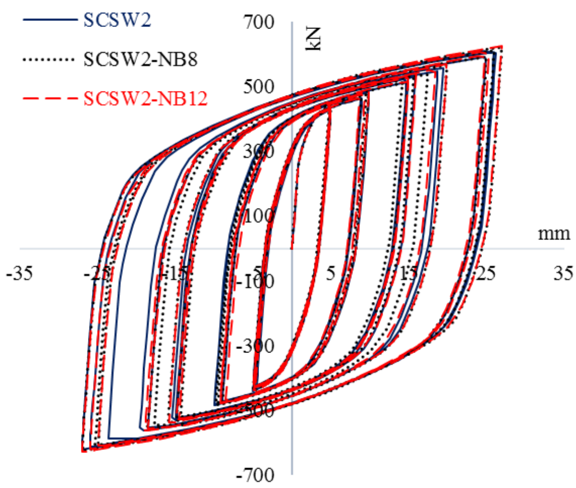


Fig. 10 Effect of bolts number

5.4. Effect of gap size between reinforced concrete and steel frame

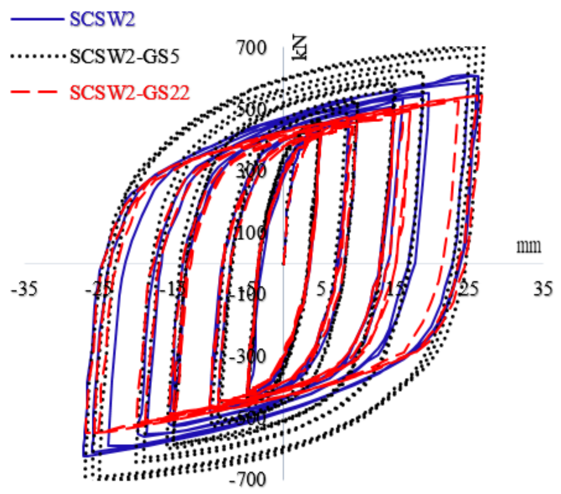


Fig. 11 Effect of gap size between reinforced concrete and steel frame

There is a gap between the reinforced concrete and steel frame in these SCSWs; accordingly, it is studied as one of the variables. Gap sizes of 5.625 mm, 11.25 mm, and 22.5 mm have been analyzed. Results demonstrate that decreasing the gap size from 11.25 mm (SCSW2) to 5.625 mm (SCSW2-GS5) improves the ultimate load capacity for 16.9%. Moreover, reducing the gap size from 22.5 mm (SCSW2-GS22) to 11.25 mm (SCSW2) results in 10.9% enhancement of the ultimate load capacity (Table 4). This is owing to the point that the existence of the gap in these shear walls reduces the damages to the concrete, which delays buckling of the steel plate while the walls carry the loads. Considering the major role of the steel plate in withstanding the lateral loads, the use of the minimum gap (5.625 mm) associates better performance of the walls, while the use of the larger gaps (22.5 mm and 11.25 mm) has a less significant role in increasing the ultimate load capacity due to less contribution of the walls for carrying the loads.

Further, comparing the obtained hysteresis curves in Fig. 11 illustrates that the increase of the gap size reduces the areas of their load-displacement hysteresis curves, which shows the reduction of the ductility and energy dissipation of the SCSW2s.

5.5. Effect of reinforcement percentage

Reinforcement percentages of 0.25, 0.5, and 1 have been taken into account as a variable for the analyses of the SCSW2s. Results indicate that the ultimate load capacity of the walls was not considerably changed by the reduction of the reinforcement percentage of SCSW2 (1%) to those of SCSW2-R0.50 (0.5%) and SCSW2-R0.25 (0.25%), as listed in Table 4. This issue is because of the fact that the major task of the reinforcements in the composite shear walls is to strengthen the reinforced concrete, especially when the reinforced concrete enters its tensile phase. But, since the reinforced concrete is not attached to the steel frame in these SCSW2s and the reinforced concrete only stiffens the steel plate and delays its buckling, the reinforced concrete does not directly contribute to carrying the lateral loads of the walls, therefore, it does not enter its tensile phase. Therefore, the change in reinforcement percentage does not considerably influence the ultimate load capacity of the walls and the minimum reinforcement percentage (0.25%) can be adequate.

Fig. 12 shows that the walls have similar behavior and increasing reinforcement percentage does not significantly change the areas of their load-displacement hysteresis curves. Thus, increasing reinforcement percentage does not considerably improve the ductility and energy dissipation of the walls.

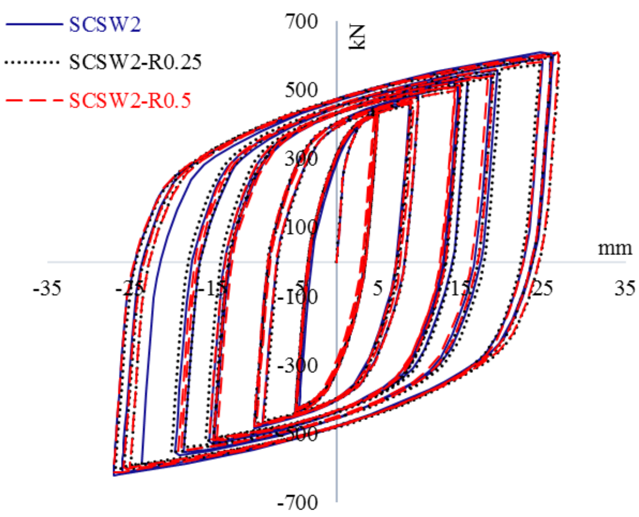


Fig. 12 Effect of reinforcement percentage

5.6. Effect of beam and column profiles of steel frame

Beam and column profiles of the steel frame as IPE100 and IPE140 have been considered in the analysis of the SCSW2s. Comparison of the obtained results in Table 4 uncovers that increasing the beam and column profiles from IPE100 (SCSW2) to IPE140 (SCSW2-BP140-CP140) improves the ultimate load capacity of the walls for 96.1%. However, this improvement of the ultimate load capacity associates with the increase of the wall weight of 14.2 kg and the cost. Also, comparison of the hysteresis curves in Fig. 13 illustrates that increasing the beam and column profiles enhance the ductility and energy dissipation of the walls.

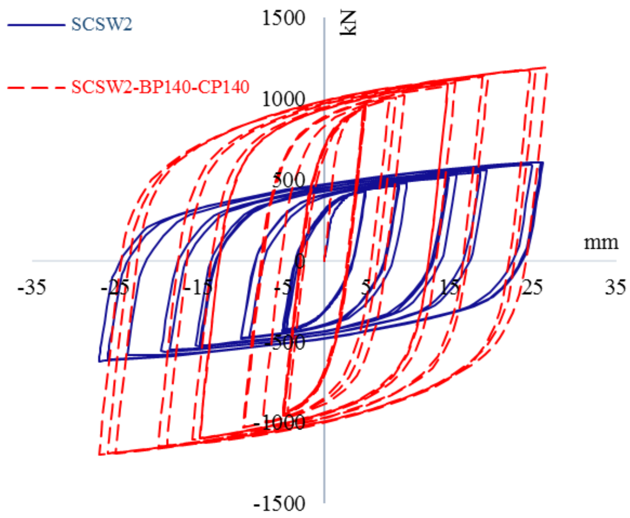


Fig. 13 Effect of beam and column profiles of steel frame

In addition, the effect of beam and column profiles was individually examined to uncover their roles. First, the column profile of SCSW2 was changed to IPE140 as the model SCSW2-CP140. Thereafter, the beam profile of SCSW2 was changed to IPE140 as the model SCSW2-BP140. The results of the analyses show that by changing the column profile of the steel frame from IPE100 to IPE140, the ultimate load capacity of SCSW2-CP140 increases for 86.7% compared with that of SCSW2. On the other hand, the ultimate load capacity of SCSW2-BP140 has been achieved 42.4% higher than that of SCSW2 due to the increase of the beam profile of the steel frame from IPE100 to IPE140 (Table 4). Considering the results obtained from the nonlinear analyses, it can be concluded that enhancing the column profile is more effective on the ductility and energy dissipation of the walls compared with increasing the beam profile (Fig. 14).

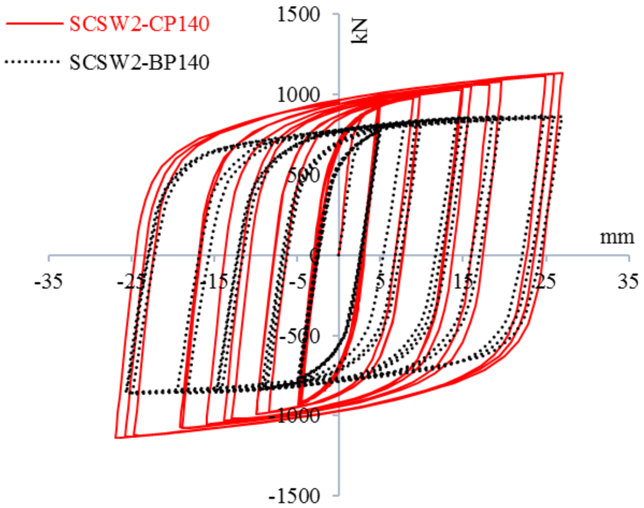
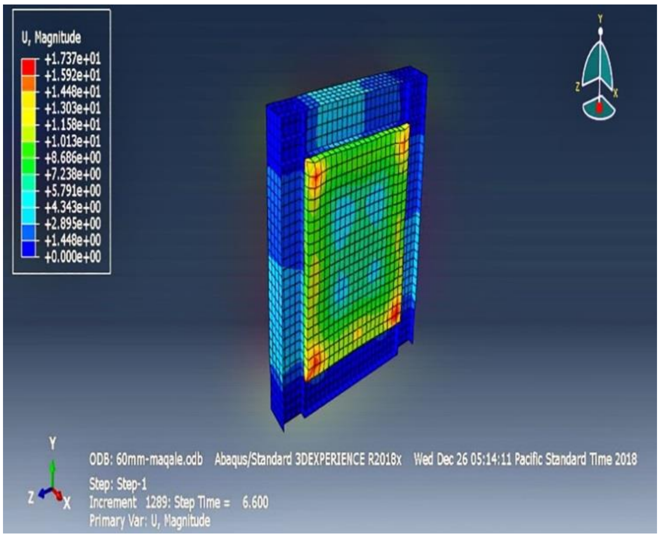


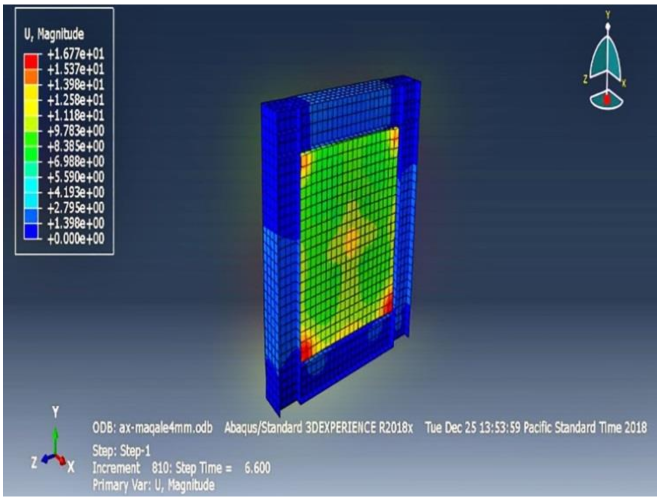
Fig. 14 Effect of beam and column profiles of steel frame

5.7. Buckling of SCSWs

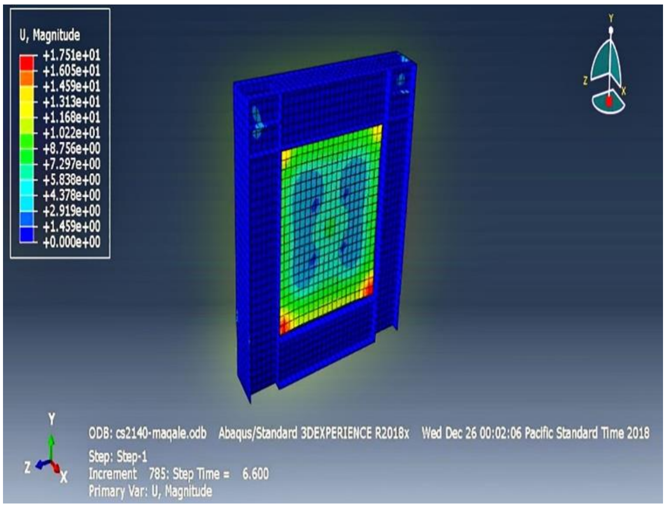
Fig. 15 demonstrates the out-of-plane displacement of the SCSW2s. The occurred maximum displacement is related to the edges of the reinforced concrete, which have been deflected due to coping with the buckling of the steel plate. Because the reinforced concrete on both sides of the steel plate has provided confinement to the steel plate, therefore, the out-of-plane displacement of the steel plate was limited and the buckling did not occur up to the maximum displacement (27 mm) in the steel plate. In other words, the existence of the reinforced concrete on both sides of the steel plate has reduced the out-of-plane displacement of the walls, but it has not considerably influenced the amount of the in-plane-displacement.



(a) SCSW2-CT60



(b) SCSW2-PT4



(c) SCSW2-BP140-CP140

Fig.15 Buckling of SCSW2s

6. Conclusions

In this research, the SCSWs with reinforced concrete on both sides of the steel plate was numerically studied using ABAQUS finite element software. Comparisons of the obtained results from modeling with those of their corresponding experimental tests demonstrated the modeling accuracy. Then, various variables were considered in the nonlinear analyses. Variables were as the thickness of reinforced concrete, thickness of steel plate, number of bolts, gap size between reinforced concrete and steel frame, reinforcement percentage, and beam and column profiles of the steel frame. Results showed that increasing the concrete thickness does not significantly influence the ultimate load capacity of the walls and the minimum thickness would be sufficient. The ultimate load capacity, ductility, and energy dissipation of the walls are enhanced by the increase of the steel plate thickness. Increasing the number of bolts does not have a considerable effect on the ultimate load capacity, ductility, and energy dissipation of the SCSW2s. The increase in the gap size from a certain amount decreases the ultimate load capacity, ductility, and energy dissipation. The increase of the reinforcement percentage does not considerably affect the ultimate load capacity, ductility, and energy dissipation of the walls. Enhancing beam and column profiles of the steel frame increases the ultimate load capacity, energy dissipation, and ductility of the walls, but increasing the column profile of the steel frame is more effective than the beam profile on the behavior of the walls. Moreover, the reinforced concrete on both sides of the steel plate limited the out-of-plane displacement of the steel plate and the buckling did not occur up to the maximum displacement in the steel plate.

Conflicts of Interest

The authors declare no conflict of interest.

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