

Investigation of the Behaviour of Boundary Elements in Mild Steel Shear Walls with an Externally Bonded Plate to One Boundary Element

Amir Azarpour^{1*}, Sajjad Javanmard Zargabad², Faraz Najafi³, Sayyad Karimi Gabalou⁴, Maryam parvaresh⁴

¹PhD, Department of Civil Engineering, Faculty of Engineering, Urmia University, Urmia, Iran

²MSc, Engineering Faculty, Department of Civil Engineering, Urmia branch, Islamic Azad University, Urmia, Iran

³Ph.D. Candidate, Engineering Faculty, Department of Civil Engineering, Urmia branch, Islamic Azad University, Urmia, Iran

⁴Ph.D. Candidate, Department of Civil Engineering, Faculty of Engineering, University of Mohaghegh Ardabili, Ardabil, Iran

*Corresponding author

Abstract

Recently, steel shear walls have been utilized as lateral load-resisting systems due to their good seismic performance in the structure as retrofitting and constructions of present ones. Steel shear walls are usually set up in three different ways: with stiffener, without stiffener, and composite, which the second type is more common. A stiffened steel shear wall contains a steel plate surrounded by columns and beams in which the filling plate is completely connected to the boundary elements. In this study, several specimens with different lengths whose inner plates were fully connected to the boundary elements were analyzed in ABAQUS. Then, the steel shear wall's behaviour with partial length connection was assessed by connecting the plate to one of the boundary members. The outcomes endorse that initial stiffness and the base shear of steel shear walls with partial-length connections are less than those of steel shear walls with full connections. Furthermore, the plate yield stated board in steel shear walls with partial length connection was lower than the amount observed in steel shear walls with full connections. In addition, higher axial forces were applied to the columns in the cases of full connections compared to the other specimens. In the CF cases, the axially compressive force applied to the columns was less than that in the BF mode. In addition, the less bending moment is applied to the BF specimens. It is also concluded that the ductility factor of models with full connection is higher than that of partial-length connection specimens.

Keywords: Boundary Elements, Strength, Steel Shear Wall, Beam, Column, Stiffness.

1. Introduction

Nowadays, the reputation of Steel Plate Shear Walls (SPSWs) has been considerably enhanced. SPSW (as a structural system) supplies adequate lateral resistance through strength, sufficient ductility, and stiffness. Steel shear walls are constructed with or without stiffener. At the beginning of the use of shear walls, steel plates were used with stiffeners to prevent the buckling of the plate; however, researchers nowadays recommend using thin SPSWs deprived of stiffeners. Initially, the ultimate limit state design of the steel shear walls was based on the prevention of filler plates' out-of-plane buckling. This led to the heavier stiffener sheets' design, and consequently, it is not a cost-effective design. Based on Basler's studies on the web plate of girders and the concept of post-buckling resistance due to the plates' diagonal tension, the 212 steel shear walls' utilization without stiffeners was considered [1].

The first SPSW construction instance, the Shinjuku Nomura Building, was built in 1978 as 3rd tallest structure in Tokyo with 693 ft height and fifty-one floors. The SPSW system contained ten-foot high by 16.5-foot-long steel panels and strengthening stiffeners in the vertical and horizontal axes.

The computational relations for the diagonal tension of the plates were 1st obtained in 1997 and were recognized with the experiments' results done by Timler and Kulak this year too [2]. Research for assessing the strength, hysteresis manner, and ductility of steel shear walls non-stiffeners by Timmler and Kulak demonstrates their remarkable capability of energy absorption and economic benefits [3].

Darren Vian and Michel Bruneau in an investigational program of steel panel shear walls verified specimens applied LYS (low yield strength) steel infill panels and RBS (reduced beam sections) at the beam-ends.

In steel shear walls without stiffeners, the web plate is usually associated with the border elements on all four sides. However, in a different case, the web plate can only be connected to one member of the boundary elements in a way that there will be no connection between other boundary elements and the plate. Precisely, the web plate is connected only to the beams and there is no connection between the columns and the plate [4]. Choi and Park tested a steel shear wall wherein the plate was just connected to the beam. The plates were welded to the top and bottom beams and the columns had no connection to the web plate. Choi and Park tested the FSPW2 steel shear wall to compare the steel shear wall's manner wherein the plate is related to both border elements namely beams and columns. It should be noted that the mechanical properties and geometrical dimensions of the border elements and web plates in both FSPW4 and FSPW2 specimens are quite similar and the only difference is how to be connected the boundary elements. It was concluded that the FSPW4 and FSPW2 specimens had the same initial stiffness, but the FSPW4 specimen's ultimate strength in which the plate was connected only to the beams, was less than that of the FSPW2 specimen. The energy dissipation of the FSPW4 was also reported about 65% of that of the FSPW2 specimen [5]. Guo et al. investigated steel shear walls linked only to beams. They tested two specimens. The only difference between the two specimens was that one of the specimens had stiffeners on the edges; however, the presence of these stiffeners had no impact on the strength and ductility. That is because the tensile field appeared in the specimens to resist the lateral loads so the stiffeners had no apparent effect on the occurrence of these tensile fields [6]. Jahanpour et al scrutinized the collaboration among the surrounding frame and wall plate experimentally for distinctive semi-supported steel shear walls (SSSW) systems wherein the wall frame has a bending foremost manner. The results of the experimental study show that the frame can enhance a tension field in the wall plate to the wall plate that has been yielded before the frame [7]. Saeid Sabouri-Ghomi and Seyed Ramin Asad Sajjadi considered 2 one-story the same SPSWs without and with stiffeners, and one of their surrounding frames was verified and the manner of them was considered. The outcomes indicated that the stiffeners' installation enhanced the SPSWs' behavior. It made a 26% rise in energy dissipation volume and a 51.1% surge in the steel plate's shear stiffness whilst its impact on the steel plate shear strength was too small [8]. Meng Wang A, Yongjiu Shi b, Jian Xu c, Weiguo Yang A, and Yixin Li scrutinized unstiffened thin steel plate shear wall structure's seismic behaviors, and investigations of 4 three-story unstiffened steel plate shear wall specimens under cyclic loads were done. The numerical and experimental outcomes presented that this type of construction reveals great strength, the best energy dissipation volume, and the best ductility [9]. Shekastehband et. al did experimental and numerical research on the seismic behavior of LYS steel plates and high yield strength (HYS) SPSWs where the plates were related only to the beam. They verified 4 specimens, 2 of which had LYS steel plates and the other 2 had HYS steel plates. Regarding the hysteresis curves, they observed severe pinching in the specimens, which could mainly occur due to out-of-plane buckling and tensile performance, along with the boundary elements' flexibility caused by the rotational capacity of the connections. As seen, the strengths of the specimens decreased due to the presence of the openings. The specimens with high-strength plates dissipated more energy due to the larger under-curve area [10]. Additionally, Shekastehband et al. did an experimental and numerical pursuit on partial-length connection steel shear walls' behavior [11]. Additionally, AlHamaydeh, M.; Sagher, and AlHamaydeh, M.; Elayyan, L. scrutinized the SPSW's behavior by the diverse main factors [12,13]. A. Farahbakhshooli and Anjan K. Bhowmick offered a novel component strength deterioration model for stiffened infill plates to assess stiffened steel seismic performance with the FEMA P695 method. The results indicate that stiffened SPSWs' capacity is more sensitive to ductility capacity modifications [14]. Zaigen Mu and Yuqing Yang searched the impact of frame-to-plate connections and oblique

channel-shaped stiffeners on SPSWs' seismic behavior, cyclic quasi-static investigations were done on 2 one-bay two-story specimens. Consequently, this type of SPSW must regard the extra stiffeners' impact on the boundary elements. The link part's rational design allows the frame to have sufficient strength to protect the stiffened plate in addition to evading early yielding at the link [15]. Farahbakhshtooli and Anjan K. Bhowmick conducted a complete examination of Special Plate Shear Walls (SPSW) featuring regularly spaced circular perforations, a recent innovation in structural design. A robust macro-model was developed specifically for these perforations. The analysis revealed a subtle reduction in the median shear envelopes for P-SPSWs (Perforated Special Plate Shear Walls) compared to their solid counterparts. Notably, when contrasted with solid SPSW archetypes, the significance of boundary columns in resisting story shear was observed to surpass that of the infill plate across the majority of stories in P-SPSW archetypes. Finally, a sensitivity assessment was undertaken to assess the impact of perforation diameter on various responses of P-SPSWs, including peak column axial forces, peak column bending moments, and maximum interstorey drift [16]. Chen Wang et al studied built-in constitutive models in commercial FE (finite element) packages and concentrated on SPSWs using a certain kind of structure formation, therefore yielding insufficient precision and insufficient validation. The outcomes specify that the axial forces could stabilize the tensile forces from the tension fields; however, meanwhile could aggravate the compression-side column's compressive burden, which makes the column yield or buckle. Once the axial force rate surges, the adversative influence slowly overcomes the useful impact and especially reduces the loading volumes [17]. Hao Jun Sun, et al offered the global and local buckling manner of T-CSPSWs' load-bearing capacity (trapezoidal corrugated SPSWs) and S-CSPSWs (sinusoidally corrugated SPSWs) under pure shear. On the basis of broad numerical outcomes, the elastic global and local buckling loads are calculated for S-CSPSWs and T-CSPSWs under shear loads, in turn. Consequently, the flat folds' or curved sub-panels normalized width-to-thickness proportions, and the CSPSWs' normalized slenderness proportions are accompanied [18]. Elkafrawy, M et al. and AlHamaydeh, M. et al found that the stiffness and resistance of system occupations provide fertile ground for the structural engineers to apply regions including moderate-length, high, and mid-rise structures. The thin plates' plan is usually organized by their buckling behaviour. A major issue for engineers designing the SPSWs is buckling [19,20]. Whilst Hou et al, Zhao et al, Hou et al, and Sun et al provide the mathematical models for the SPSWs' buckling design [18,21].

In this research, several one-story and 1-span conventional steel shear walls were considered and selected using the LRFD method according to AISC341 Guideline No. 20. Since then, three different scenarios have been considered. The 1st case is that the distance between the web plate of the studied steel shear wall and the beam is 2% of the story height and is connected only to the columns. In the latter scenario, the distance between the web plate of the steel shear wall and the above and below columns is 2% of the story width and is connected only to the beams. The 3rd case is that the plate is connected to both beams and columns. The mechanical properties and the geometrical dimensions of the web plate and the boundary elements in the above three scenarios are the same. A four-node, quadrilateral, stress/displacement shell element with decreased integration and a large-strain formulation (S4R) was employed to model the boundary elements and steel plates in ABAQUS. Every node of this component has 6 degrees of freedom counting 3 degrees of rotational freedom and 3 degrees of translational freedom. The buckling possibility in the analysis of the selected models was produced by utilizing a first limitation along with the 1st buckling mode. The columns' out-of-plane deformations are also prevented to avoid torsional effects. The Von Mises criterion was utilized as the failure criterion for the analyses. The geometric and material nonlinearity was also considered to predict the behavior of the structure.

2. Verification

To validate the results, the FSPW4 experimental specimen (Fig. 1) was simulated in ABAQUS software. This specimen had been previously verified by Choi and Park at the National University of South Korea [5]. The plates in the specimen were connected only to beams, without any connection to the adjacent columns. The schematic diagram of the FSPW4 specimen is shown in Fig. 1.

In the experimental model, H-200 × 200 × 16 × 16 sections were used for columns, while H-150 × 100 × 12 × 20 sections were utilized for the beams of the first and second stories. H-250 × 150 × 12 × 20 sections were used for the upper story beam. The plate thickness was 4 mm across all stories. The connections between columns and

beams were fixed. SM400 steel with a yield stress of 240 MPa was used for the web plates, and SM490 steel with a yield stress of 330 MPa was used for the boundary elements.

The hysteresis curve obtained from the experiment, along with the test results of the FSPW4 specimen, is displayed in Fig. 2. It is evident that there is a good agreement between the experimental outcomes and the finite element model (FEM). The maximum strengths of the experimental specimen and the FEM specimen of FSPW4 were 1425 kN and 1317 kN, respectively. The difference between the finite element model and the experimental specimen in predicting the maximum strength is 7.57%, which is considered acceptable. The ABAQUS FE software also accurately predicts the deformation of the specimen. Fig. 1 compares the deformed shape of the experimental specimen and the FEM of FSPW4 at the end of loading.

The experimental specimen utilized in this study is based on the experiment conducted by Choi and Park [5]. Fig. 2 presents a schematic diagram of the FSPW3 specimen. A one-span, three-story frame was subjected to cyclic loading during the experiment. The inner length of the columns and the inner height of the beams in each story are 2200 mm and 1000 mm, respectively. The plate thickness is 4 mm, and the columns are made of H 150 × 150 × 8 × 20 sections. The beams of the first and second stories are made of H 150 × 100 × 12 × 20 sections, and the upper story beam is made of H 250 × 150 × 12 × 20 sections. SS490 steel was used for the boundary elements, while SS400 steel was used for the web plate [5].

The FSPW3 experimental specimen and the FEM's strength were 1565 kN and 1582 kN, respectively. Thus, the ultimate strength difference between the finite element and experimental models for FSPW3 is 1.08%. Fig. 2 illustrates the deformation of both the experimental and finite element models of FSPW3 at the end of the loading. According to Choi and Park, the failure in FSPW3 initially occurred due to column suction caused by the plate post-buckling field on the first story, followed by the failure of the column base plate connection. The FEM and the experimental specimen's deformations are very similar, with the plastic hinge formation in the experimental model being closely reproduced in the FEM.

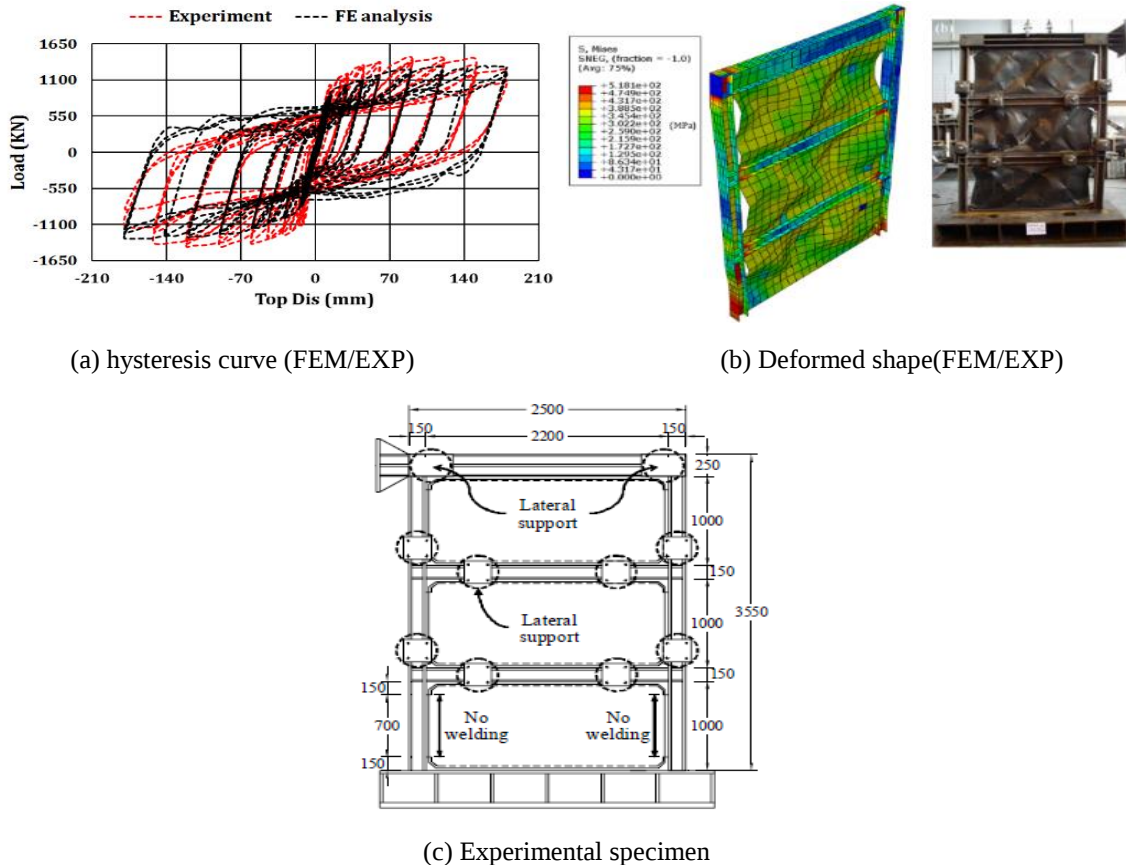


Fig. 1. schematic diagram of FSPW4 experimental specimen [5], hysteresis curve (FEM) and deformation shape

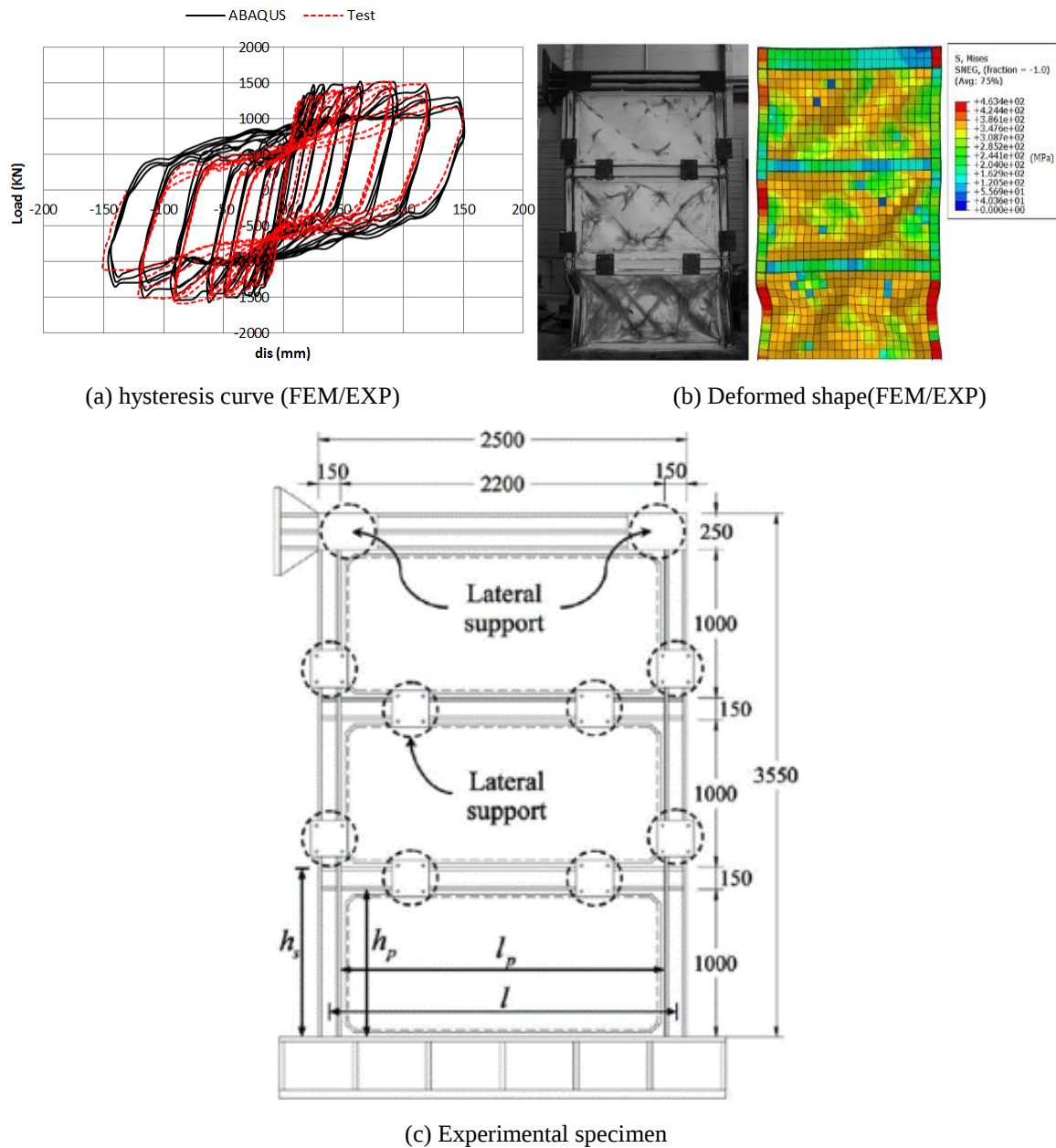


Fig. 2. schematic diagram of FSPW3 experimental specimen [5], hysteresis curve (FEM) and deformation shape

3. Introducing Research Models and Methodology

Note that LYP100 steel, with a yield strength of 100 MPa, was used for the web plates. This type of steel is known for its low yield point, making it suitable for applications where significant ductility and energy absorption are required. On the other hand, St37 steel, with a yield strength of 240 MPa, was utilized for the boundary elements, including both the columns and beams. St37 steel is a commonly used structural steel that offers a balance of strength and ductility, making it ideal for load-bearing elements.

To thoroughly investigate the behavior of these materials in steel shear walls, the specimens were divided into four groups, each with different lengths. These variations in length were designed to assess how the size of the shear walls influences their structural performance. Specifically, the center-to-center distances of the columns in these specimens were set at 4500 mm, 5500 mm, 6500 mm, and 7500 mm, respectively. These distances represent a range of typical spans found in practical engineering applications.

All specimens maintained a consistent height of 3200 mm, which is a standard dimension for many structural systems, ensuring that the height factor remained constant across all tests. This consistency allowed for a focused study on the impact of the varying lengths alone.

The detailed geometrical properties of the studied specimens, including the lengths, heights, and cross-sectional dimensions of the columns and beams, are indicated in

Table 1.

Table 1 provides a comprehensive overview of the physical characteristics of each specimen, facilitating a clear understanding of their structural configurations and aiding in the interpretation of the experimental results.

Table 1. geometric features of the specimens considered in this research

Model	Width (mm)	height (mm)	plate thickness (mm)	Beam	Column
A	4500	3200	4.2	H 400-250-40-30	Box 350×350×32×32
B	5500	3200	4.2	H 400-250-40-30	Box 350×350×32×32
C	6500	3200	4.2	H 400-250-40-30	Box 350×350×32×32
D	7500	3200	4.2	H 400-250-40-30	Box 350×350×32×32

4. Investigating the Specimens' Behaviour

For To investigate the behavior of steel shear walls, hysteresis analysis was conducted using the ATC24 loading protocol. This protocol is widely recognized for evaluating the cyclic performance of structural components under simulated seismic loading conditions.

In this study, the term Column-Free (CF) refers to the configuration where the plates are connected to the columns while the beams remain unconnected or free. Conversely, the term Beam-Free (BF) describes the configuration where the plates are connected to the beams, leaving the columns unconnected or free. The "Full Connection (Full)" model represents a configuration where the plates are fully connected to both boundary elements, i.e., both the beams and the columns.

These different connection configurations were chosen to assess the impact of varying levels of connectivity on the structural behavior of the steel shear walls. By comparing the CF, BF, and Full models, the study aims to determine how the connections influence the initial stiffness, base shear, ductility, and overall seismic performance of the shear walls.

The detailed analysis provided by these configurations helps in understanding the optimal design and retrofitting strategies for steel shear walls to enhance their seismic performance and ensure the safety and resilience of structures.

4.1. Shear Capacity

The specimens' shear capacities with and without openings in both loading directions are represented in

Table 2 and Table 3, respectively. According to the base shear values, it is evident that when the plate is connected only to the beam or column, the base shear is reduced compared to the full connection configuration. Additionally, the base shear of the CF specimen is higher than that of the BF specimen.

Table 4 presents the proportions of the maximum base shear of specimens with partial connections compared to those with full connections, without openings. According to

Table 4, the average base shear of CF specimens is 17.25% lower than that of full connection specimens. Furthermore, the average base shear of BF specimens is 20.64% lower than that of full connection specimens.

Table 5 shows the proportion of the maximum base shear of specimens with partial connections compared to those with full connections, with openings. According to Table 5, the average base shear of CF specimens is 15.16%

lower than that of full connection specimens. Additionally, the average base shear of BF specimens is 14.45% lower than that of full connection specimens.

From

Table 4 and Table 5, it is apparent that the reduction in base shear capacity for specimens with partial connections is less significant when compared to full connection specimens, even in the presence of openings. This indicates that the presence of openings affects the shear capacity of steel shear walls, but the impact is more pronounced in fully connected configurations compared to partially connected ones.

Table 2. Amounts of shear capacity of studied specimens with opening

Model	Full connection		CF		BF	
	+	-	+	-	+	-
A	2141.84	2168.99	1827.73	1841.49	1910.78	1927.85
B	2219.19	2278.79	1909.13	1922.45	1946.84	1963.21
C	2355.22	2424.31	2025.60	2046.27	1994.17	2036.99
D	2408.39	2472.14	2116.36	2119.33	2024.36	2054.98

Table 3. base shear values of the studied specimens without opening

Model	Full connection		CF		BF	
	+	-	+	-	+	-
A	2417.92	2457.93	2022.29	2024.25	2060.29	2094.00
B	2663.12	2686.61	2166.96	2196.89	2113.36	2149
C	2813.98	2878.99	2392.46	2411.87	2119.17	2183.97
D	2880.77	2936.06	2440.96	2424.38	2193.08	2244.32

Table 4. Base shear ratio comparison between specimens with partial and full connection (without opening)

Model	CF / full connection	BF / full connection
A	-17.65	-14.81
B	-18.23	-20.02
C	-16.23	-24.15
D	-16.87	-23.56

Table 5. Base shear ratio comparison between specimens with partial and full connection (with opening)

Model	CF / full connection	BF / full connection
A	-15.1	11.12
B	-15.64	13.85
C	-15.6	15.98
D	-14.28	16.88

4.2. Initial Stiffness

In this section, the initial stiffness of the studied samples has been investigated. As indicated by the values in Table 6 and Table 7, the initial stiffness of samples with full connection is higher than that of the corresponding partially connected samples. Consequently, it is determined that if the complete connection between the boundary members and the inner sheet is not established, the system's initial stiffness will decrease significantly.

The values in Table 6 show that the initial stiffness of samples connected only to the beam or column, with and without openings, is lower than that of the fully connected samples. On average, the initial stiffness of samples connected to the column, with and without openings, is lower than that of fully connected samples by 37.6% and 27.7%, respectively. Similarly, the average resistance of samples connected to the beam, with and without openings, is lower than that of fully connected samples by 50.45% and 51.5%, respectively.

Additionally, the average strength of samples connected to the beam, with and without openings, is lower than that of samples connected to the column by 11.3% and 31.8%, respectively. This indicates that the type of partial connection (whether to the beam or the column) significantly influences the initial stiffness and overall structural performance of the shear walls.

In summary, the findings demonstrate that full connections between the boundary members and the inner sheet result in higher initial stiffness and strength. Partial connections, whether to beams or columns, lead to a notable reduction in these properties, with the extent of the reduction varying depending on the type of connection and the presence of openings.

Table 6. Amounts of initial stiffness for studied specimens without opening

Model	Full connection	CF	BF
A	2494.50	1590.52	1418.54
B	2876.94	2052.62	1563.32
C	3265.25	2417.47	1423.62
D	3443.94	2764.1	1347.84

Table 7. Amounts of initial stiffness for considered specimens with openings

Model	Full connection	CF	BF
A	1751.57	1054.55	987.65
B	1898.45	1116.01	1080.24
C	2087.87	1298.65	1141.59
D	2336.95	1591.06	1215.28

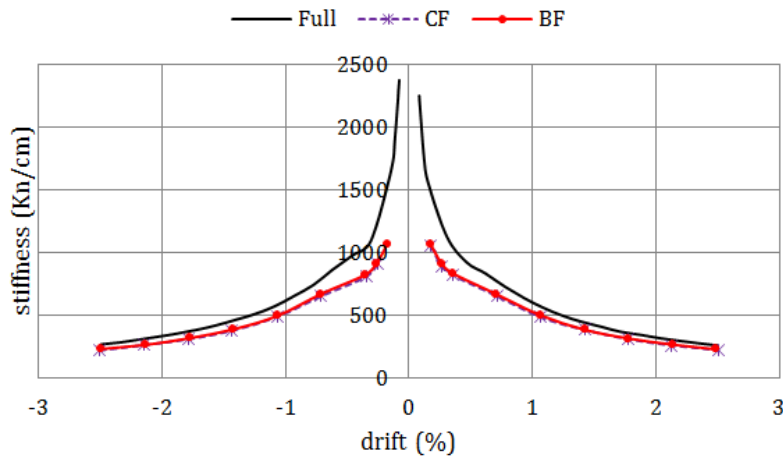


Fig. 3 . Comparison of stiffness of specimens during loading for group A specimens with opening

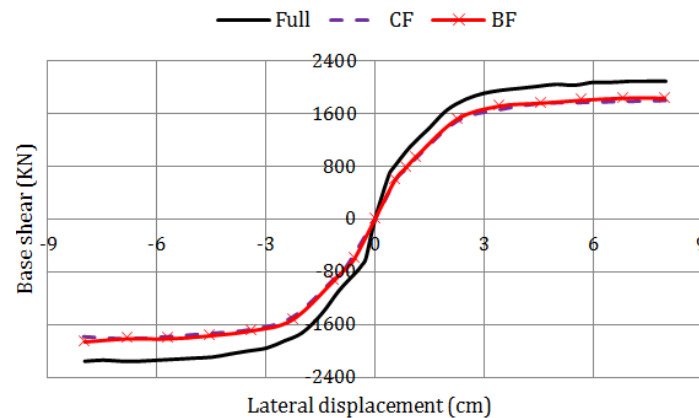


Fig. 4. Comparison of the skeleton curves of group A specimens with opening

4.3. The Forces Affecting Boundary Elements

4.3.1 Axial Force of the Columns

In this study, three different modes were used to understand how the forces affecting the boundary elements vary based on the type of plate connection. Specifically, the axial compressive force exerted on the columns was investigated, and its backbone curve was analyzed.

Fig. 4 presents the axial force curves for the studied columns. According to Fig. 5, the maximum axial compressive force occurs at the lower level of the columns. The compressive axial force in fully connected specimens is larger than that in CF and BF specimens. For specimens with openings, the maximum axial forces in BF samples are larger than those in CF samples because the columns in CF specimens are not connected to the plates, leading to less pressure on them.

For specimens without openings in groups A and B, the maximum axial force in BF cases is larger than that in CF specimens. However, the axial force in the CF case becomes higher than in the BF case as we consider the height distribution. In the CF specimens, the axial compressive force is approximately constant along the lower two-thirds of the column height and then decreases as the height increases. In contrast, the compressive axial force in full connection and BF specimens decreases steadily from the lower level of the column height.

The presence of openings was found to reduce the axial compressive force in the columns. Table 8 shows the maximum values of compressive axial force for the columns in specimens without openings. The reduction in compressive axial force in specimens with partial connections compared to fully connected specimens is also presented in this table. According to Table 8, the maximum axial compressive force along the column height for fully connected specimens is larger than that for partially connected specimens. The highest difference in axial force between fully connected and CF specimens is 36.02%, occurring in group B. Similarly, the highest difference between fully connected and BF specimens is 34.7%, occurring in group C. Overall, the axial force in fully connected columns is 33.03% higher than in CF specimens and 29.48% higher than in BF specimens.

Table 9 shows the maximum axial compressive force for the columns in specimens with openings. According to Table 9, the maximum axial compressive force along the column height for fully connected specimens is again larger than that for CF and BF specimens. The greatest difference in axial force between fully connected and CF specimens is 35.05%, occurring in group C, while the highest difference between fully connected and BF specimens is 31.37%, also in group C. On average, the axial force in fully connected columns is 32.55% higher than in CF specimens and 25.18% higher than in BF specimens.

These findings illustrate those full connections between boundary elements and the inner sheet result in significantly higher axial compressive forces. Partial connections lead to notable reductions in these forces, with the type of connection (CF or BF) and the presence of openings further influencing the results.

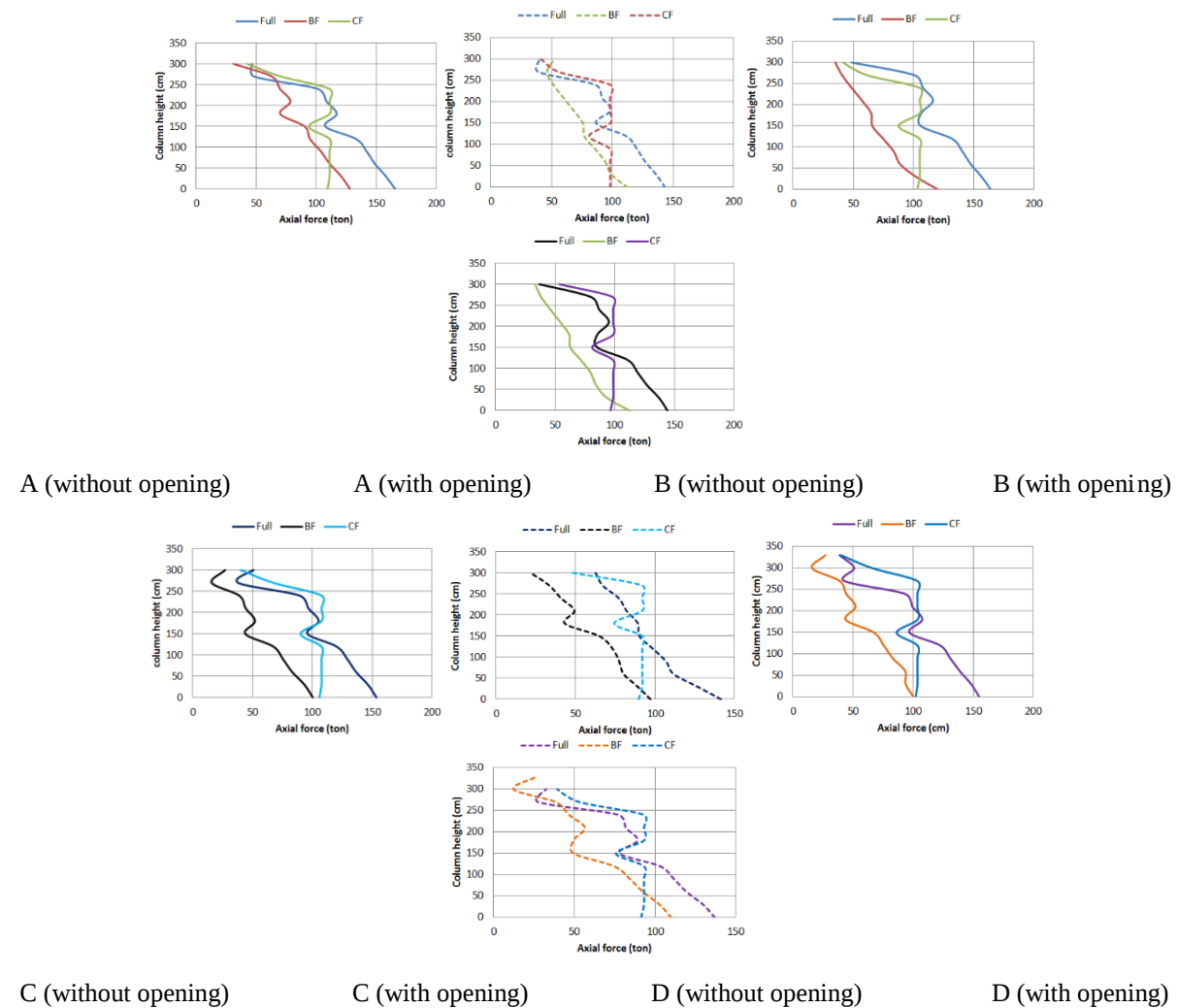


Fig. 5. Axial force curve for the column of specimens

Table 8. The max compressive axial force's amounts for the column of specimens without opening

Model	State	The column's max axial force (KN)	The percentage reduction of the compressive axial force of the CF specimen compared to the full connection specimen	The percentage reduction of the compressive axial force of the BF specimen compared to the full connection specimen
A	Full	165.5	32.95	22.7
	CF	111.0		
	BF	127.9		
B	Full	164.1	36.02	22.68
	CF	105.0		
	BF	120.3		
C	Full	153.5	29.95	34.7
	CF	107.5		
	BF	100.3		
D	Full	155.0	33.22	33.84
	CF	103.5		
	BF	102.5		

Table 9. The max compressive axial force's amounts for the column of specimens with openings

Model	State	The maximum axial force of the column (KN)	The percentage reduction of the compressive axial force of the CF specimen compared to the full connection specimen	The percentage reduction of the compressive axial force of the BF specimen compared to the full connection specimen
A	Full	143.2	31.5	21.52
	CF	98.1		
	BF	112.4		
B	Full	144.1	31.55	22.18
	CF	98.6		
	BF	112.1		
C	Full	141.3	35.05	31.37
	CF	91.8		
	BF	97.0		
D	Full	136.9	32.05	25.68
	CF	93.0		
	BF	101.8		

4.3.2 Comparison of the Axial Compressive Force of the Columns in Specimens with Partial Connection

In Table 10, the axial compressive forces for the columns of specimens with a partial connection without openings are displayed. As seen, the column's axial force in CF specimens is 13.26% and 12.72% lower compared to BF specimens in groups A and B, respectively. However, as the span of the frame increases in groups C and D, the axial force of the columns in CF specimens is 7.22% and 1% higher than that in BF ones, respectively.

In Table 11, the axial compressive forces for the columns of specimens with a partial connection with openings are provided. As shown, in all groups, the axial force of the columns in CF specimens is less than that in BF ones. The percentage decrease in the axial force of the column for groups A, B, C, and D is 12.72%, 12.32%, 5.35%, and 8.57%, respectively.

These results indicate that for specimens without openings, the axial force in CF specimens tends to be lower than in BF specimens in shorter spans (groups A and B). However, as the span increases (groups C and D), the trend reverses, with CF specimens exhibiting slightly higher axial forces than BF specimens. For specimens with openings, CF specimens consistently show lower axial forces compared to BF specimens across all groups, though the percentage of the decrease varies. This demonstrates that the presence of openings has a significant impact on the distribution of axial compressive forces, with CF configurations generally experiencing less force compared to BF configurations.

Table 10. Comparison of the axial compressive force for the column of specimens with a partial connection without opening

Model	State	The column's max axial force (KN)	Axial force of CF / BF specimen
A	CF	110.98	13.26
	BF	127.94	
B	CF	105.01	12.72
	BF	120.32	
C	CF	107.51	+7.22
	BF	100.27	

D	CF	103.51	+1
	BF	102.53	

Table 11. Comparison of the axial compressive force for the column of specimens with partial connection with the opening

Model	State	The column's max axial force (KN)	The percentage changes of compressive axial force of CF specimen compared to BF specimen
A	CF	98.144	12.72
	BF	112.45	
B	CF	98.64	12.32
	BF	112.5	
C	CF	91.83	5.35
	BF	97.02	
D	CF	93.08	8.57
	BF	101.8	

4.3.3. Effect of Openings on the Axial Compressive Force of the Columns

Table 12 illustrates the reduction in the axial force values of the columns in the cases with openings compared to the specimens without openings. According to the values presented in Table 12, it is evident that the axial compressive force of the columns is generally lower in samples containing openings than in samples without openings.

The reduction in axial force varies between 0.7% and 14.5%, indicating that the presence of openings consistently reduces the axial compressive force experienced by the columns. This reduction highlights the impact of openings on the structural integrity and load distribution of the steel shear walls, as openings tend to weaken the overall stiffness and strength, leading to lower axial force values in the columns.

Table 12. Investigating the effect of the opening on the axial compressive force of the boundary column of the steel shear wall

Model	Connection conditions	The max axial force of column without opening (KN)	The max axial force of column with the opening (KN)	Reduction of compressive axial force (%)
A	Full	165.51	143.29	13.4
	CF	110.98	98.144	11.5
	BF	127.94	112.45	12.1
B	Full	164.11	144.12	12.1
	CF	105.01	98.64	6.0
	BF	120.32	112.15	6.8
C	Full	153.473	141.387	7.8
	CF	107.513	91.83	14.5
	BF	100.273	97.02	3.25
D	Full	154.98	136.99	11.6
	CF	103.507	93.08	10.1
	BF	102.529	101.8	0.7

4.3.4. Effect of Openings on Shear Capacity

Effect The effect of openings on the shear capacity of the steel shear walls is presented in Table 13. It is clear that the presence of openings has led to a decrease in shear capacity, with reductions varying between 6.7% and 15.8%. On average, the maximum resistance in samples with a full connection containing openings is 14.5% less than that of samples without openings.

For the samples connected to the column (CF), the addition of openings has led to a decrease of 12.4% in the maximum lateral resistance. In the samples connected to the beam (BF), the presence of openings has resulted in an average reduction of 7.9% in maximum strength. These findings indicate that openings significantly affect the shear capacity of steel shear walls, with the extent of reduction depending on the type of connection. Full connection samples experience the greatest reduction in shear capacity due to openings, followed by CF and BF samples. This information is crucial for the design and analysis of steel shear walls in buildings, particularly when considering the inclusion of openings for functional purposes.

Table 13. Investigating the effect of the opening on the shear capacity of the steel shear wall

Model	Connection conditions	Without opening (KN)	With opening (KN)	Percentage reduction in shear capacity
A	Full	2457.929	2168.996	11.7
	CF	2024.249	1841.493	9.02
	BF	2094.004	1927.85	7.9
B	Full	2686.61	2278.79	15.1
	CF	2196.89	1922.45	12.4
	BF	2149	1963.21	8.6
C	Full	2878.99	2424.31	15.7
	CF	2411.87	2046.27	15.1
	BF	2183.97	2036.99	6.7
D	Full	2936.06	2472.139	15.8
	CF	2440.963	2119.325	13.1
	BF	2244.32	2054.984	8.4

4.3.5. Bending Moment of the Beam

In Fig. 5, the bending moment curve of the beam is shown across the beam length in specimens with and without openings. These graphs reveal several key insights:

1. **Consistency in Beginning and End Regions:** The bending moment at the beginning and end regions of the beams is consistent across different samples, indicating similar stress distributions at these points regardless of the connection type or presence of openings.
2. **Lower Bending Moment in Beam-Only Connections:** In samples where the plate is connected only to the beam (BF samples), the bending moment is significantly lower, especially in the middle regions of the beam. This suggests that the beam-only connection does not effectively transfer loads to the beams, resulting in reduced stress.
3. **Higher Bending Moment in Fully Connected Samples:** Except for the beginning and end regions, the bending moment in fully connected samples is higher than in the other two sample types. This indicates that the full connection configuration provides better load transfer and structural integrity, resulting in higher bending moments along the beam's length.
4. **Intermediate Bending Moment in CF Samples:** The CF samples exhibit bending moments that fall between those of the BF and fully connected samples. This suggests that while the CF configuration is more effective than the BF configuration in load transfer, it is not as effective as the fully connected configuration.

In summary, the bending moment distribution along the beam length varies significantly depending on the type of connection and the presence of openings. Fully connected samples show the highest bending moments,

particularly in the middle regions, indicating superior structural performance. BF samples, with the lowest bending moments, highlight the limitations of beam-only connections, while CF samples demonstrate intermediate performance.

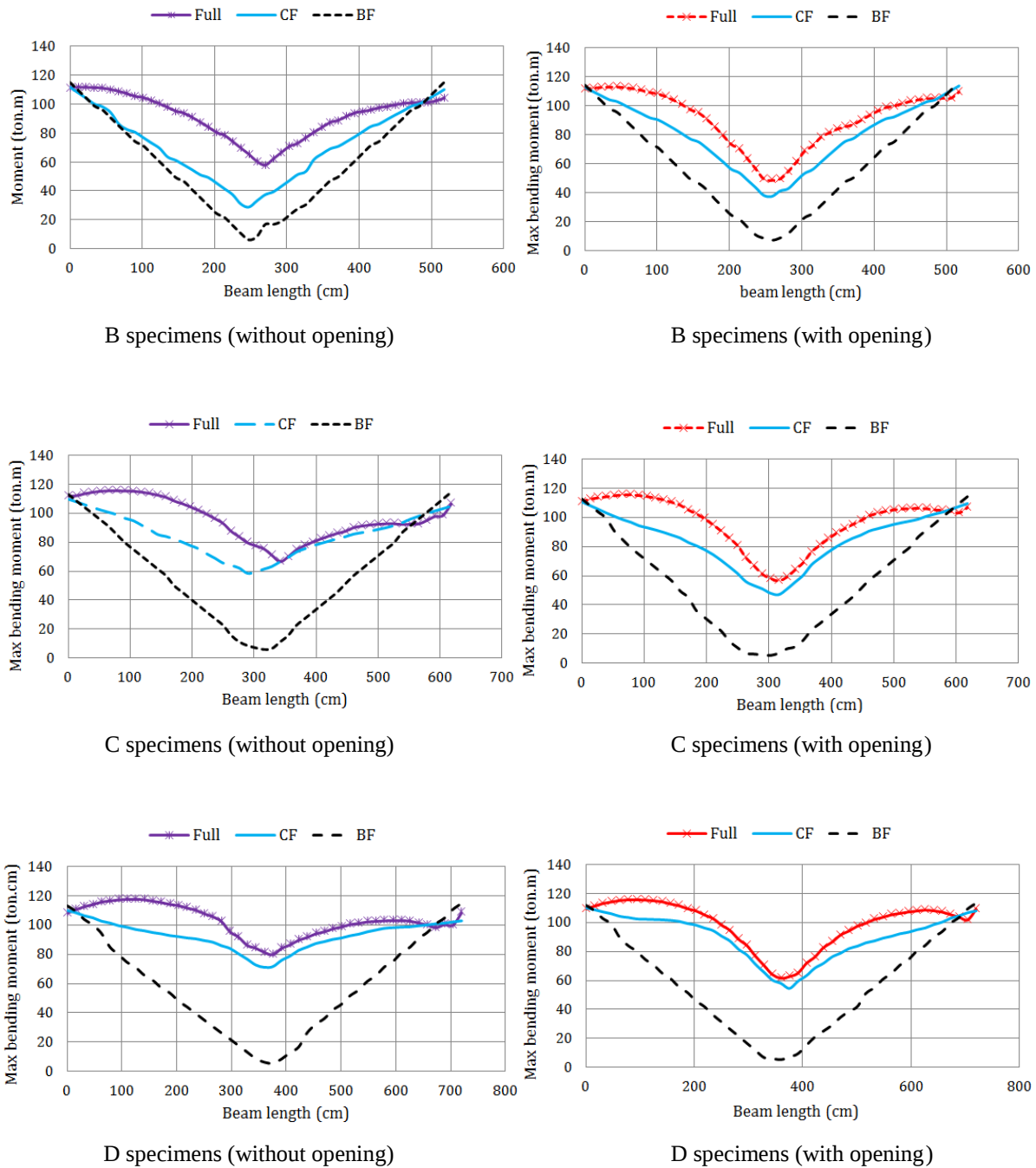


Fig. 6. Bending moment curve for beam

5. Deformations

Fig. 7 illustrates the deformation of the studied models at the end of loading. It is evident that fully bonded plates have larger yield zones compared to CF and BF specimens. This indicates that, according to the stress contour, most parts of the panel have exceeded the yield stress of 100 MPa. In contrast, in the BF specimens, the areas near the beams did not even yield.

The CF samples, however, provide a higher product surface compared to the BF samples, indicating a more efficient use of the plate capacity when the plate is connected only to the columns. The formation of diagonal tensile fields is clearly evident in all studied samples. In fully connected samples, these diagonal tensile fields are formed along the diameter of the samples. In samples containing a central opening, the diagonal tensile fields have formed on the side plates around the opening.

The study of the equivalent plastic strain contour in samples with openings reveals that stress concentration occurs in the corners of the openings. Therefore, it is essential to use boundary members adjacent to the openings to mitigate these stress concentrations and enhance structural performance. This consideration is crucial for ensuring the integrity and durability of the steel shear walls, especially in the presence of openings.

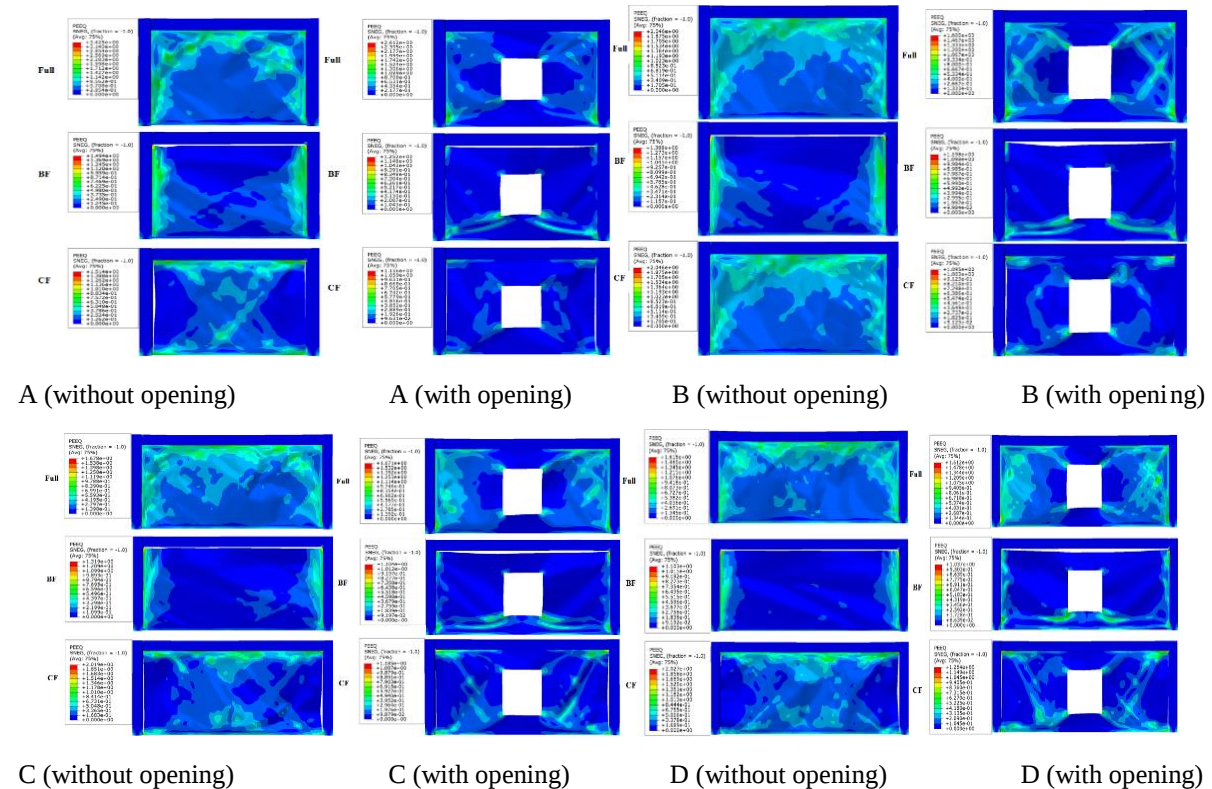


Fig. 7. Failure mechanism of group

5.1. Seismic Coefficients

According to the hysteresis curves from the finite element analyses, seismic coefficients such as the behavior factor, ductility factor, and overstrength factor can be obtained. There are several methods to calculate these seismic coefficients. In this research, the method suggested by Yang et al. is used. The real and bi linearized response curve of Yong is shown in Fig. 7. V_y is the yield load at the effective yield strength point, and Δ_y is the corresponding displacement. V_s corresponds to the base shear that the first element reaches the plastic zone (first significant cracks occur in the structure), and the structure significantly exceeds the elastic zone. This load level is usually used in seismic codes that apply ultimate load methods. Some design codes which use the allowable stress design method, employ V_w load in design. The corresponding displacements for V_s and V_w loads are Δ_s and Δ_w , respectively. Δ_{max} is the maximum (ultimate) displacement of the structure.

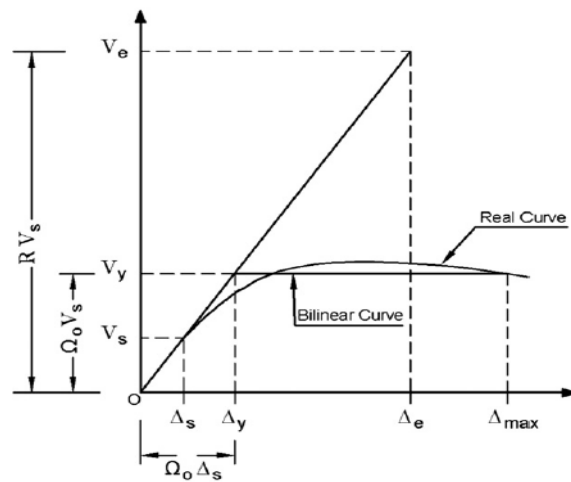


Fig. 8. real curve and idealized bilinear curve

The behavior factor (R) is computed through the next formula:

$$R = \frac{V_e}{V_s} = \frac{V_e}{V_y} \times \frac{V_y}{V_s} = R_\mu \Omega_0 \quad (1)$$

The over strength factor is obtained from the next equation:

$$\Omega = \frac{V_y}{V_s} \quad (2)$$

The overall ductility of the structure is determined as the structure's ultimate displacement separated by the structure's yield displacement.

$$\mu = \frac{\Delta_{max}}{\Delta_y} \quad (3)$$

Table 14 provides the values of seismic coefficients for specimens without openings, calculated with loading up to 2.5% of the frame height. The data indicate that fully connected specimens exhibit higher ductility factors than partially connected specimens, demonstrating superior capacity for plastic deformation and energy dissipation during seismic events. This trend is consistent across all groups. Table 15, which shows the seismic coefficients for specimens with openings, also highlights that fully connected specimens outperform partially connected ones in terms of ductility, behavior, and overstrength factors. Despite the presence of openings reducing overall seismic performance, fully connected configurations maintain better resilience, emphasizing their importance in enhancing the structural integrity and seismic performance of steel shear walls.

Table 14. Seismic coefficients of specimens without opening

Model	Connection condition	Ω	μ	Response factor (r)
A	Full	1.97	9.73	8.47
	BF	2.40	6.65	8.43
	CF	2.43	7.56	9.13
B	Full	2.06	10.7	9.31
	BF	2.52	6.7	8.88
	CF	2.49	8.85	10.19
C	Full	1.9	9.43	8.19
	BF	2.65	6.13	8.88
	CF	2.62	9.42	11.07
D	Full	2.09	12.78	10.38

	BF	2.29	6.33	7.82
	CF	2.20	8.66	8.91

Table 15. Seismic coefficients of specimens with openings

Model	Connection Condition	Ω	μ	Response Factor
A	Full	2.53	8.12	9.88
	BF	2.55	5.65	8.19
	CF	2.42	5.81	7.9
B	Full	2.33	8.61	9.4
	BF	2.43	5.83	7.96
	CF	2.49	5.66	8.01
C	Full	2.60	8.51	10.42
	BF	2.46	5.7	7.93
	CF	2.55	6.3	8.71
D	Full	2.57	9.11	10.67
	BF	2.25	6.22	7.63
	CF	2.74	6.73	9.70

6. Conclusions

In this research, the behavior of steel shear walls with different connection conditions between the plate and boundary elements was analyzed. Four groups of steel shear walls of varying lengths were examined, considering three connection modes: plates connected to both columns and beams, only to the beam, and only to the column. A quasi-static analysis was conducted according to the ATC-24 Standard, and the impact of openings in the panels was also studied. Results indicated that partial length connections reduced the shear capacity and initial stiffness. Specimens with full connections showed higher compressive axial forces in the columns, while the connection method did not significantly affect the maximum bending moment of the beam. However, bending moments in beam-connected specimens were notably lower in the mid-regions. Openings further reduced shear capacity.

This study primarily relied on simulations with specific material properties, which may not fully capture real-world conditions, and was limited to certain configurations, restricting generalizability. Future studies should include experimental validation and explore a broader range of materials and structural arrangements to enhance applicability across various engineering scenarios.

Funding

This research did not receive any specific grant from funding agencies in the public, commercial, or not-for-profit sectors.

Conflict of interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Authors contributions

First name and Last name of First Author: Amir Azarpour.

Conceptualization, Methodology, Formal analysis, Writing - original draft.

First name and Last name of Second Author: Sajjad Javanmard Zargabad

Methodology, Formal analysis, Writing - review & editing

First name and Last name of Third Author: Faraz Najafi

Methodology, Formal analysis, Writing - review & editing.

First name and Last name of Fourth Author: Sayyad Karimi Gabalou

Methodology, Formal analysis, Writing - review & editing

First name and Last name of Fifth Author: Maryam parvaresh

Methodology, Formal analysis, Writing - review & editing.

REFERENCES

1. Basler K. Strength of Plate Girders in Shear. J Struct Div. 1961 Oct;87(7):151–80. doi:10.1061/JSDEAG.0000697
2. Driver RG, Kulak GL, Kennedy DJL, Elwi AE. Seismic behaviour of steel plate shear walls. University of Alberta; 1998.
3. Timler PA, Kulak GL. Experimental study of steel plate shear walls. 1983.
4. Vian D, Bruneau M. TESTING OF SPECIAL LYS STEEL PLATE SHEAR WALLS. In 2004.
5. Choi I, Park H. Cyclic test for framed steel plate walls with various infill plate details. In: Lateral. 2008. p. 3550.
6. Guo L, Rong Q, Ma X, Zhang S. Behavior of steel plate shear wall connected to frame beams only. Int J Steel Struct. 2011 Dec 10;11(4):467–79. doi:10.1007/s13296-011-4006-7
7. Jahanpour A, Jönsson J, Moharrami H. Seismic behavior of semi-supported steel shear walls. J Constr Steel Res. 2012 Jul;74:118–33. doi:10.1016/j.jcsr.2012.02.014
8. Sabouri-Ghomi S, Sajjadi SRA. Experimental and theoretical studies of steel shear walls with and without stiffeners. J Constr Steel Res. 2012 Aug;75:152–9. doi:10.1016/j.jcsr.2012.03.018
9. Wang M, Shi Y, Xu J, Yang W, Li Y. Experimental and numerical study of unstiffened steel plate shear wall structures. J Constr Steel Res. 2015 Sep;112:373–86. doi:10.1016/j.jcsr.2015.05.002
10. Shekastehband B, Azaraxsh A, Showkati H. Experimental and numerical study on seismic behavior of LYS and HYS steel plate shear walls connected to frame beams only. Arch Civ Mech Eng. 2017 Jan;17(1):154–68. doi:10.1016/j.acme.2016.09.006
11. Shekastehband B, Azaraxsh AA, Showkati H, Pavir A. Behavior of semi-supported steel shear walls: Experimental and numerical simulations. Eng Struct. 2017 Mar;135:161–76. doi:10.1016/j.engstruct.2017.01.004
12. AlHamaydeh M, Elayyan L. Impact of diverse seismic hazard estimates on design and performance of Steel Plate Shear Walls buildings in Dubai, UAE. In: 2017 7th International Conference on Modeling, Simulation, and Applied Optimization (ICMSAO). IEEE; 2017. p. 1–4. doi:10.1109/ICMSAO.2017.7934869
13. AlHamaydeh M, Elayyan L, Najib M. Impact of eliminating web plate buckling on the design, cost and seismic performance of steel plate shear walls. In: Proceedings of the 2015 International Conference on Steel and Composite Structures (ICSCS15), Incheon, Republic of Korea. 2015. p. 25–9.
14. Farahbakhshtooli A, Bhowmick AK. Seismic collapse assessment of stiffened steel plate shear walls using FEMA P695 methodology. Eng Struct. 2019 Dec;200:109714. doi:10.1016/j.engstruct.2019.109714
15. Mu Z, Yang Y. Experimental and numerical study on seismic behavior of obliquely stiffened steel plate shear walls with openings. Thin-Walled Struct. 2020 Jan;146:106457. doi:10.1016/j.tws.2019.106457
16. Farahbakhshtooli A, Bhowmick AK. Nonlinear seismic analysis of perforated steel plate shear walls using a macro-model. Thin-Walled Struct. 2021 Sep;166:108022. doi:10.1016/j.tws.2021.108022

17. Wang C, Xu L yan, Song L han, Fan J sheng. Numerical study of steel plate shear walls with diverse construction configurations. *Eng Struct.* 2023 Jan;274:115141. doi:10.1016/j.engstruct.2022.115141
18. Sun HJ, Guo YL, Wen CB, Zuo JQ. Local and global buckling prevention design of corrugated steel plate shear walls. *J Build Eng.* 2023 Jun;68:106055. doi:10.1016/j.job.2023.106055
19. Alashkar A, Elkafrawy M, Hawileh R, AlHamaydeh M. Buckling Analysis of Functionally Graded Materials (FGM) Thin Plates with Various Circular Cutout Arrangements. *J Compos Sci.* 2022 Sep 18;6(9):277. doi:10.3390/jcs6090277
20. Elkafrawy M, Alashkar A, Hawileh R, AlHamaydeh M. FEA Investigation of Elastic Buckling for Functionally Graded Material (FGM) Thin Plates with Different Hole Shapes under Uniaxial Loading. *Buildings.* 2022 Jun 10;12(6):802. doi:10.3390/buildings12060802
21. Zhao Q, Qiu J, Li Y, Yu C. Lateral behavior and PFI model of sinusoidal corrugated steel plate shear walls. *J Constr Steel Res.* 2023 Apr;203:107812. doi:10.1016/j.jcsr.2023.107812