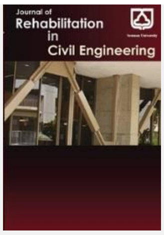

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Investigation of Double-Layer Corrugated Steel Plate Shear Walls in Multi-Story Frames

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ABSTRACT

Flat steel plate shear walls (FPSWs) are widely used in steel structures, but they are prone to buckling under lateral loads. As an alternative, corrugated steel plate shear walls (CPSWs) have been introduced, in which the infill flat steel plate is replaced with a corrugated steel plate. CPSWs have some advantages over FPSWs, such as improved in-plane and out-of-plane stiffness, buckling strength, and ductility. Nevertheless, their applicable thickness is limited due to manufacturing capabilities. In the last few years, to overcome this drawback, double-layer corrugated steel plate shear walls (DCPSWs) composed of two similar corrugated steel plates have been developed. As there are few studies on DCPSWs, mainly focused on single-story frames, this paper considers 48 two-dimensional multi-story frames with DCPSWs using the pushover method. According to the results, corrugated shear walls have less lateral strength compared to FPSWs. However, the difference decreases with increasing the number of stories. The corrugated shear walls have higher ductility, especially for low-rise frames. Furthermore, the frames with DCPSWs experienced the highest inelastic deformations, which can be considered an advantage over CPSWs.

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1. Introduction

Over the last few decades, flat steel plate shear walls (FPSW) have efficiently and widely been used as a part of lateral force-resisting systems, especially in regions with high seismicity. An FPSW system is constituted of a moment-resisting frame (MRF) and an infill plate continuously connected to the vertical and horizontal elements of the frame. These lateral load-resisting systems possess higher strength and initial stiffness compared to conventional MRFs. The coupled infill plate also significantly increases redundancy and ductility [1].

On the other hand, FPSWs have several advantages over conventional concrete shear walls. In these systems, the total structural weight is significantly less, which will reduce seismic forces and the reinforced concrete foundation costs. Additionally, the construction time using FPSWs is shorter [2–4].

In previous studies, considerable efforts have been put into investigating characteristics of FPSWs such as lateral strength, stiffness, and lateral stability [5–7]. The findings show that FPSWs are prone to buckling under lateral loads, especially in cases where the steel plate is small in thickness. The steel plate is vulnerable to buckling because of its low flexural rigidity. In recent studies, the method of equipping the steel plate with stiffeners has been proposed to boost its buckling strength. However, using stiffeners increases construction costs and requires inspection requirements [8]. An alternative to stiffeners is corrugated steel plates. Easley and McFarland [9] first studied the elastic buckling behavior of these plates under in-plane shear and subsequent studies extensively examined their post-buckling behavior [10,11]. Results from these studies indicated a preferable buckling strength of corrugated plates therefore, in an innovative corrugated steel plate shear wall system (CPSW), the infill flat steel plate has been replaced with a trapezoidal or sinusoidal corrugated steel plate. CPSWs have tangible advantages due to their features such as in-plane and out-of-plane stiffness, strength, ductility, and stable hysteretic features that have been scrutinized in many experimental and numerical studies. Clayton et al. [12] in an experimental study on the performance of CPSWs found that demands on peripheral frame members were reduced. In their experimental study, Emami et al. [8] concluded that CPSWs have higher stiffness, strength, ductility, and energy dissipation capacity than FPSWs. To investigate the effects of the plate corrugation shape, Edalati et al.'s study [13] on the nonlinear behavior of trapezoidal and sinusoidal CPSWs under monotonic loading demonstrated that CPSWs with trapezoidal corrugation have better performance in terms of energy dissipation, ductility, and ultimate capacity while requiring less material for construction. Effects of other traits of CPSWs such as the corrugation angle, the plate thickness, and corrugation number were also investigated in Klali’s parametric study [14]. Farzampour et al. [15] in their numerical study using linear regression analysis proposed relationships to predict the ultimate strength of CPSWs with opening.

Despite the numerous advantages of CPSWs over FPSWs, there is a major drawback with the applicable range of the corrugated plate thickness due to the instrumental capability of cold rolling machines. In recent years, with the growth of high-rise construction, the necessity of thicker steel plates to use in shear wall systems has exceeded the capabilities of common cold rolling machine technology, which can produce corrugated plates with a maximum thickness of up to 8 mm [16]. Since the drawback can hinder the vast usage of CPSWs, especially as the lateral load-resisting system of high-rise buildings, a double-layer corrugated plates shear wall (DCPSW) system was introduced by Tang et al. [16]. A DCPSW is composed of two similar corrugated steel plates connected at contact points with equally spaced bolts. In another parametric study [17], the effects of the aspect ratio, the bolt columns number, the corrugated plate thickness, and yield stress on the shear strength of DCPSWs are examined. The study introduced a normalized slenderness ratio for DCPSWs, adversely affecting the shear strength. Ghodratian and Maleki [18] investigated the performance of DCPSWs in one-story single-bay models experimentally and numerically under cyclic loads and approved the eligible hysteretic behavior of DCPSWs. They also focused on various situations of connections between infill plates and frame elements and specified that the column axial force is lower in the beam-only-connected DCPSWs.

The literature on CPSW and DCPSW systems reviewed so far has mainly examined numerically or experimentally the behavior of single-story shear wall systems. Undoubtedly, in multi-story buildings due to the contribution of higher vibration modes and more gravity loads, some structural responses are different from those of previous studies. On the other hand, as DCPSWs were introduced in the last few years, there are not a sufficient number of studies to conclude their various functional aspects. Therefore, in this paper, multi-story single-bay frames infilled with CPSWs and DCPSWs will be investigated through the finite element (FE) method. As in the shear wall systems, yielding and buckling of infill plates are material and geometric nonlinearity cases, therefore, the analyses should be carried out using nonlinear approaches. For this purpose, the nonlinear time history analysis (NLTHA) is the most accurate method, but at the same time, complicated and time-consuming. The nonlinear static procedure known as pushover analysis accounting for both geometric and material nonlinearities is an interesting alternative. This method can trace the yielding and failure sequence of structural elements and according to the specifications of AISC-341 [19] can be used in the design process and assessment of the shear wall systems. Moreover, in past studies, it has been utilized to identify behavioral aspects of the shear wall systems [20]. Hence, in this study pushover analysis will be used to investigate multi-story frames with various shear walls.

2. The study outline

Numerical analyses were conducted to assess 48 structural models, including 2D multi-story frames. The analyses of 3, 5, and 8-story frames were carried out without shear walls, with FPSWs, and with corrugated shear walls involving CPSWs and DCPSWs. Corrugated shear walls were modeled for 3 different corrugation angles of 30, 45, and 60 degrees. All structural models were examined using pushover analyses in which the lateral load height-wise distribution effects are adequately understood. Here, the uniform lateral load pattern and a load pattern corresponding to the first elastic natural mode shape of vibration were used in the FE analyses [21–25]. Table 1 gives a summary of the structural models. By inspecting the results obtained from the analyses, it is possible to compare the performance of the multi-story frames infilled with CPSWs and DCPSWs. This is important because there are few researches on DCPSWs and previous studies on corrugated shear walls have mainly focused on single-story frames.

Table 1. The structural models.

Number of stories	Lateral load distribution	Type	Corrugation Angle
3, 5, 8	**First mode**	MRF	-
		SSPSW	-
		CSPSW	**30, 45, 60**
		DCSPSW	**30, 45, 60**
	Uniform	MRF	-
		SSPSW	-
		CSPSW	**30, 45, 60**
		DCSPSW	**30, 45, 60**

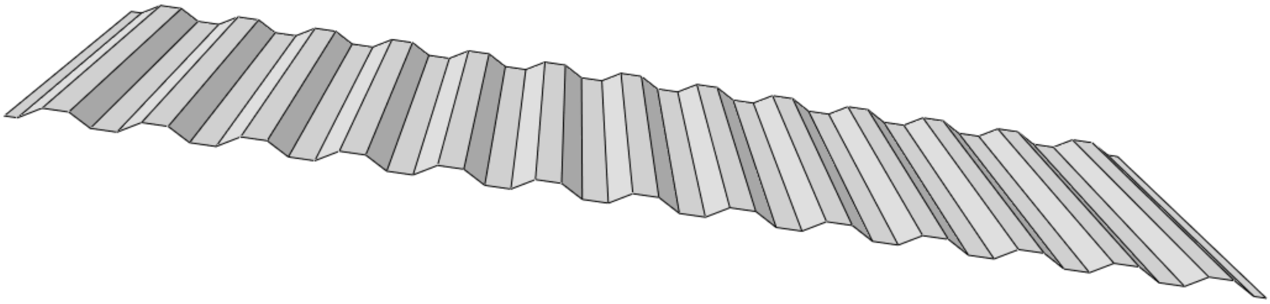
3. Modeling details and underlying assumptions

The 3, 5, and 8-story frames are 3200 mm and 4000 mm in story height and bay width, respectively. The yield stress, ultimate stress, and elastic modulus of the steel material used in the frame elements and infill plates are 240, 360, and 206000 MPa, respectively. Table 2 summarizes the steel plate thickness and the frame elements' geometric specifications as height (d), flange width (b_f), flange thickness (t_f), and web thickness (t_w). The seismic design forces of the moment-resisting frame elements must be specified so that the infill plates fully yield in tension which is provided by the capacity design procedure as described in AISC Design Guide 20 [26].

To facilitate comparison, the thicknesses of FPSWs and CPSWs are equal and two times each plate thickness in DCPSWs. Three corrugation angles of 30, 45, and 60 degrees have been considered in the corrugated steel walls. Fig. 1 depicts the trapezoidal cross-section of the corrugations in CPSWs and DCPSWs. The ABAQUS FE package features [27] were employed to perform the analyses in which material and geometric nonlinearities were taken into account for structural models (see Fig. 2). A widely purposed, functional four-node element known as S4R from the software built-in library was selected to construct the structural components. This element is a fully integrated finite-membrane-strain element.

On the other hand, the S4R element is a reduced integrated quadrilateral finite-membrane-strain element that incorporates the reduced integration theory. Figure 2 displays the DCPSW and the modeling mesh density.

In the surrounding frames, the connection of the beams to the columns is moment-resisting and the steel walls are continuously connected to the frame elements.



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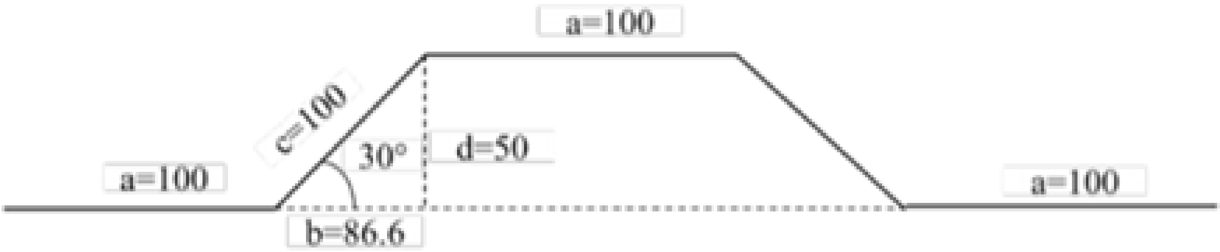


(b)

Fig. 1. (a) the CPSW (b) the dimensions of a trapezoidal corrugated shape.

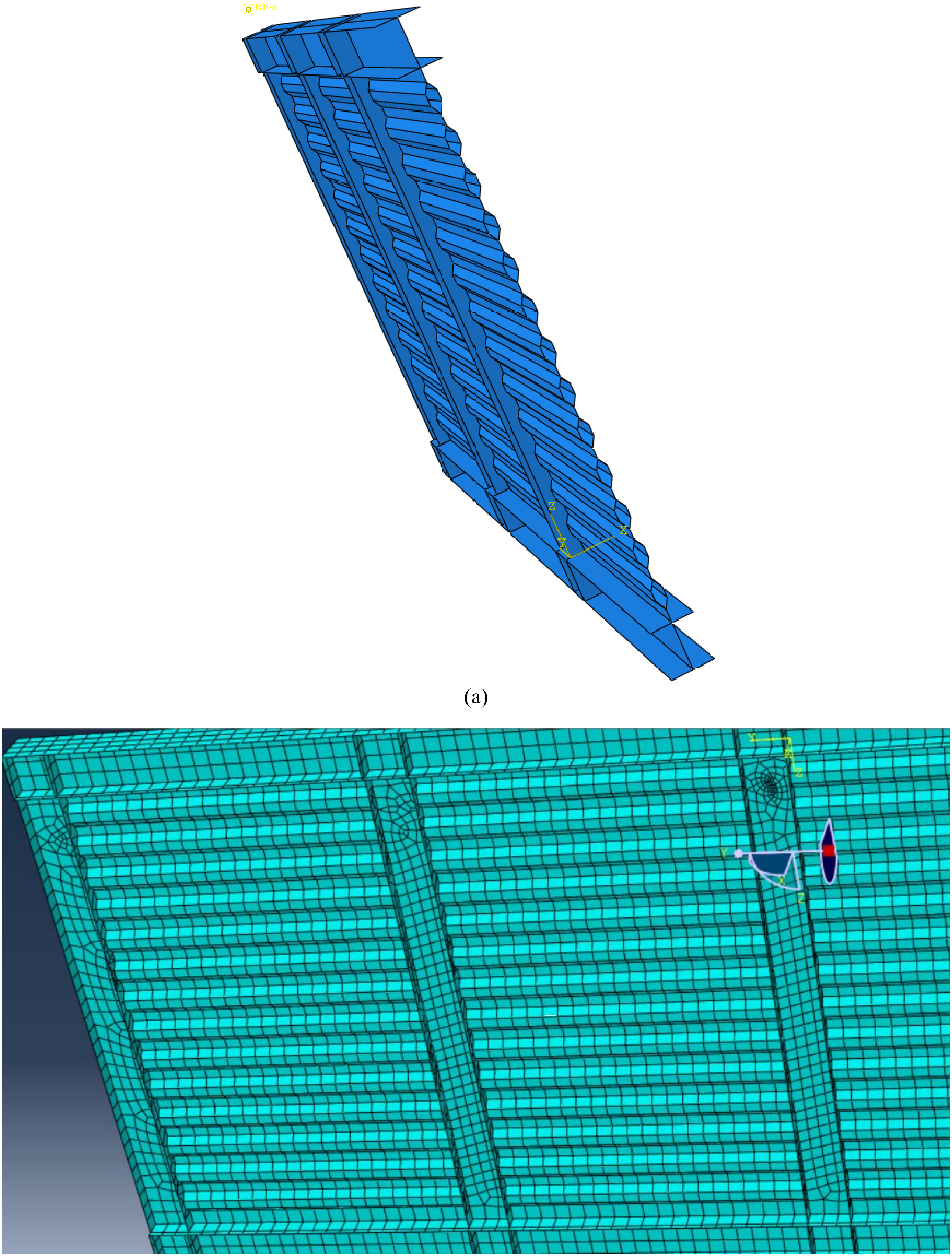
At the ground level, the columns are clamped end. To accurately represent boundary conditions, the beams at different stories are laterally supported, which restricts deformations to the longitudinal axis of the frames.

To simulate manufacturing errors, initial imperfections were introduced in the finite element (FE) models, based on the first buckling mode shapes of the shear walls. Out-of-plane imperfections in the corrugated plates should be considered as H/200, where H represents the height of the corrugated plate [28].

As shown in Fig. 3, to assess for steel strain hardening during lateral pushover loadings, the behavior of the steel material was assumed as a bilinear curve with a post-yield inclination of 0.5% of the steel elastic modulus. The roof displacement in the monotonic pushover analyses was adjusted so that the story drift does not violate the seismic codes' recommended value of 2.5 percent [29].

To ensure the capability of creating accurate FE models, Fig. 4 compares force deformation curves of a single-story frame infilled with a trapezoidal CPSW studied experimentally by Emami et al. [8] and analyzed numerically here under cyclic loading (Fig. 5). It can be observed that there is a good agreement between experimental and numerical curves.

Moreover, the maximum strength in the experimental and numerical studies were 490 and 508 kN, respectively, with a negligible difference of 3.54 percent. Regarding these observations, it is understood that the modeling and analysis processes had been successfully carried out and the same procedure can be implemented to develop the study models.



(a)

(b)

Fig. 2. (a) DCPSW in a 3-story frame (b) mesh density in the FE modelings.

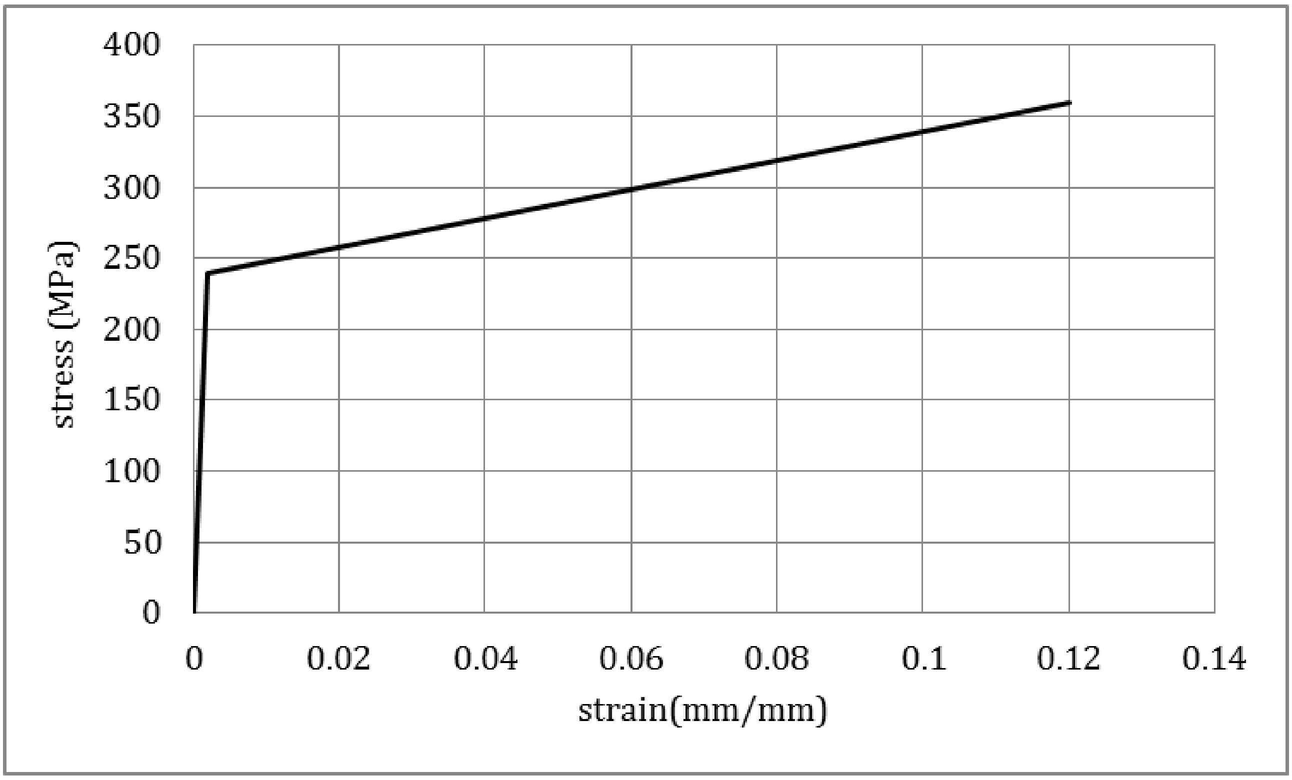


Fig. 3. The behavior curve of the steel material in Abaqus.

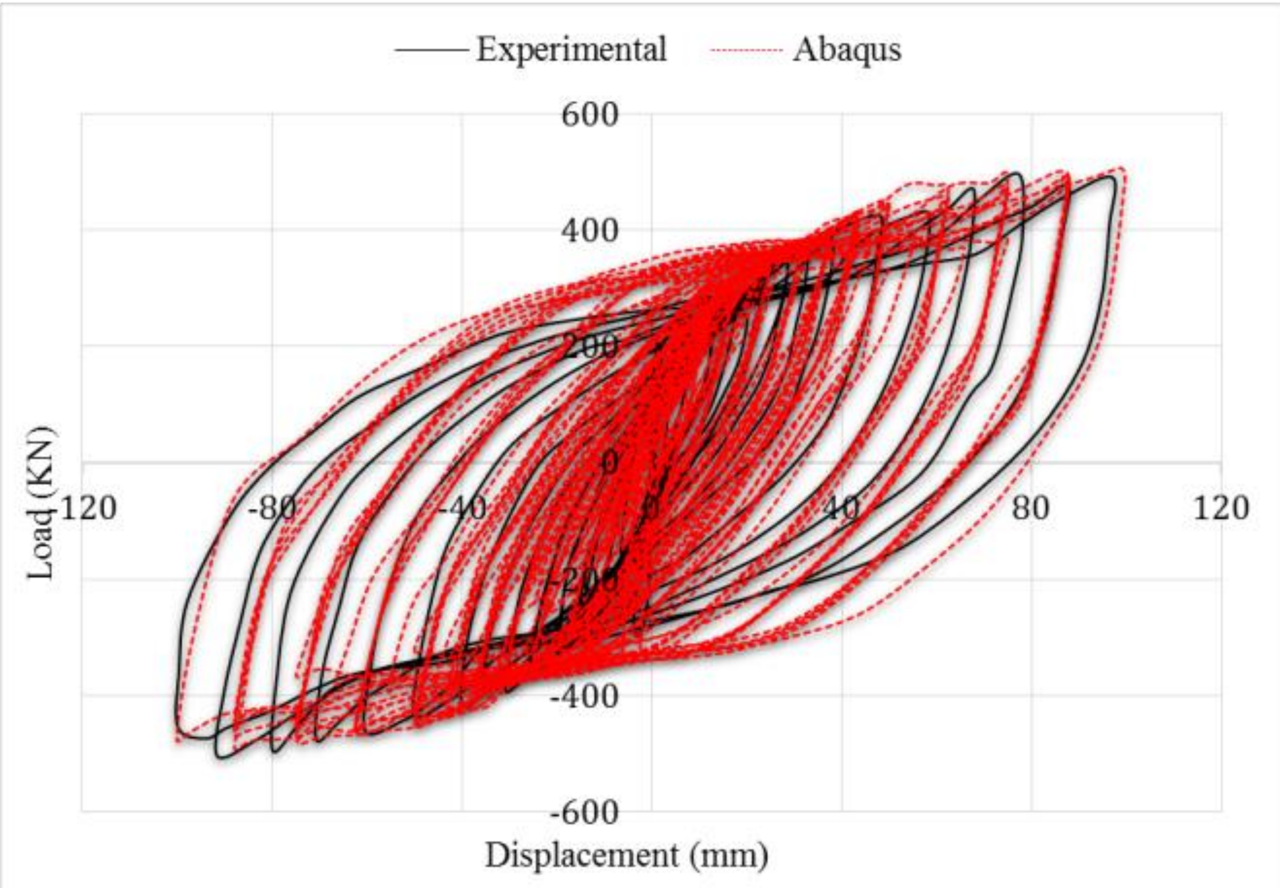


Fig. 4. Force-deformation curve of a single-story frame with CPSW under cyclic loading.

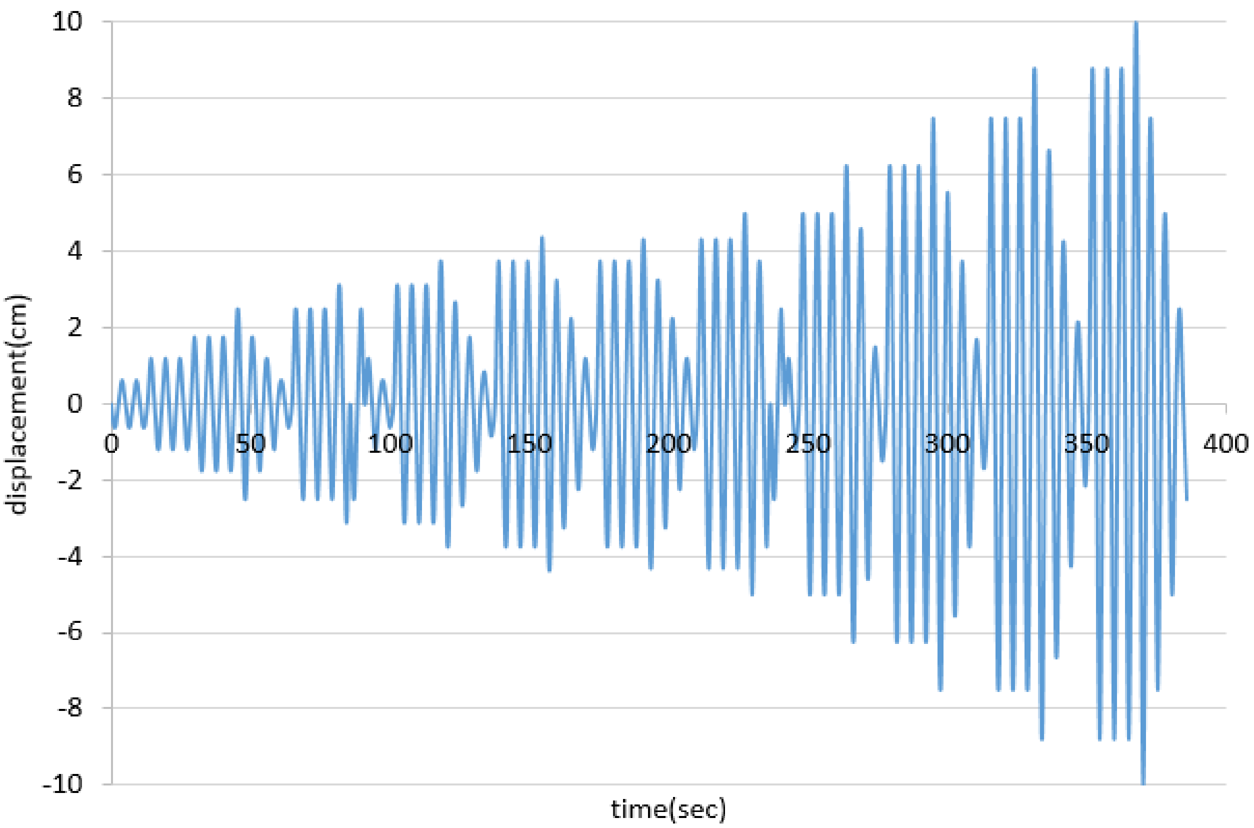
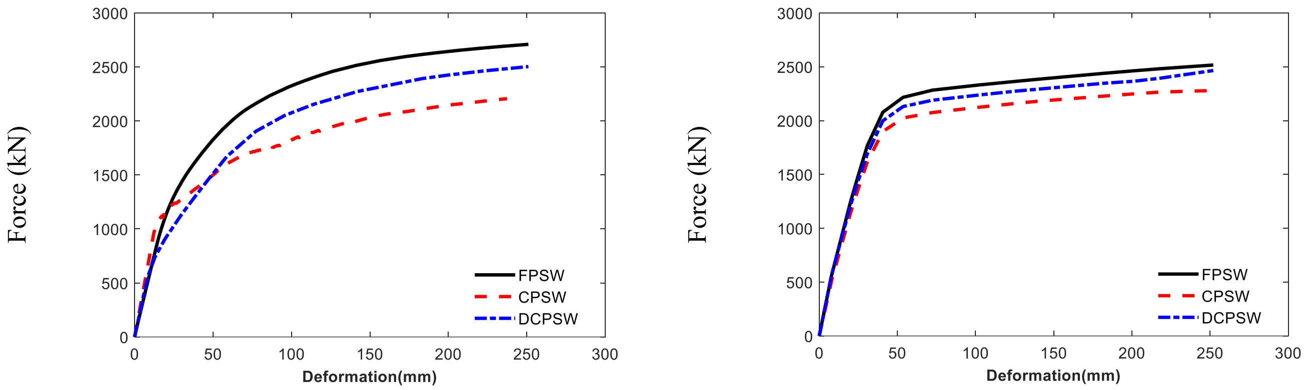


Fig. 5. The lateral cyclic loading.

4. Results

The results of the pushover analyses of 3 to 8-story frames with and without steel plate shear walls are discussed in this section. Figs. 6 to 8 show force-deformation curves of the frames with shear walls under the uniform and the first mode lateral loads and as expected, these frames have greater lateral strengths compared to frames without shear walls. In addition, according to the figures, CPSWs and DCPSWs have lower strength than FPSWs and with increasing story numbers from 3 to 8, the base shear curves for all the walls are closer. This means as the number of stories increases, differences in the lateral strength between frames with the same number of stories but different steel walls, reduce. It should be noted that this is an approximate trend along with some negligible changes implying that the performance of corrugated walls will improve with increasing the number of stories.

The figures also demonstrate that after extended excursions to the inelastic range, DCPSWs possess greater lateral strengths than CPSWs which holds for all corrugation angles. It should be noted that because of the improved wall buckling resistance at the corrugation angle of 60, the lateral strength of corrugated walls is highest at this angle, especially for 5 and 8-story frames with DCPSWs. Conceiving other general trends with the plate corrugation angle is difficult due to the impacts of the lateral load distribution on the frame's inelastic responses. Instead, reviewing the results makes clear that the uniform lateral loading in comparison to the first mode pattern leads to higher lateral strength. This demonstrates how the lateral load height-wise distribution affects structural responses in pushover analysis.



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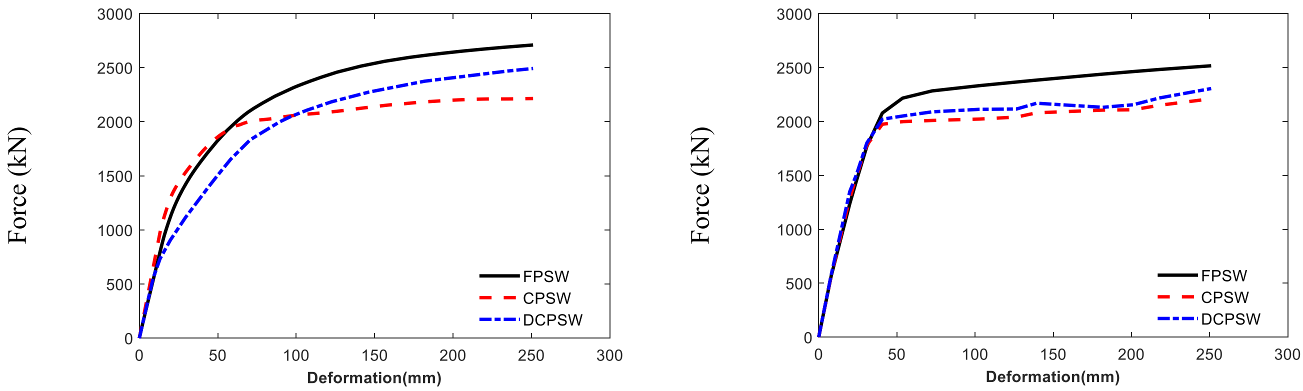

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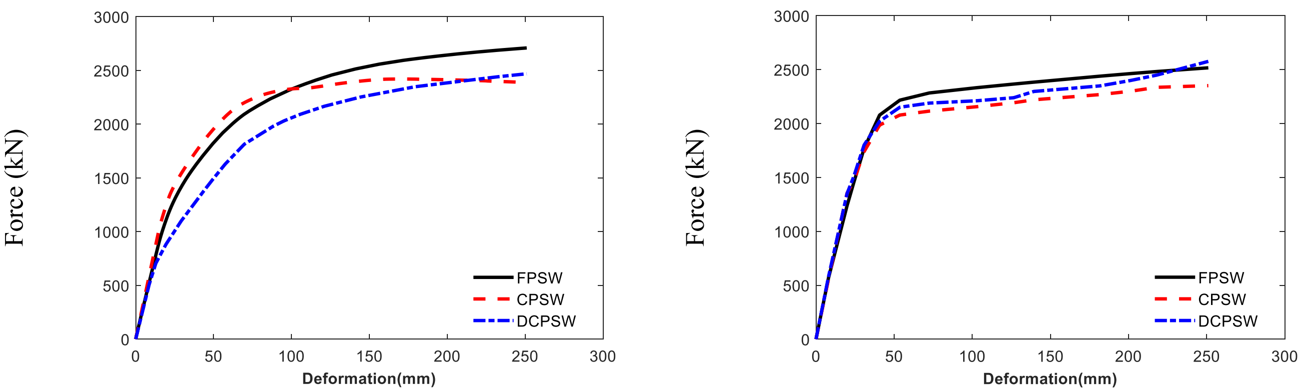
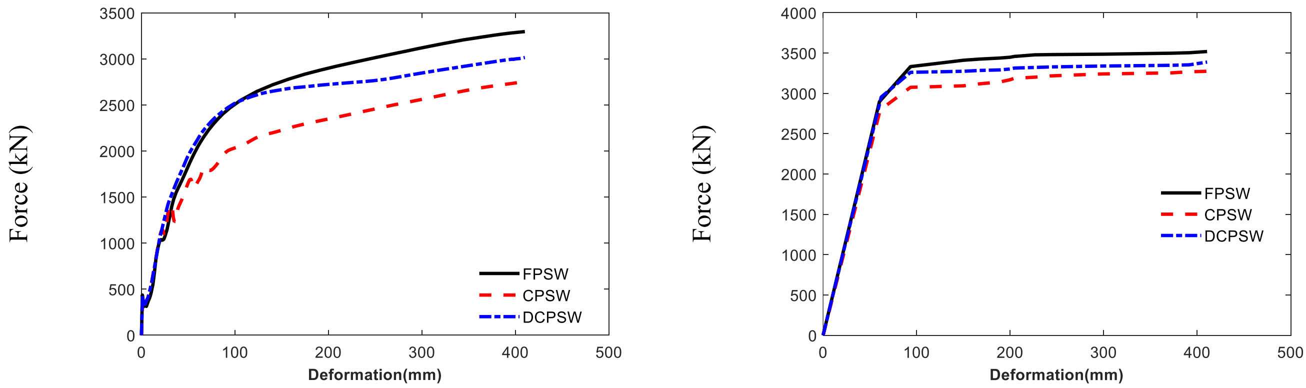


(c)

First mode Uniform

Fig. 6. Base shear versus roof displacement for the 3-story frames with the corrugation angles of a) 30, b) 45, and c) 60 degrees.



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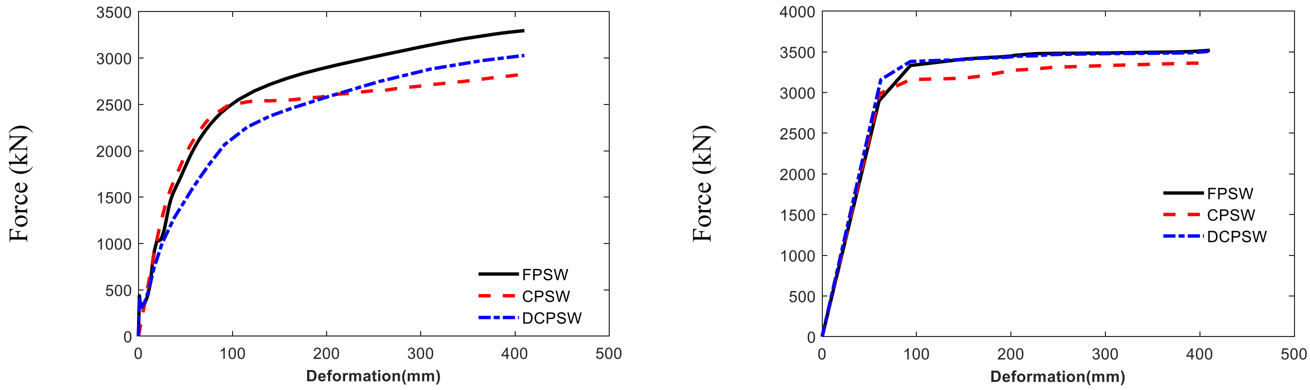

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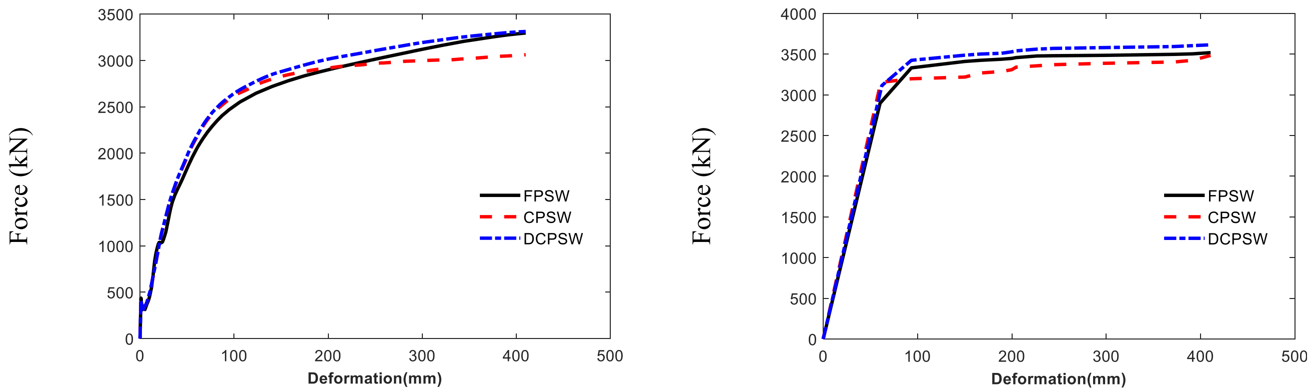


(c)

First mode Uniform

Fig. 7. Base shear versus roof displacement for the 5-story frames with the corrugation angles of a) 30, b) 45, and c) 60 degrees.

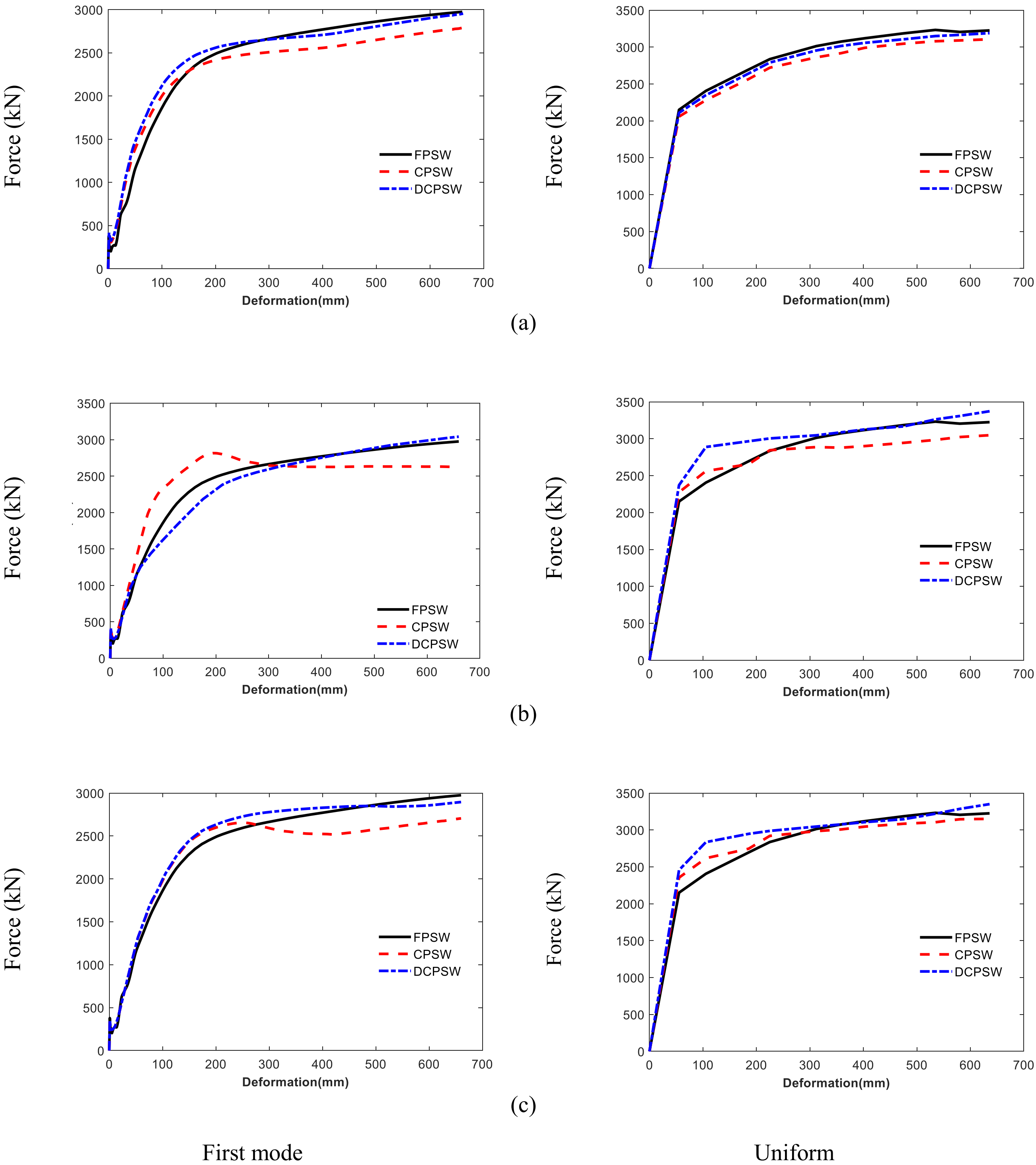


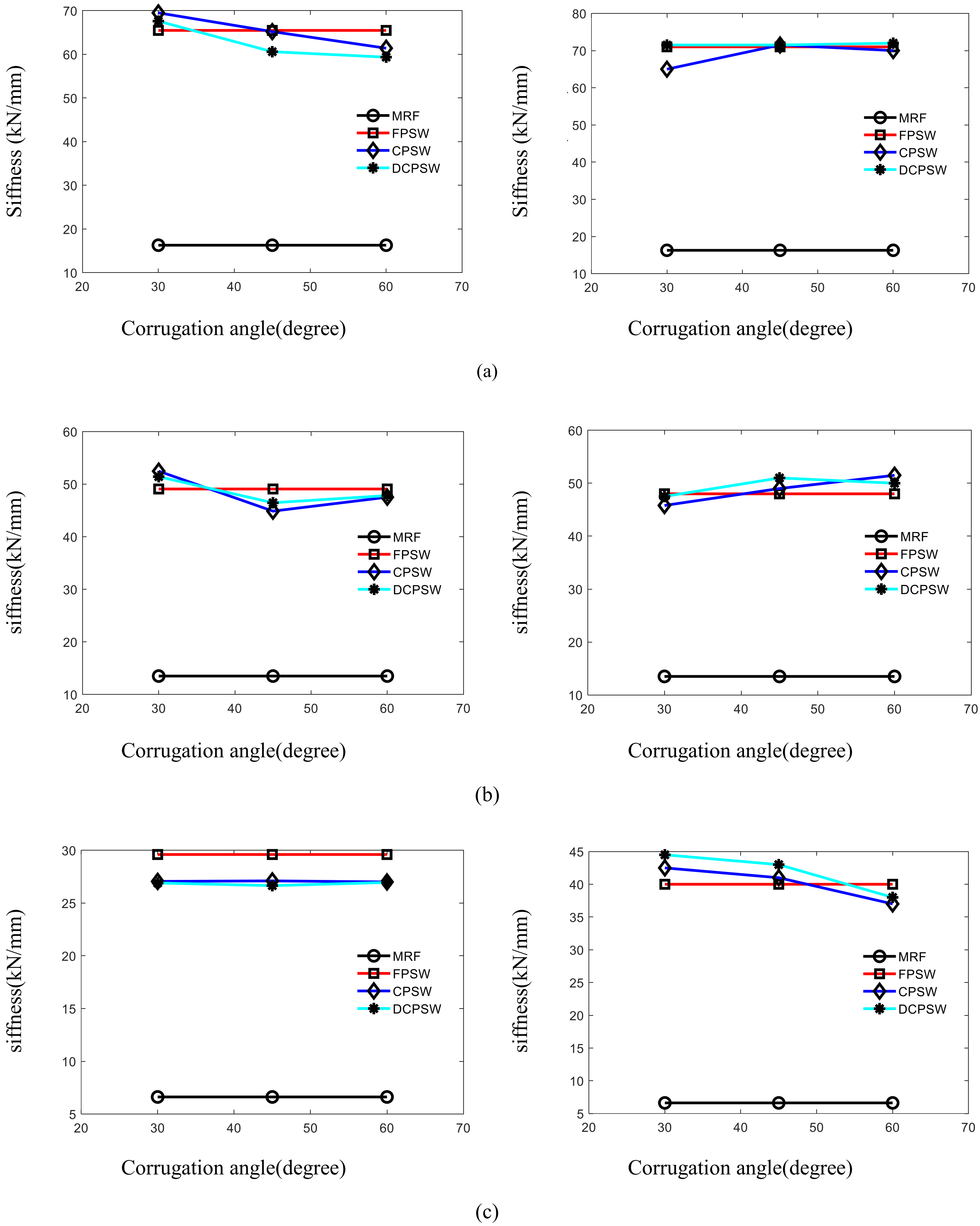
Fig. 8. Base shear versus roof displacement for the 8-story frames with the corrugation angles of a) 30, b) 45, and c) 60 degrees

Fig. 9 depicts several variational trends for the initial stiffness changes in the analyzed frames. For the corrugation angle of 30, the 3 and 5-story frames with the corrugated shear walls and FPSWs under the first mode lateral load pattern have close initial stiffness. For the corrugation angle of 45, the initial stiffness of 3 and 5-story frames with corrugated shear walls under the first mode lateral load pattern is lower. In the 3-story frames, variations of the corrugation angle from 45 to 60 degrees reduces the stiffness, while in the 5-story frames, it is vice versa. For the 8-story frames with the corrugated shear walls and FPSWs under the first mode lateral pattern, the initial stiffness values per all corrugation angles are almost equal.

For the uniform lateral load pattern, increasing the corrugation angle value raises the initial stiffness for the 3 and 5-story frames and reduces it for the 8-story frames. In addition, the initial stiffness of the frames with CPSWs is close to that of frames with DCPSWs. As the initial stiffness values concern the elastic behavior of the models, the different trends of variations between the studied models can be attributed to the higher vibration modes. Indeed, the higher modes are more likely to contribute to the responses of the taller frames. On the other hand, the effects of the lateral load patterns on the higher modes' contribution should not be neglected. In summary, concerning the lateral initial stiffness, the most efficient corrugation angle is 30 degrees. Furthermore, it can be realized that the initial stiffness of the frames under the uniform lateral load pattern is higher than the corresponding values under the first mode lateral load pattern.

Steel plate shear walls can significantly improve the ductility of flexural frames. Fig. 10 demonstrates different trends for the ductility of the frames with CPSWs and DCPSWs. The ductility of the structural models is estimated as the ratio of the ultimate roof displacement to its yield displacement obtained through the bilinear idealization of pushover curves [27]. A decreasing trend can be observed for the ductility of the 3- and 5-story frames infilled with corrugated shear walls when the corrugation angle increases from 30 to 45 degrees. With a further increase of the corrugation angle from 45 to 60 degrees, the ductility increases under the uniform lateral loading and decreases under the first mode lateral loading. The ductility of corrugated steel walls is higher than FPSWs and the peak difference occurs at the corrugation angle value of 30. Hence, it can be concluded that this corrugation value is the most effective corrugation angle for the 3- and 5-story frames under both lateral loading patterns. For the 8-story frame under both of the lateral load patterns, the ductility with the corrugated walls per all corrugation angles is close to that of FPSWs and it is not possible to identify a specific corrugation angle corresponding to the largest ductility for the 8-story frame. Regarding the diagrams of the first mode lateral loading depicted in Fig. 10, adding steel shear walls to the frames increases the ductility of the 3- and 5-story frames more than the 8-story frame. Furthermore, for all frames, the ductility under the first mode lateral loading is substantially larger than the values associated with the uniform lateral loading. Therefore, although corrugated steel plate shear walls enhance lateral stiffness and lateral strength, about ductility, particularly for high-rise buildings, more analyses are required.

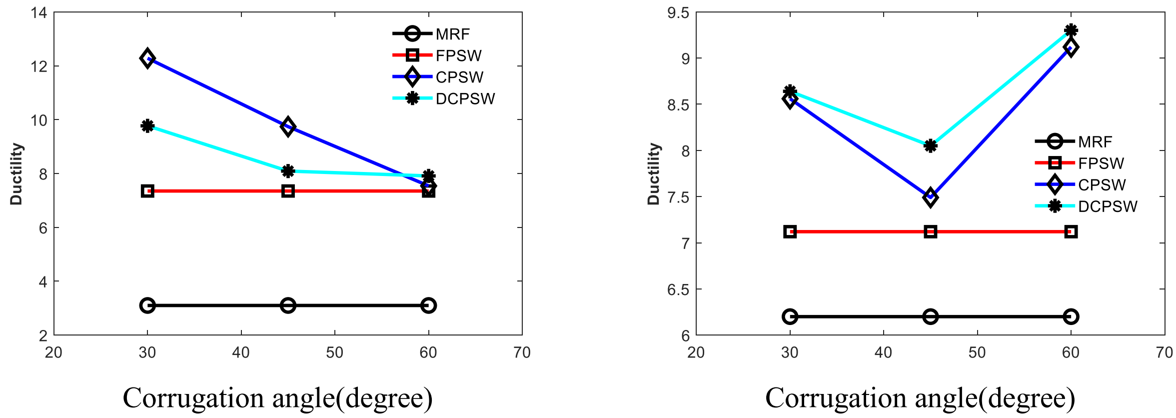
The observations made so far about lateral strength and ductility demands of corrugated and flat plate shear walls are based on the yielding, global buckling, and local buckling phenomena in steel walls and their surrounding frame members. Meanwhile, as their resistance to elastic global buckling is enhanced by the corrugation in the steel plate, local buckling (particularly in adjacent columns) and yielding will govern the behavior of corrugated steel plate shear walls. This can be further explored by considering the plastic strains in the models.



First mode Uniform

Fig. 9. Initial stiffness values versus the corrugation angles in, a) 3-story, b) 5- story, and c) 8- story frames.

Comparison between the maximum plastic strains of some of the frames in Fig. 11 demonstrates that the frames with DCPSWs have experienced profoundly higher inelastic deformations than other systems, indicating that they can perform better than CPSWs. Moreover, the figure shows high values of plastic strains at both ends of the beams. It should be emphasized that such deformations at both ends of the beams were detected in all of the frames; however, only some of them are illustrated here for brevity. As shown in Fig. 12, the rotation values at the location of plastic strains at both ends of the beams were derived from the analysis results to further investigate the issue.



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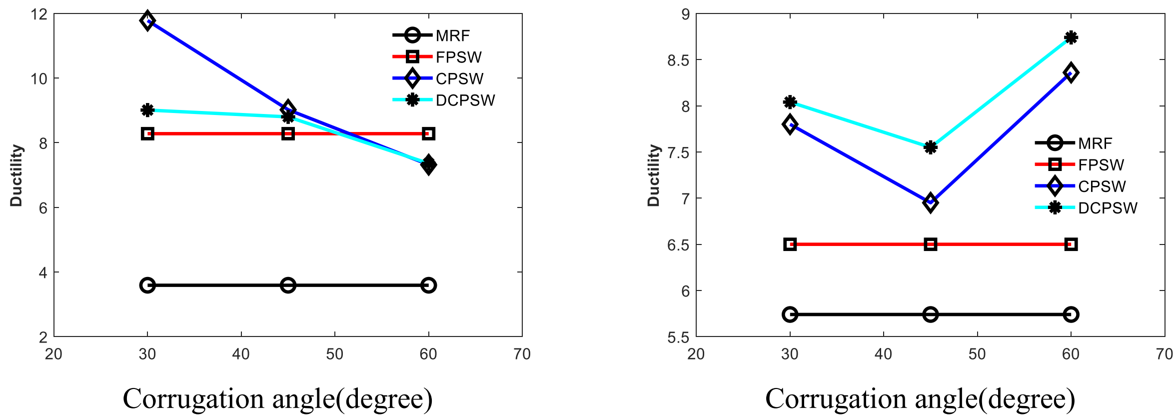

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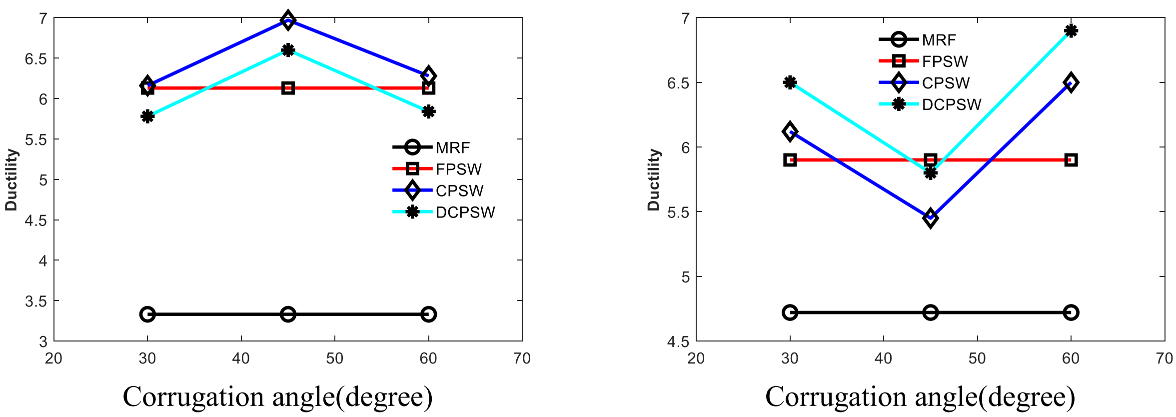
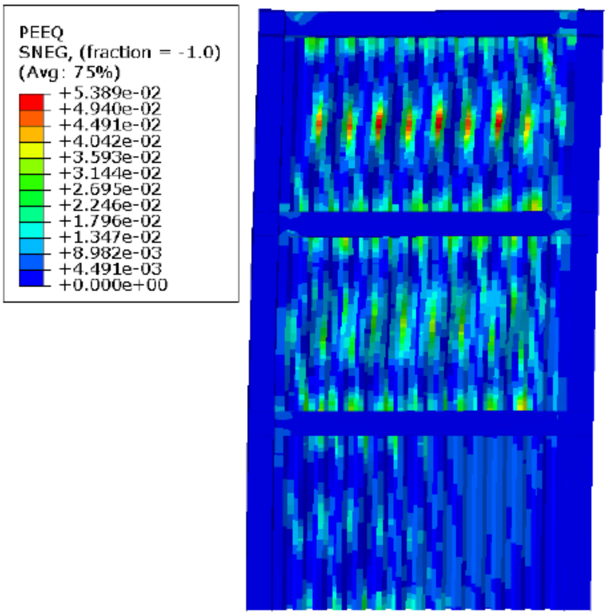


(c)

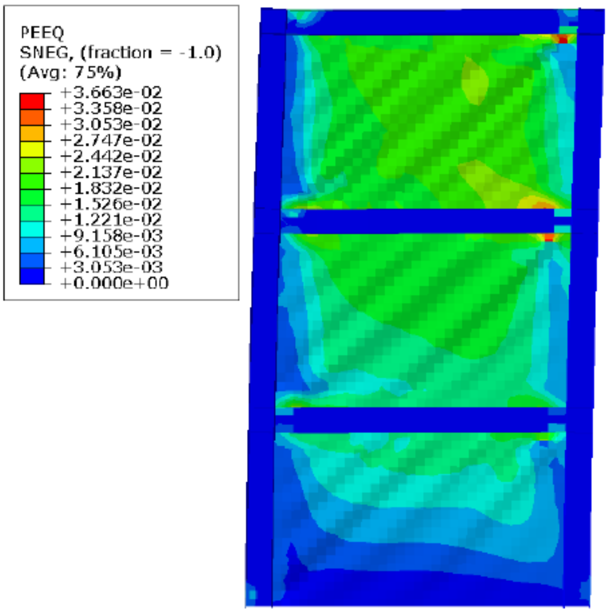
Triangular Uniform

Fig. 10. Ductility versus the corrugation angles in, a) 3-story, b) 5- story, and c) 8- story frames.

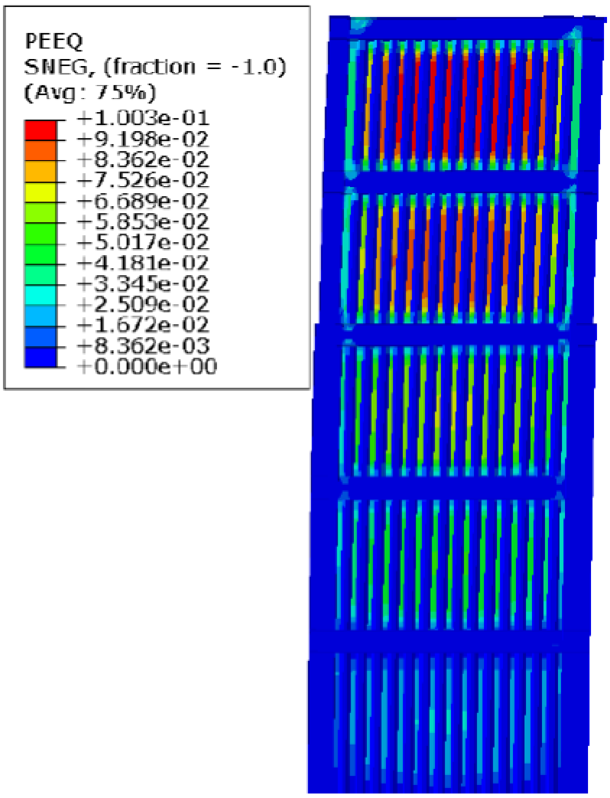
Starting with the first story as number 1 and continuing to the last story of each frame, the two ends of all beams are labeled. For instance, the beam ends of 1, 2, 3, 4, 5, and 6 on the 3-story frame diagram correspond to the beams of the first to third stories, respectively. Similarly, the same procedure is followed to label both ends of beams for the other frames. According to Fig. 13, the rotation values at both ends of the beams can be regarded as the plastic rotation which does not occur at the midspan of the beam. However, due to the rigid beam-column joints, this concentration of inelastic deformations is not surprising, as indicated by the greater end bending moments compared to those of the mid-span. The difference between the plastic rotation values at the story levels of the 3-story frame is substantially lower than that of the 5-story and 8-story frames. The rotation values in the 3-story frames are between 0.015 and 0.03, while the values in the 5 and 8-story frames are between 0.01 and 0.05. The plastic rotation values substantially increase in 5 and 8-story frames, after the third story, such that, according to the figures, they surpass the performance level of life safety and, in certain cases, collapse prevention. However, the plastic rotation at both ends of the beams might be associated with the design approach of the frame members. Since such members must sustain the stress due to yielding in the steel plate shear wall, plastic joints are expected to develop at both ends of the horizontal members and no plastic joint is expected to occur in the midspan. The shear wall yielding occurs in tensile diagonal stripes on the wall plate when a lateral force is applied. Fig. 12 shows that the plastic rotation values are higher at the upper story levels. The reason might be related to the natural vibration modes of these frames and how they are laterally excited. However, the general trend deduced from the diagrams in Fig. 12 is that for all of the 3-story frames, the plastic rotation values at the end of the beams are very close to each other. In the 5 and 8-story frames, the plastic rotation values in the upper-level beams connected to the corrugated shear walls are significant and more than the values associated with FPSWs. As these plastic rotations at the ends of the beams are due to the shear wall yielding, this observation is expected. In addition, with more elastic global buckling strength of the walls, the possibility of steel plate yielding increases. On the other hand, the amount of plastic rotations in the lower stories for the beams connected to the corrugated shear walls is less than those of FPSWs. Because of the increased lateral force at these story levels, elastic global buckling occurs sooner, especially in FPSWs, and spreading the diagonal tension field in the walls raises the plastic rotation values at the ends of the beams. Furthermore, elastic global buckling of corrugated steel walls increases axial forces in the lower levels, leading to inelastic global buckling of the walls or local buckling in the adjacent columns in the lower levels. According to Fig. 12, it appears that in this scenario, local buckling would restrict yielding in the walls and consequently the plastic rotations at the ends of the beams. Local buckling also would cause the lateral strength of corrugated steel wall frames to be lower than FPSWs.



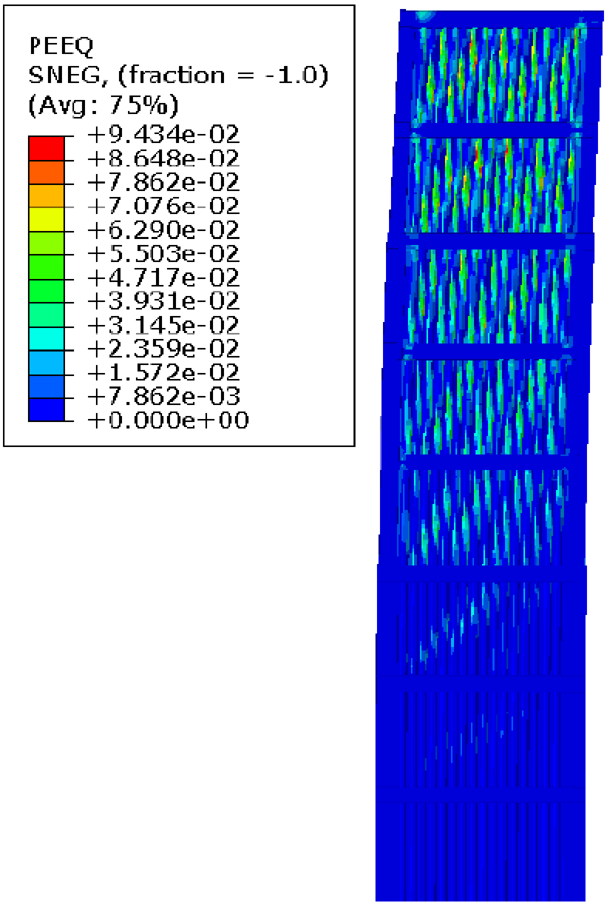
CPSW (30 degrees) under the first mode loading



FPSW under the first mode loading



DCPSW (45 degrees) under the first mode loading



DCPSW (60 degrees) under uniform loading

Fig. 11. The plastic strains of 3 and 5-story frames.

To further evaluate the performance of the corrugated steel plate walls, axial forces in the adjacent columns are considered because the column axial force is a function of global buckling in the walls as well as bending and shear in the beams.

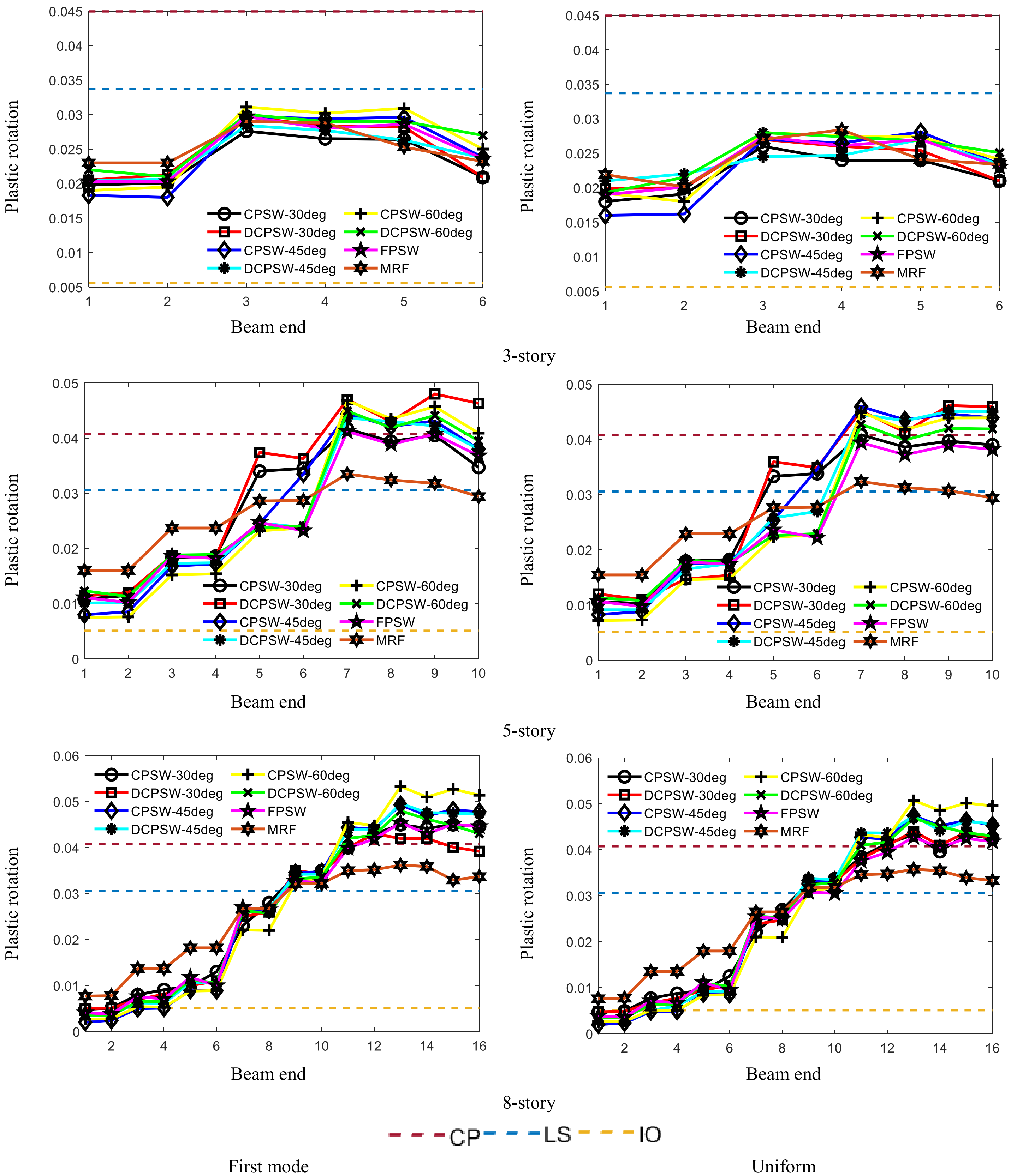


Fig. 12. The plastic rotations at both ends of the beams.

The distribution of axial compressive forces at the column height owing to gravity and lateral loads on the studied structures are shown in Figs. 13 and 14.

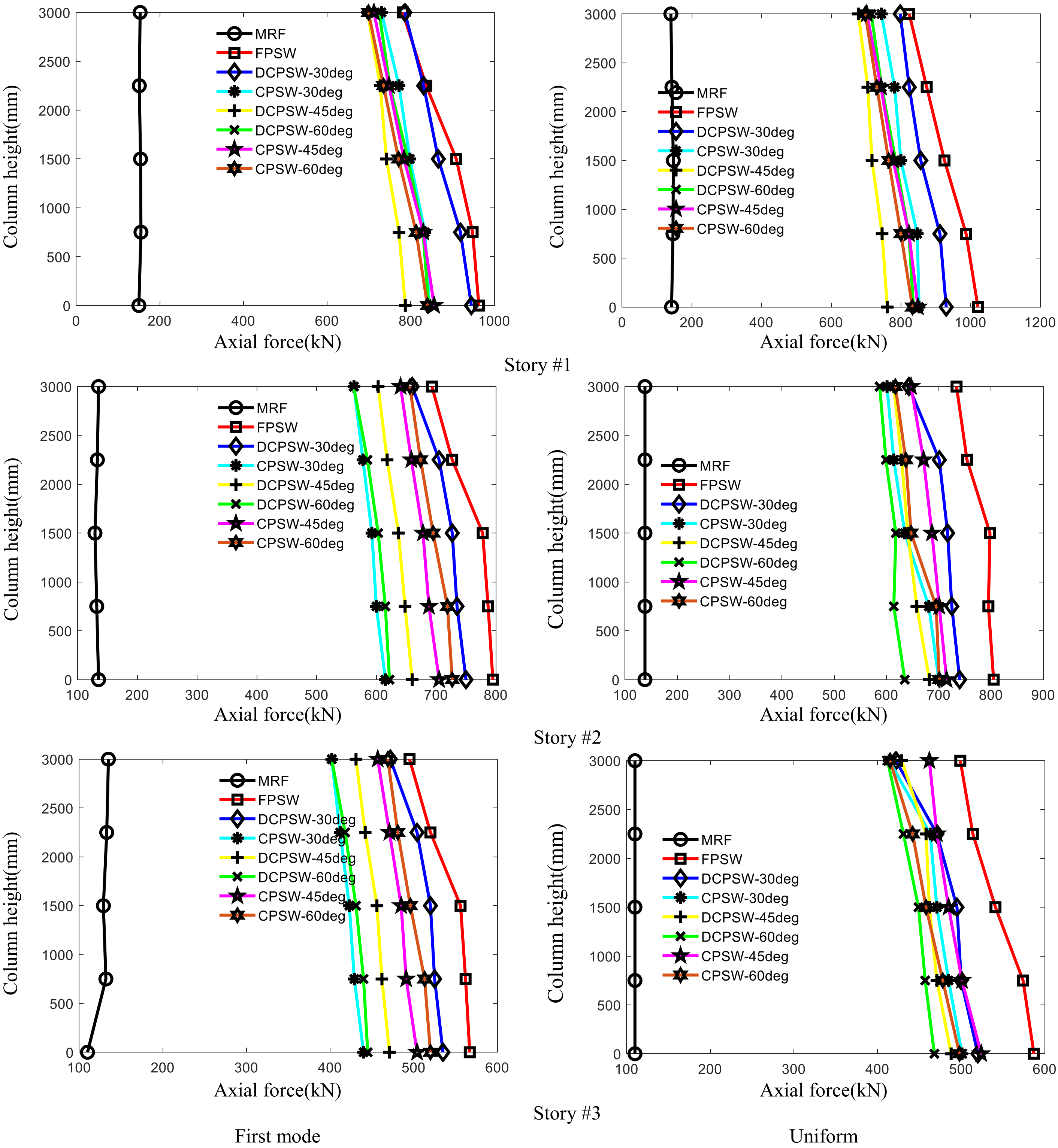


Fig. 13. The axial compressive force (kN) distribution in 3-story frame columns.

According to the figures, the height-wise distribution of compressive axial force in the frames with shear walls is not uniform, in contrast to the moment frames, due to the interaction between the wall and the adjacent leads to the highest compressive axial force in the columns in all the frames with corrugated shear walls because, at this angle, the global buckling strength of the corrugated shear walls columns during the lateral excitation. In addition, the corrugation angle of 30 degrees is less than the other angles.

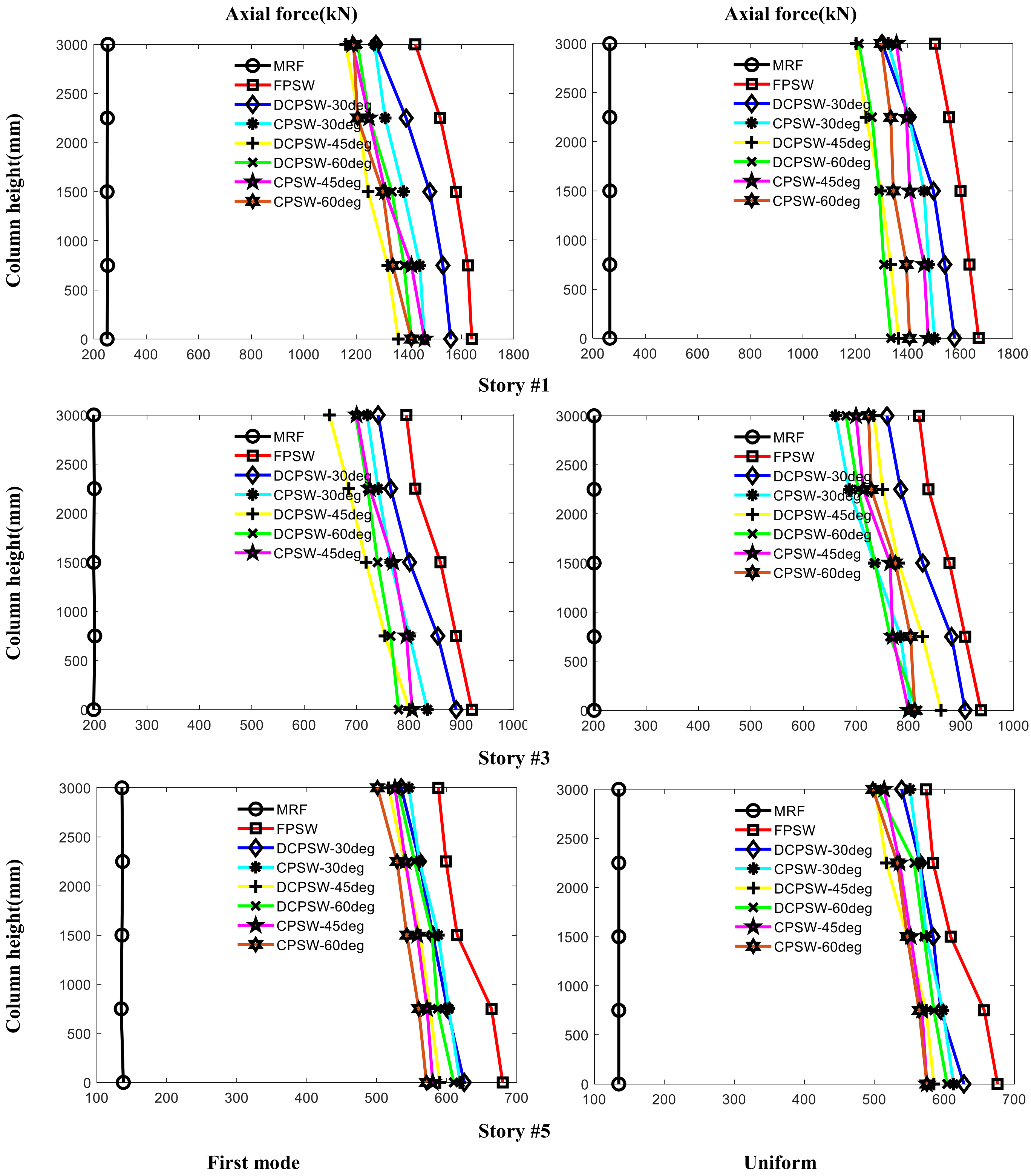


Fig. 14. The axial compressive force (kN) distribution in 5-story frame columns.

The figures show that the frames with the FPSWs have the highest axial forces. As we know, the axial force of the columns in the frames with steel shear walls is greater than that of the frames without steel shear walls due to increased lateral stiffness. However, these values of axial forces cannot be solely attributed to lateral stiffness because the frames with corrugated walls have stiffness values that are close to those of FPSWs.

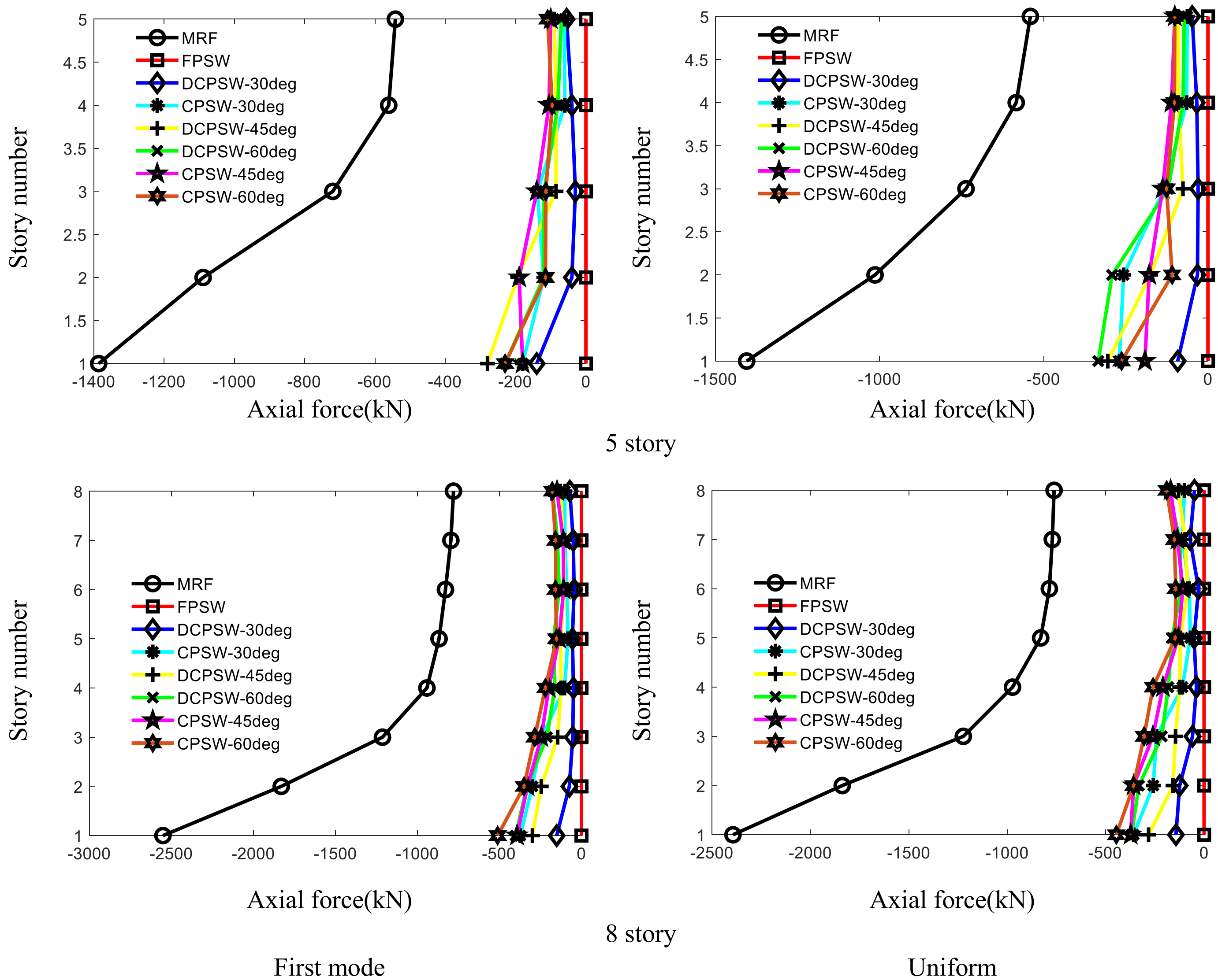


Fig. 15. The difference between the axial forces (kN) of columns in 5 and 8-story frames

Therefore, the column axial forces in the frames with corrugated walls should not be smaller than axial forces in the frames with FPSWs, while this does not comply with Figs. 13 and 14. The disparity stems from the post-buckling behavior of steel shear walls so that the diagonal tension field formed in FPSWs causes higher axial forces in the frames with the FPSWs. The diagonal tension field occurs following the shear buckling of steel plates and as a result of a truss-like mechanism between the steel wall and the surrounding frame members. Fig. 15 depicts a comparison between the compressive axial forces of the columns in the 5 and 8-story frames infilled with FPSWs and the corrugated shear walls. The difference between the values related to FPSWs and the corrugated walls is more pronounced in the lower stories, such that the difference may be considered trivial in the last story. This occurs due to the reduction of wall thickness in the upper stories and consequently their global buckling. In the upper stories, the corrugated shear walls undergo global buckling with a limited excursion to the inelastic range, which causes the difference between the values of compressive axial forces between flat and corrugated walls to be at least. The figure also shows that the axial compressive force in columns adjacent to

DCPSWs is somewhat higher compared to columns adjacent to CSPSWs. This slight discrepancy can be attributed to the tension field induced by global buckling under lateral excitations. As mentioned, the main advantage of CPSWs over FPSWs is their elastic buckling strength, so that the plate lateral corrugations somehow create a lateral support and global buckling of the corrugated walls would more likely be inelastic. In this case, before global inelastic buckling, CPSWs may fail due to local buckling. Local buckling in CPSWs causes their shear strength to be less than the shear strength of FPSWs. For DCPSWs, the situation is similar to CPSWs, i.e., the two-sided plate corrugations improve their buckling strength. However, inelastic deformations in the frames with DCPSWs are greater than the frames with CPSWs (Fig. 11), indicating that the failure of the former frames is associated with less local buckling and global inelastic buckling is more probable.

Hence, the axial compressive forces of columns adjacent to DCPSWs are slightly larger than those of CPSWs. Indeed, because of reduced local buckling in DCPSWs, their shear strength is close to and, in some cases, exceeds the FPSWs' shear strength. This behavior can be considered as a comparative advantage of DCPSWs over CPSWs.

It should be noted that to realize the lateral strength variations of the corrugated shear walls local buckling, global buckling, and yielding should be taken into consideration. On the other hand, the effects of the frames' height add to the issue's complexity. Therefore, to scrutinize this issue thoroughly the current level of analyses is not sufficient and more robust dynamic analyses are required.

5. Conclusion

Structural models, including 48 two-dimensional 3, 5, and 8-story frames were studied and compared using the FE analyses, once without shear walls, once with flat plate steel shear walls, and once with corrugated steel shear walls. Single-layer (CPSW) and double-layer (DCPSW) corrugated steel plate shear walls were modeled with three corrugation angles of 30, 45, and 60 degrees, and were evaluated using the displacement control monotonic pushover technique under two lateral load distributions of uniform and the first vibration mode.

According to the results, corrugated shear walls have less strength than FPSWs. This difference is attributed to local buckling in these walls. The lateral strength of the frames with DCPSWs is higher than that of the frames with CPSWs. The corrugation angle of 60 degrees corresponds to the largest lateral strength in the corrugated walls, especially in 5 and 8-story frames. The initial stiffness of the frames with CPSWs is almost the same as that of the frames with DCPSWs, and the corrugated walls with a corrugation angle of 30 degrees possess the highest lateral stiffness. In addition, in frames with steel shear walls, the uniform lateral loading leads to higher lateral strength and stiffness compared to the first mode loading.

The ductility of the corrugated steel plate walls is higher than FPSWs. With the corrugated steel walls, the ductility of 3- and 5-story frames is increased more than that of the 8-story frame. Furthermore, the ductility increment under the first mode lateral loading is significantly higher compared to the uniform lateral loading.

The frames with DCPSWs are associated with the highest inelastic deformations. Both ends of the beams experience significant plastic rotations due to the shear wall yielding under lateral loads. With the tension field development after global buckling in steel plates, the column axial forces increase, which explains why the highest amount of axial forces is related to the frames with FPSWs.

The corrugation angle of 30 degrees corresponds to the highest axial forces in the columns adjacent to the corrugated walls. The compressive axial forces in the columns adjacent to DCPSWs, on the other hand, are somewhat greater than those in the columns adjacent to CSPSWs because the frames with DCPSWs experience more inelastic deformations.

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Conflict of interest

The authors have no relevant financial or non-financial interests to disclose. On behalf of all authors, the corresponding author states that there is no conflict of interest. All authors read and approved the final manuscript.

Author contributions

All authors contributed to the study conception and design. Material preparation, data collection and analysis were performed by All authors. All authors read and approved the final manuscript.

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