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The Effects of Bolt Spacing on Composite Shear Wall Behavior

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***Abstract*—**Composite steel shear wall is a lateral load resisting system which consists of a steel plate with concrete wall attached to one or both sides to prevent it from elastic buckling. The composite behavior is ensured by utilizing high-strength bolts. This paper investigates the effect of distance between bolts, and for this purpose 14 one-story one-bay specimens with various bolts spacing were modeled by finite element code which is developed by the authors. To verify the model, numerical results were compared with a valid experiment which illustrate proper agreement. Results depict increasing the distance between bolts would improve the seismic behavior, however, this increase must be limited, because of large distances will cause widespread buckling of the steel plate in free subpanels between bolts and would result in no improvement. By comparing the results in elastic region, it was observed initial stiffness is not affected by changing the distance.

***Keywords*—**Composite steel shear wall, bolt, buckling, finite element.

I. Introduction

COMPOSITE Steel Shear Wall (CSSW) is a developed form of stiffened steel shear wall, in which concrete cover is manipulated instead of metal stiffeners. This concrete cover must have a minimum longitudinal reinforcement ratio of 0.0025 which is necessary for controlling out-of-plane displacement of the system under cyclic loading [1]. However, the limited thickness of the cover implies that no confining shear reinforcement would be applicable. The framing of the system is also prepared by using relatively stiff beams and columns. The experimental project carried out by A. Astaneh-Asl, and as well his project is the most important work in the field of CSSW system, so the aim of the project was to test traditional and innovative CSSWs cyclically (under cyclic loading) and to propose seismic design recommendations. The difference between traditional and innovative walls was the presence of a gap around the concrete cover in the latter one. Results demonstrate that this gap leads to a more ductile behavior [1]. Another investigation on the behavior of CSSW system has been conducted by F. Hatami and A. Rahai, which includes both numerical and experimental works [2],[3]. These researchers finally proposed a formula for optimum thickness of concrete cover [3].

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A. Arabzadeh *et al.* studied extensive experimental of one and three-story composite steel plate shear walls (CSPSW). The results depicted a proper agreement for the recommended values of (b/t) by an AISC code for preventing plate buckling (b is the spacing of bolts) [4].

In the case of both-sided concrete having concrete on both sides of the panel, XB. Ma *et al.* suggested an equivalent simplified model, based on eccentric cross-bracing model, for this composite system. Generally, using concrete on both sides of steel plate would improve system behavior, although it is uneconomical than the one-sided case [5]. Furthermore, using high-strength concrete would reduce the damage to RC cover, although it would not seriously affect the strength of the system [4]. Attaching concrete to just one side of the steel plate would provide a kind of buckling problem named "contact problem", in which the plate is restrained in the direction of the stiffeners, but free in opposite. P. Seide was the first researcher to study this kind of problem, who achieved about 33% increase in compressive buckling strength of a simply-supported long plate, by using rigid constraints (Foundation) instead of unrestrained condition [6]. This increase is about 26% and 34% for shear buckling strength of a rigid-constrained long plate with respectively simply-supported and clamped boundary conditions [7]. Seide's research was also extended numerically to account for different material properties and boundary conditions by K.W. Shahwan and A.M. Waas [8] and contact problem between two adjacent delaminated plates of different thicknesses and material properties has been formulated by Ma *et al*. [9]. Using connectors between the plate and its foundation would make the problem more complicated. J. Cai and Y.L. Long estimated the effect of binding bars on the buckling of steel plates in rectangular concrete-filled tube (CFT) columns [10] .

By conducting a theoretical study, they determined a relationship between the distance between the bars (connectors) and elastic buckling strength of the steel tube. A. Arabzadeh and his colleagues investigated the contact problem for CSSW system where they determined the elastic buckling coefficient for stiffened plates for different number of bolts. They concluded that the influence of concrete constraint is more highlighted in case of using a small number of bolts, as the interaction between steel and concrete panel is much larger and less likely to provide stiffness with the concrete cover [11].

In spite of the some papers were mentioned, so far there is little information about CSSW in seismic codes, but even these limited specifications about steel or concrete wall are mentioned without taking into account their interaction in composite behavior.

This necessitates more research in order to improve understanding of this complicated structural system.

In this paper the distance between bolts, as an important parameter is investigated numerically and for this purpose, a finite element analysis has been conducted by the author. The concrete wall which study in this paper has no gap around it and is merely attached to one side of the steel plate. The plate is assumed to have continuous connection to the surrounding frame. Also, the connections between beams and columns are considered rigid in which the stiffeners of the columns connected to the top beam have been fully modeled for studying specimens. Furthermore, surrounding frames are assumed to be interior frames of a generic structure, so that they only contribute to resisting lateral loads by forming a dual system together with infill walls, in addition, most of the gravity loads are carried by relatively stiff corner columns built in concrete-filled tube sections with little load remaining for interior frames. Hence, the effect of gravity load is not taken into account in the model analysis.

II. Numerical Modeling and Analysis Method

In the developed Finite Element (FE) code, an eight-node brick element was used for surrounding beams and columns and also for stiffeners attached to the webs of the columns on both sides of the top beam. This brick element was utilized for concrete cover as well, since it can absolutely model the two important interactions of this cover: *first*, connection to the bolts and *second*, frictionless normal contact with steel plates which both applied via the adjacent nodes in different parts. Although there is no gap around the concrete cover in the model, however an infinitesimal space (2×10^{-2}mm) has been considered between the frame and infill concrete and as well as between the steel plate and concrete wall to obtain a more realistic condition for contact problem. For steel plate a four-node quadratic shell element was selected to model such a thin component. A two-node linear beam element with six degrees of freedom per node (i.e. three translational components and three rotational components) was selected for bolts. The nodes of this element were coupled with the same nodes on concrete cover and steel plate, so that they have consistent deformations in the location of these nodes. The inefficiency of the brick element for the concrete cover in modeling the rotational degrees of freedom has provided a desirable situation, because bolts should be released in order to have free in-plane rotation in connection to the cover, similar to what was observed in experiments. In analysis of the models, smaller meshes are used for concrete and steel walls (beside the surrounding frame), in order to attain an accurate result where free subpanels and bolts are located. These infill walls (steel plate with concrete cover) must have the same mesh sizes, so they bind together on the location of adjacent nodes due to supply precise Finite Element (FE) analysis.

A schematic illustration of meshing condition is demonstrated in Fig. 1. The effect of rebar has been included in analysis by using bilinear elasto-plastic (without hardening) behavior for the concrete element, which eliminates the need for modeling concrete cracking and helps get rid of difficulties raised by separate reinforcement modeling.

As suggested by A. Astaneh-Asl [12], this assumption would not lead to considerable errors and the results would have proper consistency with experimental data.

Loading of the model was carried out by pushing nodes at the top beam laterally and incrementally in a displacement controlled manner. As far as nonlinear static analysis of the models is concerned, an iterative solution based on well-known Newton-Raphson method was employed which takes into account nonlinear geometry.

Fixed boundary condition has also been applied to the base of the model, in accordance with real condition. A schematic illustration of loading condition is shown in Fig. 2.

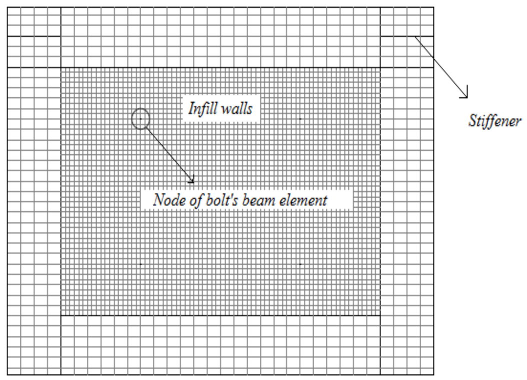

Fig. 1 A schematic illustration of meshing condition

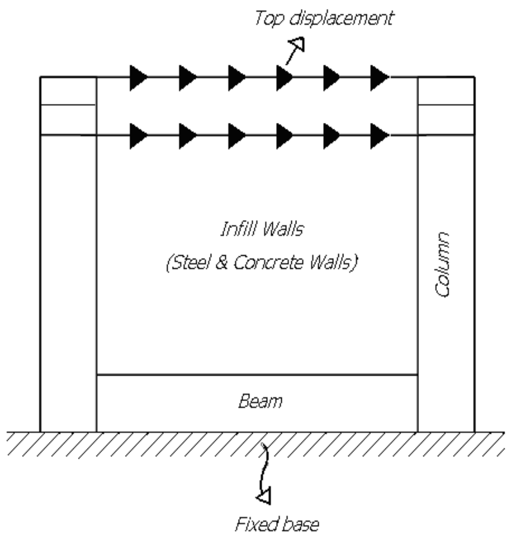

Fig. 2 A schematic illustration of loading condition

To investigate the validity of the modeling, the specimen used by Astaneh-Asl (one without gap) [1] was modeled and results were compared with experimental data.

The materials used in this experiment include: (a) A572Gr50 steel with yield stress of 3515.34 kg/cm^2 (50 ksi) and A36 steel with yield stress of 2531.05 kg/cm^2 (36 ksi) for respectively boundary frame and steel plate, (b) A325 bolt with tensile strength of 6327.62 kg/cm^2 (90 ksi) and (c) normal-weight concrete with f_c of 281.22 kg/cm^2 (4000 psi).

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Fig. 3 (a) and fig. 3 (b) illustrate numerical and experimental models.

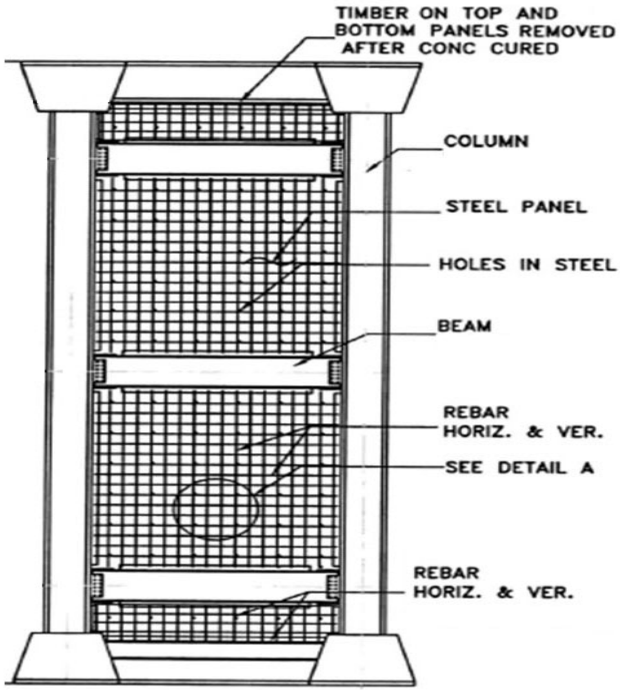

Fig. 3 (a) Experimental models of Astaneh specimen [1]

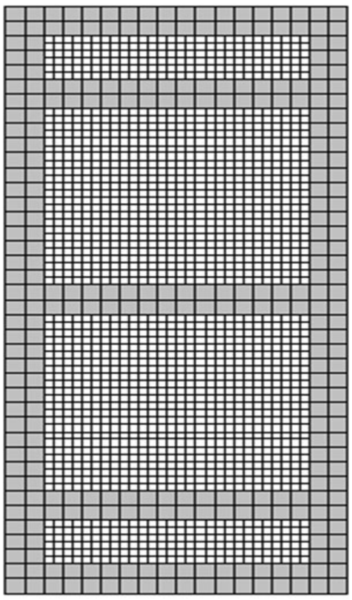

Fig. 3 (b) two-dimensional meshed numerical models [1]

In Fig. 4 the load-displacement curve for the analyzed model is plotted "push-over "curve of test hysteresis. It is clearly shown from the figure that there is proper compatibility between these curves, although the FE model is a little stiffer. It is interesting to see that there is a small drop in the middle of the experimental curve (in top displacement interval of 10.16- 15.24 centimeters) which doesn't exist in the numerical curve.

Cracking of the concrete cover is the reason for this drop which is not considered in the numerical model, as described before. However, the smooth numerical curve obtains the ultimate load and ultimate deformation in the experiment with proper accuracy. In addition to consistency of the curves, the steel wall edges and corners of the concrete walls (especially in the lower full story) around beam to column connections at the top and bottom of the lower story have shown high values of equivalent Von Mises stress (show in detail in Fig. 5 (a) and Fig. 5 (b)), while cracking of concrete wall, severe yielding of steel plate and yielding of beams close to connection to the columns have been observed in experiment, all of which occurring at the same time of having high values of Von Mises stress.

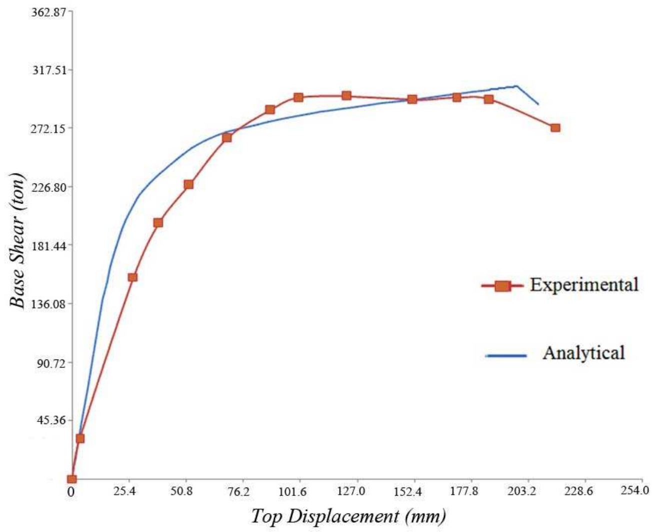

Fig. 4 Comparison of Experimental and analytical curves of Astaneh specimen

The share in the base shear carried by each component of the system (steel plates, columns and concrete covers) was also plotted in Fig. 6. From this figure, the following can be easily indicated: *first*, the steel plate has played the main role in providing ductility for the specimen.

While both the concrete cover and the columns tend to decrease load sharing in inelastic domain at high levels of loading, the steel plate continues to carry load without reduction in strength. *Second*, the contribution of concrete cover to stiffness and shear resistance of the system is limited.

This matches well with the experimental data, where contribution of less than 20% (in both stiffness and ultimate strength) was extracted from this cover. *Third*, evaluating the numerical results show the decrease in load-sharing in columns is undoubtedly related to bending effects caused by applying displacement at the top of this relatively high specimen, which led to columns yielding near the base.

This was clearly observed and notified on the experimental specimen. All the evidence clearly points to the verified modeling procedure used for numerical analysis of the system.

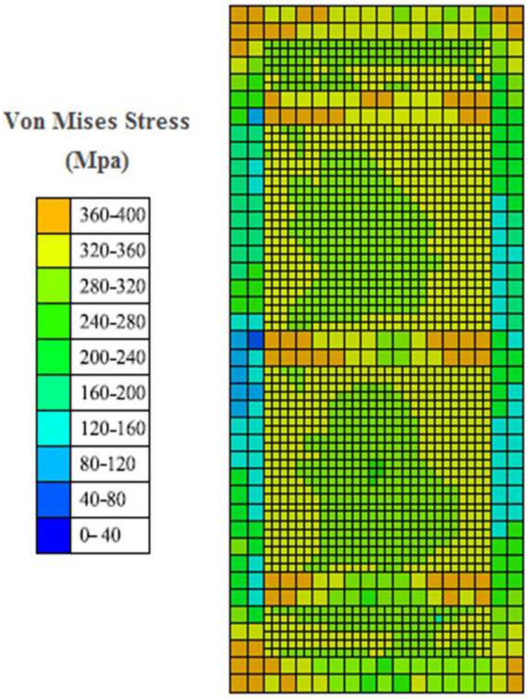

Fig. 5 (a) Von Mises stress distribution for steel

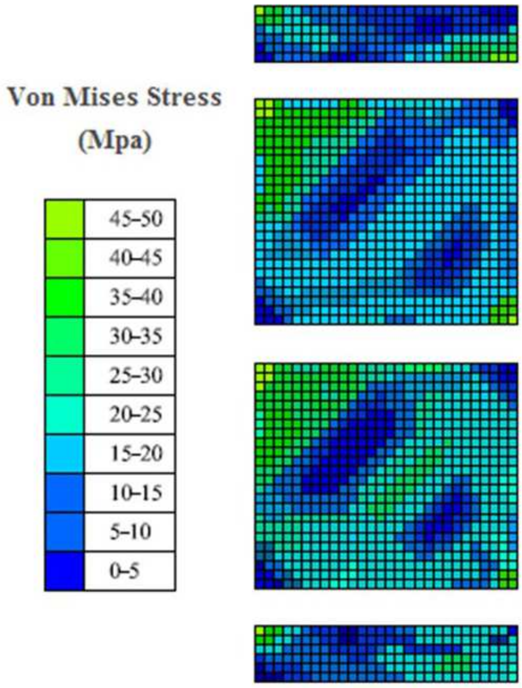

Fig. 5 (b) Von Mises stress distribution for concrete parts of the model

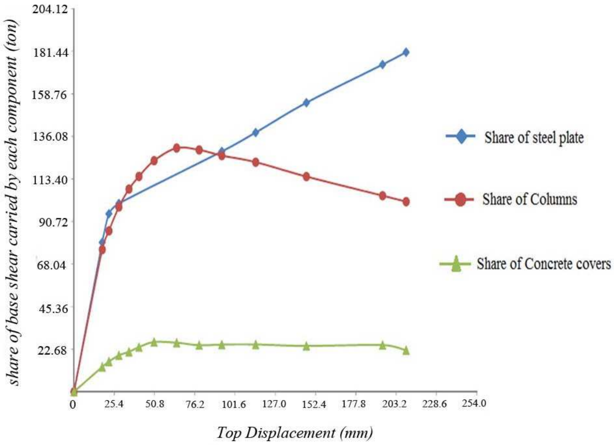

Fig. 6 Proportion of base shear carried by each component of the system

III. Design of the Numerical Specimens

To investigate the effects of distance between bolts, the specimens were first designed as described below before being analyzed and the results were finally compared with different cases. The arrangements of bolts considered in this research are shown in Fig. 7 (a) and Fig. 7 (b). These were namely four-bolt and nine-bolt arrangements by which the panels were divided in to the same number of subpanels in both horizontal and vertical directions. The parameter ″b″ in this figure would be used later to introduce different distances between bolts.

As far as the design of the specimens is concerned ST37 with modulus of elasticity and Poisson's ratio of respectively 2×10^5 Mpa and 0.3 and nominal yield and ultimate stresses of 240 Mpa and 400 Mpa (Fig. 8) was used for steel parts (except bolts) and concrete with specified compressive strength of 28 Mpa and Poisson ratio of 0.2 was used for concrete wall (modulus of elasticity is equal to $5000\sqrt{f'c}\approx25000$ Mpa). A490 High-strength bolts with yield stress and ultimate strength of respectively 900 Mpa and 1000 Mpa were used as connectors. This type of heavy hexagon-head bolt is usually used when diameters over 38.1 mm (1.5in) up to 76.2 mm (3in) are needed [13], similar to bolts studied herein.

All panels have 3m height and 4m width and have 5mm and 8mm thicknesses for steel and concrete walls respectively. These infill walls were designed based on the assumption that the shear force is entirely carried by the steel plate, without any prior-to-yield buckling. Next, the concrete thickness was determined based on the initial stiffness or the ultimate strength (whichever results in thicker cover) equal to that of a metal-stiffened steel wall, with no elastically buckled subpanels.

An I-section IPB600 profile was used for columns and an IPE550 profile for beams, which were designed based on satisfying the width to thickness ratio requirements to avoid buckling, in accordance with AISC seismic provision [14].

However, since the unilateral inelastic buckling is inevitable in steel plates, the frame of the model should be checked to be able to withstand the tension field action of the buckling plate, similar to a thin steel plate shear wall [15].

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In despite of this, utilizing the same procedure for stiffened case would be conservative because buckling of the steel plate is postponed and consequently the effect of tension field action would be weakened.

In analyzing each different arrangement of distances, it is crucial to have stiff bolts which do not reach the yielding point limit in order to achieve a desirable ductile mode of failure. For this purpose, the bolt diameter required for resisting the minimum capacity of steel and concrete walls was first calculated but then a commonly-used bolt with larger diameter from the one calculated was manipulated as a safety measure.

In evaluating the shear capacity of the concrete cover, shear resistance of reinforcing bars was ignored as they were only expected to distribute stress uniformly throughout the panel and control crack propagation. The shear capacity of steel and concrete walls were calculated based on part 17.2 of AISC seismic provision [14] and part 11.9.6 of ACI-318 code [16] respectively.

From these provisions, values of 37ton and 288ton were obtained for shear capacities of concrete cover and steel plate respectively. The minimum required diameter would be 14mm and 10mm for four-bolt and nine-bolt arrangements respectively. Therefore, the 24mm diameter high-strength bolts were used for the studied specimens.

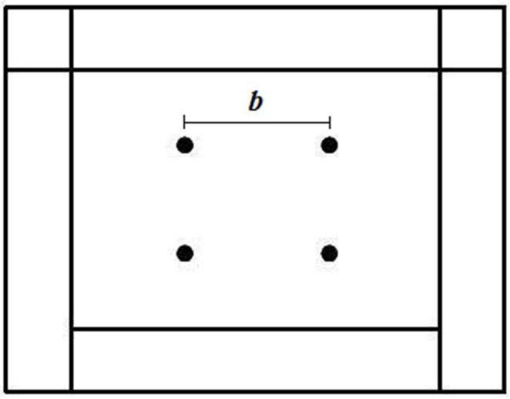


Fig. 7 (a) Arrangements of 4-bolts studied

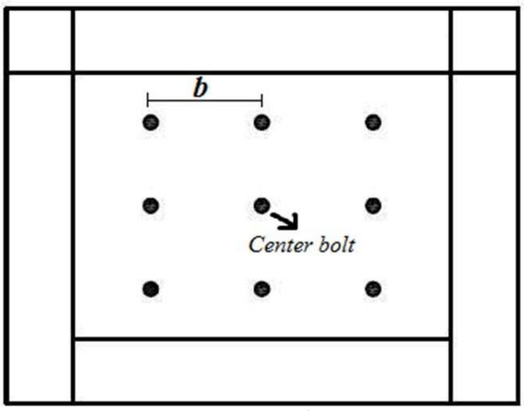


Fig. 7 (b) Arrangements of 9-bolts studied

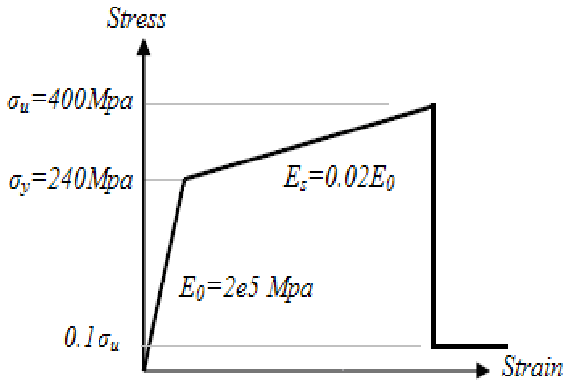


Fig. 8 Stress-Strain curve for steel parts of the models (Frames and steel walls)

IV. Results and Discussions with Specimens

1. Models with four-bolt arrangement

The distance between bolts considered in this arrangement (parameter ″b″ in fig. 7 (a)) contains 8 values of 200 mm, 500 mm, 667 mm, 1333 mm, 2000 mm, 2667 mm, 3000 mm and 3200 mm.

Results depict that despite these specimens had nearly the same elastic behavior up to the shear force of about 370 ton at the base with almost the same initial stiffness equal to 70 ton/mm, their differences appeared in inelastic behavior. Maximum values of lateral displacement and base shear in different specimens of four-bolt arrangement have been compared in bar chart 1.

It can be obviously seen from the bar chart 1 that the specimen with distance of 500mm between bolts has the distinguishing desirable behavior.

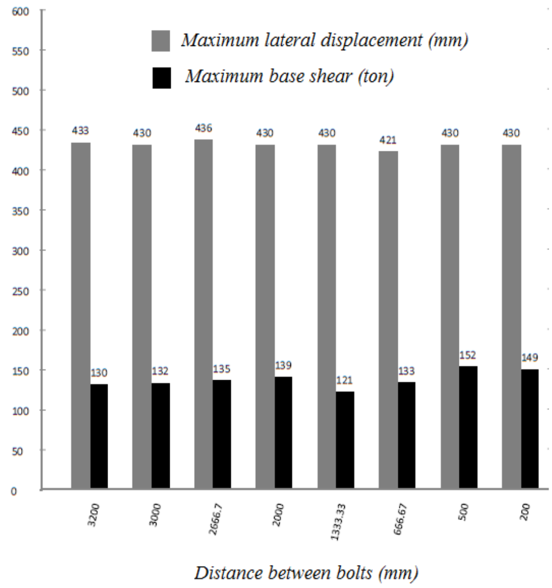
This is due to it has reached the largest inelastic deformation and the most ductile behavior, while achieving the same ultimate strength as the others. Generally, concrete covers in specimens with this arrangement, experienced two different phases of behavior: *first,* at initial phases of loading, in which they carried the shear load through diagonal compression. *Second,* out-of-plane lifting of the concrete cover at high levels of loading resulted in widely extended smooth distribution of stress throughout the cover with some stress concentrations in vicinity of the bolts.

More or less, the steel plate buckles unilaterally toward its free side in all specimens. Furthermore, after the unilateral buckling of the steel plate, the bolts became under relatively large tension, in order to prevent the steel plate from free out-of-plane displacement at the location of the bolts.

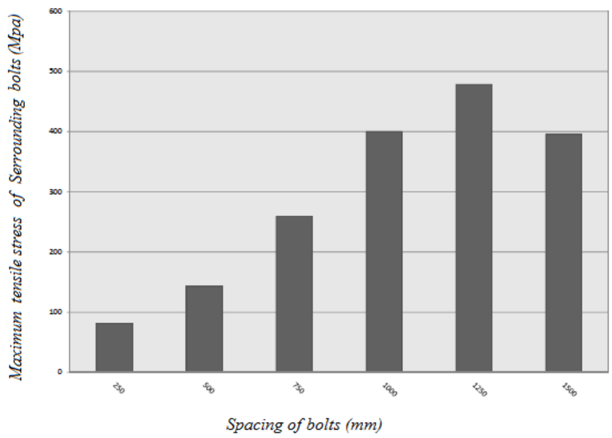
The maximum tensile stress of the surrounding bolts in different specimens with this arrangement is compared in bar chart 2. It can be concluded from this bar chart that increasing the distance between bolts up to a specified limit (i.e. distance of 2667mm) would increase the maximum tension in the bolts, but no more increase is observed by further increase in the distance.

Seeking the reason, results from different specimens show that the unilateral buckling of the steel plate first occurred on unrestrained corners rather than in interior subpanels.

This is especially so, when the distance between bolts are short (such as 200 or 500 mm) which facilitates buckling on the corners, but simultaneously stiffens the interior subpanels. The corner buckling tends to spread into central parts, while high-strength bolts strictly limit out-of-plane displacement of the plate and therefore prevent the buckling from spreading into the interior subpanels (inwards). This prevention creates tension in the bolts. When the distance between bolts increases up to a certain limit (i.e. 2667mm), the restriction from bolts would become more severe and the tensile stress of bolts would consequently increase. On the contrary, too large interior spaces between bolts (like what occurred in specimens with 3000 or 3200mm spacing) would lead to the buckling of the plate. This takes place only in the interior subpanels with infinitesimal out-of-plane displacements on the edges, and as a result restriction at the location of bolts and tension in the surrounding bolts would decrease.



Bar chart 1 Comparison of Maximums of Lateral displacement and base shear for different distance between bolts in four-bolt arrangements



Bar chart 2 Comparison of maximum tensile stress of the corner bolts in different specimens of four-bolt arrangement model

2. Models with nine-bolt arrangement

The distance between bolts considered in this arrangement (parameter ″b″ in Fig. 7 (b)) contains 6 values of 250 mm, 500 mm, 750 mm, 1000 mm, 1250 mm and 1500 mm. In Fig. 9, The buckling pattern of the steel plate for the specimen with 1000 mm of distance between the bolts is compared with that of the specimen with 1333mm of distance between the bolts with four-bolt arrangement.

The reason for choosing these specimens for comparison is that the panel is divided into equal parts in these specimens, one to nine subpanels and the other one to sixteen. This figure obviously demonstrates that for a certain panel, increasing the number of bolts would increase the buckling constraint by bolts, since smaller free subpanels would form. Similar to four-bolt arrangement, result indicates that the same elastic behavior occurs with similar initial stiffness for different distances between bolts, though different values of ultimate strength and inelastic deformation were obtained.

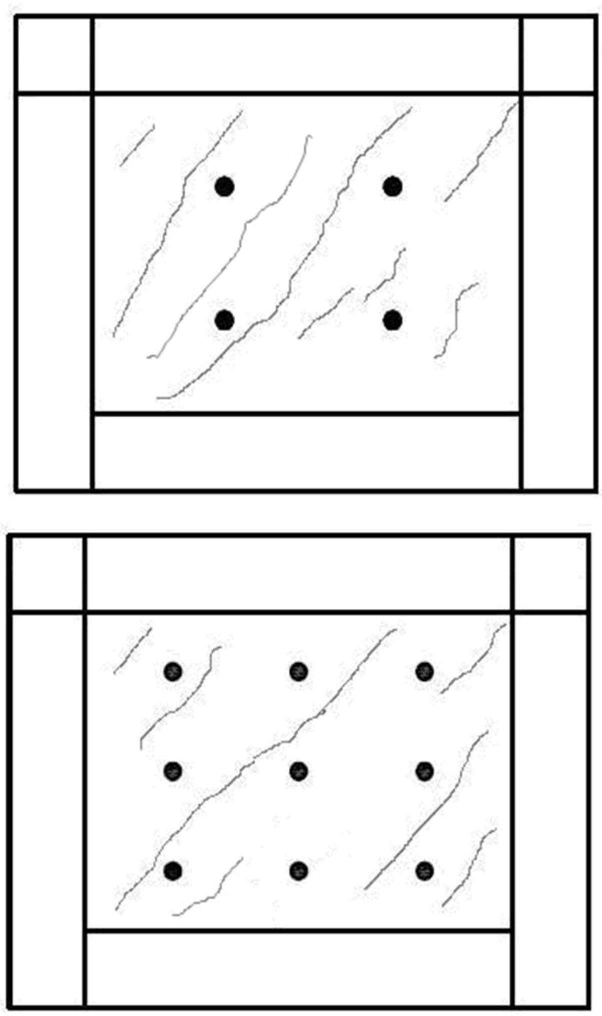
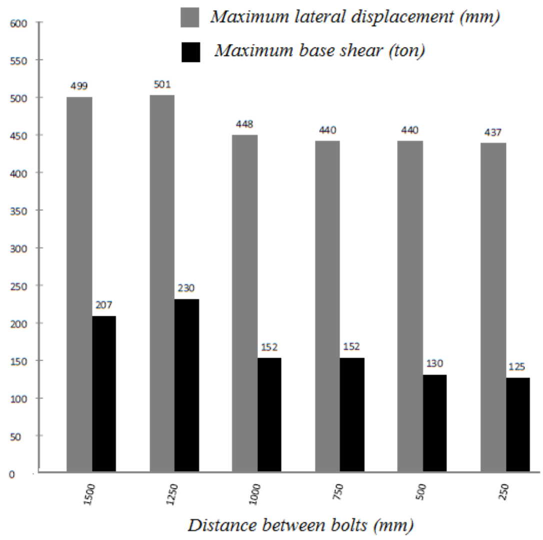


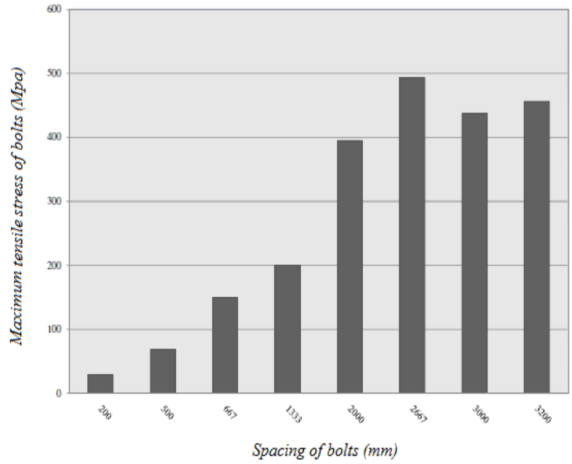
Fig. 9 Patterns of unilateral buckling in different arrangements studied in this paper

(Specimen with distance of 500 mm in four-bolt arrangement & one with distance of 1250 mm in nine-bolt arrangement)

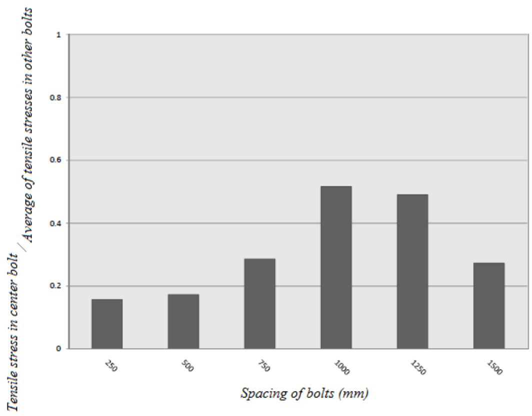
In bar chart 3, Maximum values of lateral displacement and base shear are compared for different specimens with nine-bolt arrangements. The comparison of tensile stress in corner bolts for different specimens is also shown in bar chart 4. Bar chart 3 demonstrates that the specimen with distance of 1250 mm between its bolts had a more ductile behavior than the others. In addition, bar chart 4 is in agreement with the conclusion mentioned and justified for four-bolt arrangement regarding tensile stress of surrounding bolts. One of the interesting results observed in nine-bolt arrangement is that the small value of tension in the center bolt (introduced in Fig. 6 (b)), compared to its surrounding bolts. In bar chart 5, the ratio of tension created in the center bolt, respect to average tension in other bolts is compared for different distances of nine-bolt arrangement which illustrates that this ratio has never exceeded 50% in the studied specimens. This indicates that the interior subpanels are stiffer in protecting buckling than surrounding subpanels. This is guaranteed by using stiff surrounding bolts which prevent corner buckling from spreading into central subpanels. This stiff behavior is more severe, when the distance between bolts is small enough (such as specimens with distances of 250 mm and 500 mm) to entirely avoid buckling in the interior subpanels.



Bar chart 3 Comparison of Maximums of Lateral displacement and base shear for different distance between bolts in nine-bolt arrangements



Bar chart 4 Comparison of maximum tensile stress of the corner bolts in different distance between nine-bolt arrangement models



Bar chart 5 Comparing the ratio of tensile stress in center bolt divided to average of tensile stresses obtained in other bolts

V. Conclusion

In this paper the numerical model for composite steel plate shear wall (CSSW) with two different arrangements of bolts, namely four-bolt and nine-bolt arrangements, each with different bolt spacing, has been analyzed by using a finite element code. The following were concluded:

- Results of the study arrangements depicts that increasing the distance between bolts up to a specified point (i.e. 500 mm in four-bolt and 1250 mm in nine-bolt arrangement) would improve system behavior by stiffening the surrounding subpanels while simultaneously preventing the interior subpanel from buckling. But further increase in spacing would create too large interior subpanels which would facilitate buckling of the steel plate in these areas. Hence no more desirable behavior could be anticipated.
- Increasing distance between the bolts up to a specified value (i.e. 2667 mm in four-bolt and 12.50 mm in nine-bolt arrangement) would increase the maximum tension in the surrounding bolts. This occurs when the distance between the bolts is lower than above-mentioned values and buckling of the plate effortlessly takes place on unrestrained corner subpanels. The bolts located at the corners should

prevent this outer casing in a buckling from spreading to the central part of the panel. The closer the distance is to the mentioned limits, the more restraint is expected from the corner bolts and more tension would be obtained. But beyond that the corner subpanels are small enough to make buckling impossible at the corners and instead, buckling would occur in the central subpanel. As a result, less constraint is needed at the location of bolts which decreases tension in the bolts.

- Regardless of the distance between bolts, the bolt located at the center of the panel in nine-bolt arrangement is under a relatively small amount of tension compared to the surrounding bolts. This illustrates that interior subpanels are subjected to less unilateral buckling than the ones at the corners, which highlights the important role of high-strength bolts in stiffening the central subpanels. This therefore leads to a decrease in tensile restriction needed in the center of the panel to prevent out-of-plane displacement. This stiffening becomes more severe when the distance between bolts is small enough (such as 250 mm or 500 mm) to make buckling of the steel plate at interior subpanels completely impossible. In this paper an upper limit of 0.5 is obtained for the ratio of tension in the center bolt relative to average tension in other bolts.
- The same initial stiffness is obtained from other specimens with different distances between bolts, either in four-bolt arrangement or in nine-bolt one. This seems reasonable due to the fact that in the elastic region, the relative movement between the steel and concrete walls is small enough to help high-strength bolts (help/force) different specimens behave with similar stiffness.
- Finite element modeling is able to predict the behavior of the system precisely and accurately. This is evident from comparing load-displacement curves and locations with high stress in Astaneh's experimental specimen with similar numerical model analyzed by the authors.

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