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EVALUATING THE EFFECTS OF DISTANCE BETWEEN BOLTS ON THE BEHAVIOR OF COMPOSITE STEEL SHEAR WALL

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ABSTRACT

Composite steel shear wall is a lateral load resisting system that consists of steel plate as a primary component with concrete wall (cover) attached to one side or both sides of the plate to prevent it from elastic buckling. The composite action of the system is ensured by using high-strength bolts. This paper investigates the effects of the distance between bolts on the behavior of the system. For this purpose, 14 one story one bay specimens with various distances between bolts were modeled and analyzed in the finite element software ABAQUS. To verify the ability of the model, numerical results were compared with a valid experiment, which shows very good agreement. Results demonstrate that increasing distance between bolts would improve the seismic behavior of the system. However, this increase in distance should be limited, since permission to widespread buckling of steel plate in free subpanels between bolts would result in no more improvement of the behavior. By comparing the results in elastic region, it was clearly observed that the initial stiffness of the system is not affected by changing the distance between bolts.

Keywords: composite steel shear wall; bolt; buckling; FE analysis; behavior; tension

1. INTRODUCTION

Composite Steel Shear Wall (CSSW) is a developed form of stiffened steel wall, in which metal stiffeners were replaced by concrete cover. This concrete cover should 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 here. The framing of the system is also prepared by using relatively stiff beams and columns. The

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experimental project carried out by Astaneh-Asl and his coworker is the most important work in the field of CSSW system, where the aim of the project was to test traditional and innovative CSSWs cyclically and to propose seismic design recommendations [1]. The difference between traditional and innovative walls was the presence of a gap around the concrete cover in the latter one. Results demonstrated that this gap leads to a more ductile behavior [1]. Another investigation on the behavior of CSSW system has been conducted by Hatami and Rahai, which includes both numerical and experimental works [2, 3]. These researchers finally proposed a formula for optimum thickness of concrete cover [3]. Furthermore, Hatami and Sehri have shown that for a constant concrete cover thickness, increasing steel plate thickness up to an optimum value would be effective in reducing out-of-plane displacement of the plate [4]. In the case of both-sided concrete, Ma *et al* suggested an equivalent simplified model, based on eccentric cross-bracing model, for this composite system [5]. Generally, using concrete on both sides of steel plate would improve system behavior, although it is less economical than one-sided case [6]. Furthermore, using high-strength concrete would reduce the damage to RC cover, although it would not seriously affect the strength of the system [6].

Attaching concrete to just one side of the steel plate would provide a kind of buckling problem entitled "contact problem", in which the plate is restrained in direction of the stiffener, but free in opposite. 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 constraint (Foundation) instead of unrestrained condition [7]. This increase is about 26% and 34% for shear buckling strength of a rigidly-constrained long plate with respectively simply-supported and clamped boundary conditions [8]. Seide's research was also extended numerically to different material properties and boundary conditions by Shahwan and Waas [9] and contact problem between two adjacent delaminated plates of different thicknesses and material properties has been formulated by Ma *et al* [10]. Using connectors between the plate and its foundation would make the problem more complicated. Cai and Long estimated the effect of binding bars on the buckling of steel plates in rectangular concrete-filled tube (CFT) columns [11], whereby fulfilling a theoretical study, they determined a relationship between the distance of the bars (connectors) and elastic buckling strength of steel tube. Arabzadeh and his coworkers have investigated the contact problem for CSSW system, where they determined the elastic buckling coefficient of stiffened plate for different number of bolts. They concluded that the influence of concrete constraint is more highlighted in case of using small number of bolts, as the interaction between steel and concrete panel is much larger and less likely to provide stiffness with concrete cover [12].

In spite of the works described above, not only there is little about CSSW in seismic codes, but even these limited specifications on steel or concrete wall is without considering their interaction in the composite action. This necessitates more research for better 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 authors, using finite element software ABAQUS. The concrete wall studied in this paper has no gap around it and just attached to one side of the steel plate. The plate is assumed to have continuous connection with the surrounding frame. Also, the connection between beams and columns is considered rigid, in which the stiffeners of the column in connection to top beam has been fully modeled in the studied specimens. Furthermore, surrounding frames are assumed as interior frames of a generic structure, so that they just contribute in resisting lateral load by introducing a dual system together with infill walls. Instead, most of the gravity load is carried by relatively stiff corner columns built in concrete-filled tube sections, with little remains for interior frames. Therefore, the effect of gravity load is not considered in the analysis of the models.

2. NUMERICAL MODELING AND ANALYSIS METHOD

In the developed Finite Element (FE) analysis, an 8-node brick element was used for surrounding beams and columns and also for stiffeners attached to the webs of the columns at both sides of the top beam. This brick element was used for concrete cover too, since it can easily model the two important interactions of this cover: 1) connection to bolts and 2) frictionless normal contact with steel plates, both applied via the adjacent nodes of different parts. Although there is no gap around the concrete cover in the model, but a negligible distance of low order (of 2×10^{-2}mm) has been considered between frame and infill concrete and also between steel plate and concrete wall to make a realistic condition for contact problem. For steel plate, a 4-node quadratic shell element was selected for modeling such a thin component. A 2-node linear beam element with 6 degrees of freedom per node (three translational and three rotational) 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 brick element of concrete cover in modeling the rotational degree of freedom has provided a desirable situation, because bolts should be released to have free in-plane rotation in connection to cover, similar to what observed in experiments. In analysis of the models, smaller meshes were used in concrete and steel walls (than surrounding frame), in order to get a more accurate result at the location of free subpanels and bolts. These infill walls (steel plate and concrete cover) should have same mesh sizes, so that they connect to each other on the location of adjacent nodes in order to provide more stable and accurate FE analysis. A schematic illustration of meshing condition is shown in Figure 1. The contribution of rebar has been included in this analysis by using bilinear elastic-plastic (without hardening) behavior for concrete element, which eliminates the need for modeling concrete cracking and helps get rid of difficulties raised from separate modeling of reinforcement. As suggested by Astaneh-Asl [13], this assumption would not lead to considerable errors and results would have good consistence with experimental data.

Loading of the model was introduced by pushing nodes on top beam level laterally and incrementally in the displacement-control manner. For nonlinear static analysis of the models, 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 agreement with actual condition. A schematic illustration of loading condition is shown in Figure 2.

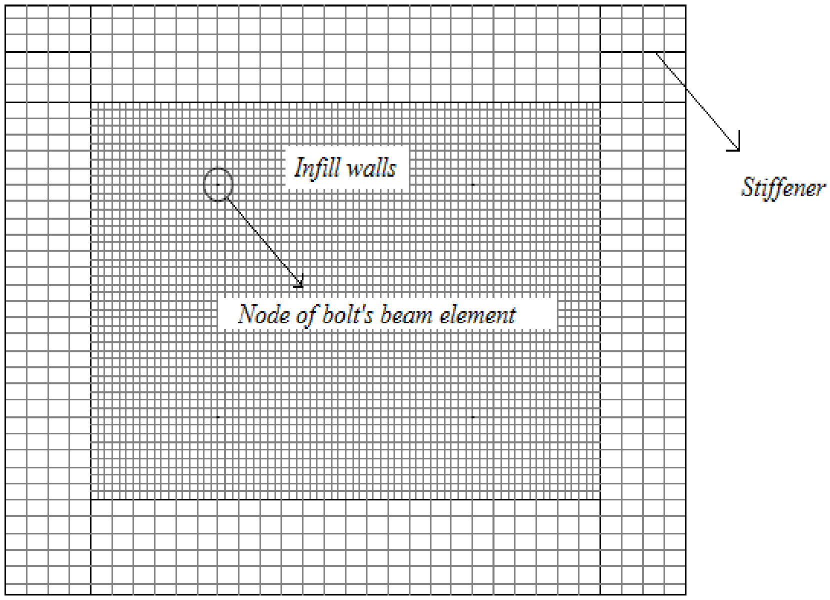

Figure 1. A schematic illustration of meshing condition

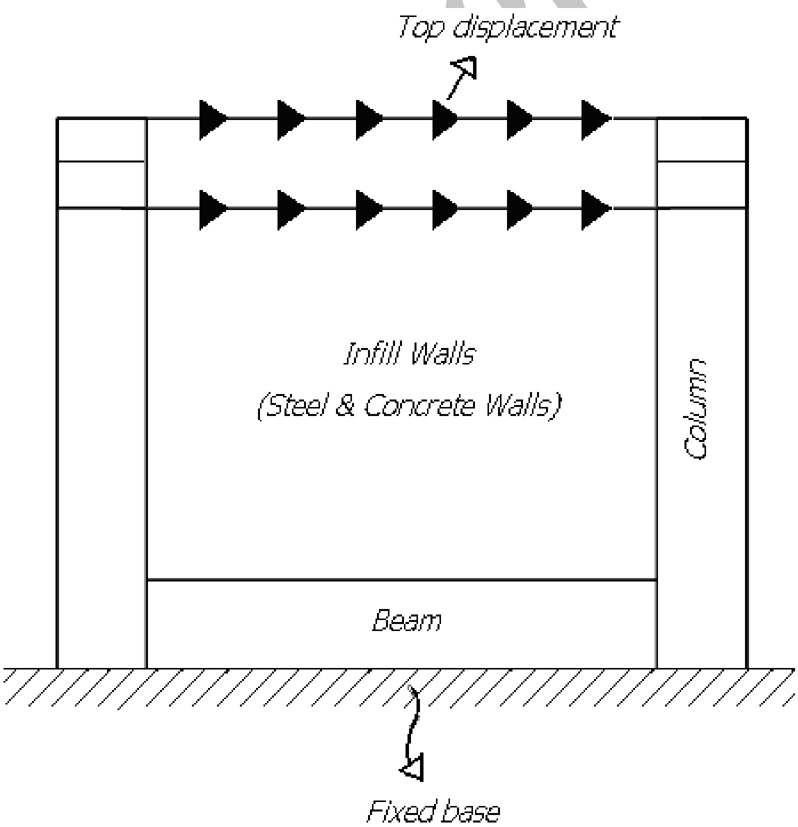

Figure 2. A schematic illustration of loading condition

To control the validity of modeling, Astaneh-Asl specimen (one without gap) [1] was modeled and results were compared with experimental data. The material used in this experiment include: a) A572Gr50 steel with yield stress of 50 *ksi* and A36 steel with yield stress of 36 *ksi* for respectively boundary frame and steel plate, b) A325 bolt with tensile strength of 90 *ksi* and finally, c) normal-weight concrete with f'_c of 4000 *psi*. Figure 3 shows numerical and experimental models. In Figure 4, the Load-displacement curve of the analyzed model is plotted versus “push-over” curve of the test hysteresis. It is clear from the figure that very good agreement is observed between these curves, although 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 4-6 inches), that is not observed in the numerical curve. Cracking of concrete cover is the reason for this drop, which is not considered in the numerical model, as described previously. However, the smooth numerical curve reaches the ultimate load and ultimate deformation of the experiment with a very good accuracy. In addition to consistency of the curves, steel wall edges and corners of concrete walls (especially in the lower full story) and also, area of beams near connection to columns at top and bottom of lower full story have shown high value of equivalent von mises stress (shown in detail in Figure 5), while cracking of concrete wall and severe yielding of steel plate and also yielding of beams near connection to columns has been observed in experiment, all at the same location of obtaining high values of von mises stresses.

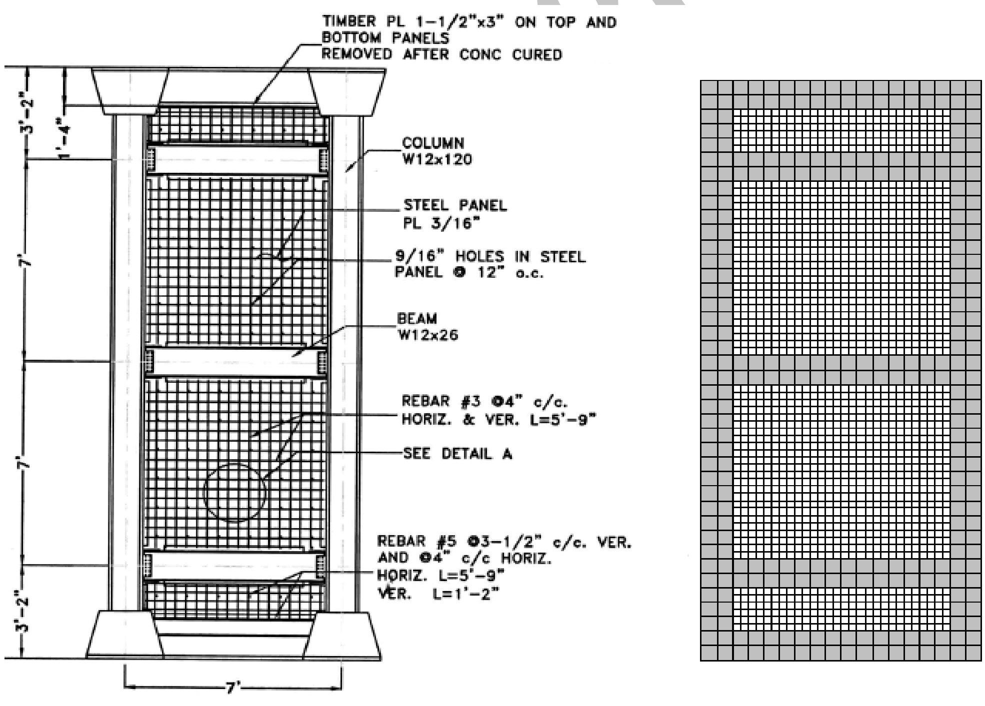

Figure 3. (a) Experimental (reprinted from Ref. No [1]) and (b) Two-dimentional meshed numerical models of Astaneh specimen

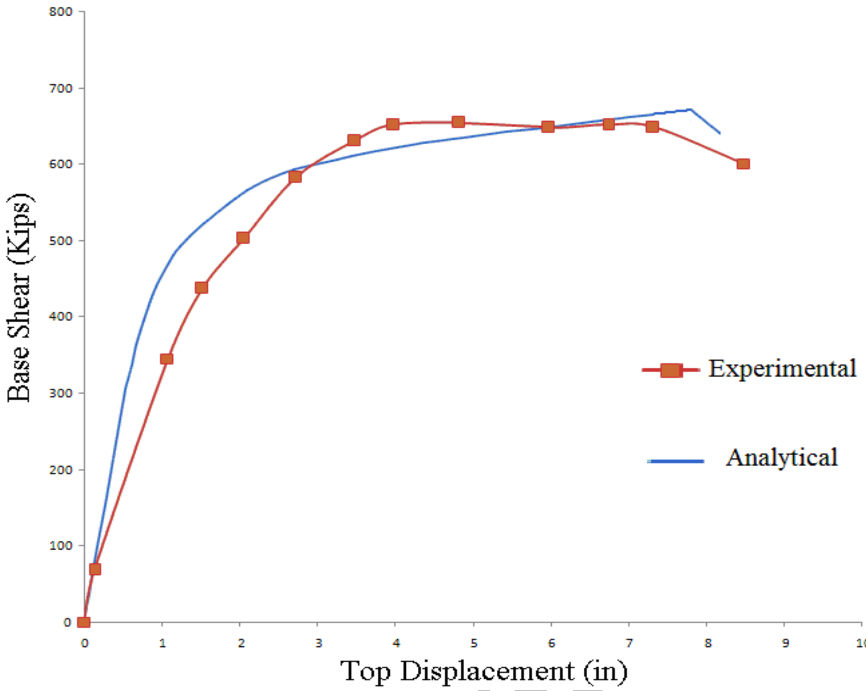

Figure 4. Comparison of experimental and analytical curves of Astaneh specimen

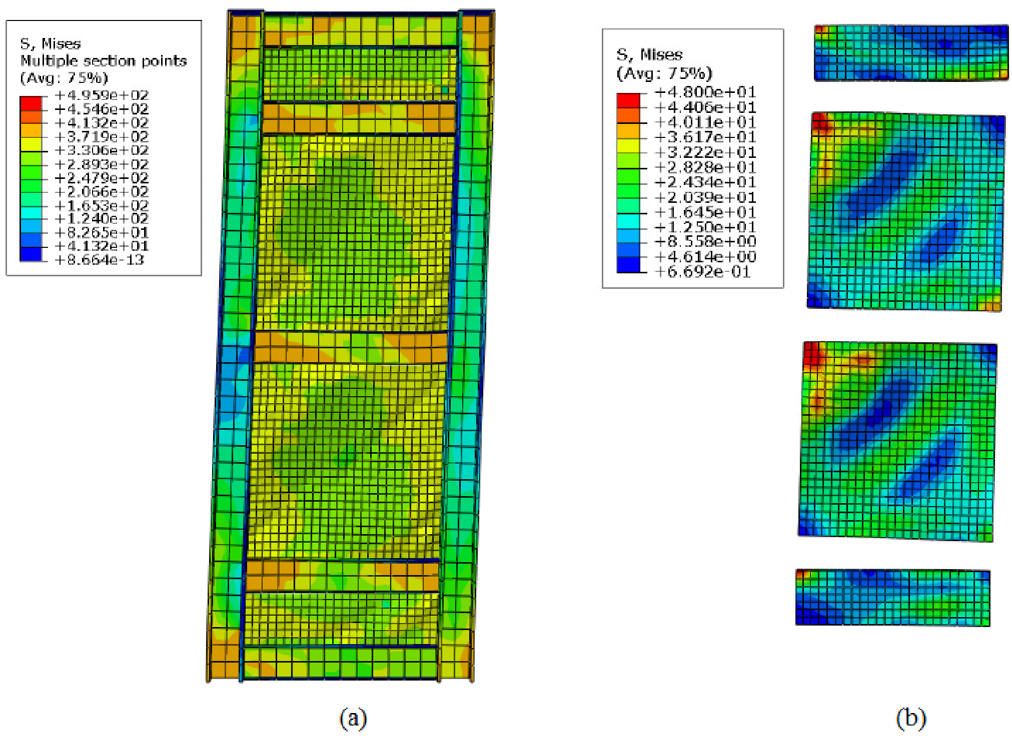

Figure 5. Von Mises stress distribution for (a) Steel and (b) Concrete parts of the model

The share in base shear carried by each component of system (steel plates, columns and concrete covers) was also plotted in Figure 6. From this figure, we can easily indicate the following remarks: first, steel plate has played the main role in providing the ductility of the specimen, since both concrete cover and columns tend to decrease load sharing in the inelastic domain at high levels of loading, while steel plate continues carrying load without reduction of strength. Second; the contribution of concrete cover in stiffness and shear resistance of the system is small. This matches well with experimental data, where the contribution (in both stiffness and ultimate strength) of lower that 20% was extracted for this cover. Third, evaluating the numerical results show that the decrease of load-sharing in columns is undoubtedly related to bending effects caused by applying displacement at top of this relatively high specimen, which has led to yielding of columns near the base that was clearly observed and notified in experimental specimen. All these evidences clearly verify the procedure of modeling used for numerical analysis of the system.

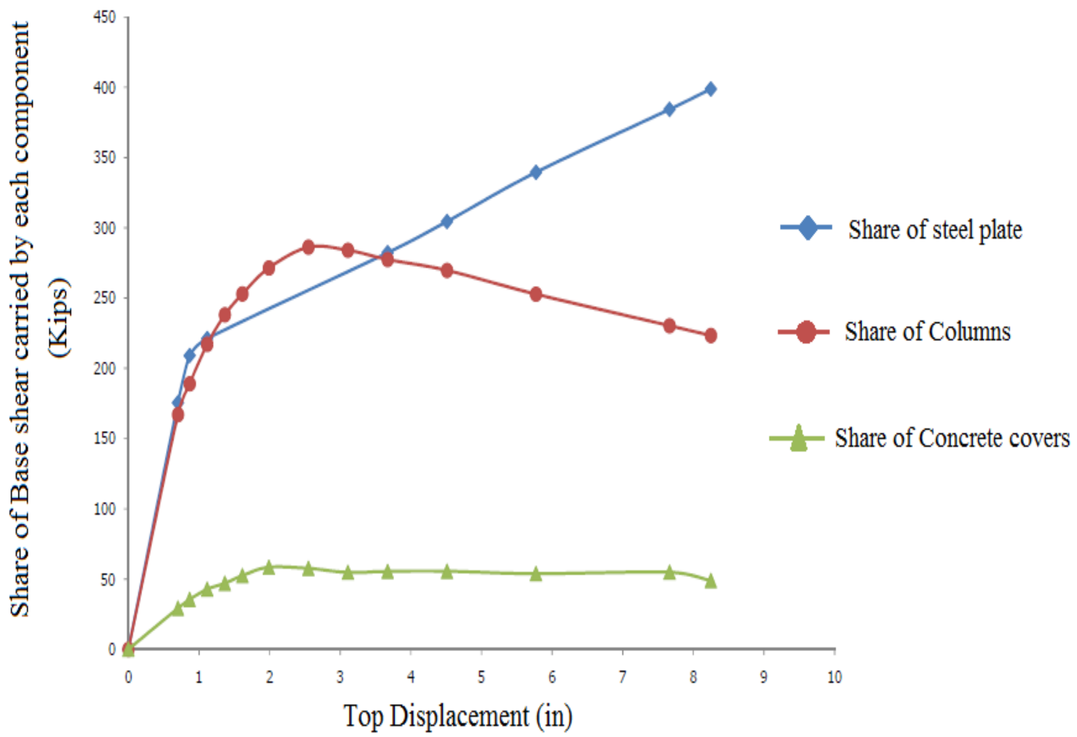

Figure 6. Proportion of base shear carried by each component of the system

3. DESIGN OF THE NUMERICAL SPECIMENS

To investigate the effects of distance between bolts, the specimens were first designed as described below, then analyzed and the results are finally compared with different cases. The arrangements of bolts considered in this research are shown in Figure 7, which are named 4-bolt and 9-bolt arrangements and introduced so that each panel is divided in to 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.

In design of the specimens, ST37 with elastic modulus and poisson ratio of respectively 2×10^5 *Mpa* and 0.3 and nominal yield and ultimate stresses of 240 and 400 *Mpa* (Figure 8) has been used for steel parts (except bolts) and also concrete with specified compressive strength of 28 Mpa and Poisson ratio of 0.2 for concrete wall (elastic modulus equals to $5000\sqrt{f'_c}\approx25000$ *Mpa*). A490 High-strength bolt with yield stress and ultimate strength of respectively 900 and1000 *Mpa* was used as connector. This type of heavy hexagon-head bolt is usually used, when diameters over 1.5 *in* up to 3 *in* is needed [14], similar to bolts studied herein. All panels have 3*m* height and 4*m* width and also have 5*mm* and 80*mm* thicknesses of steel and concrete walls, respectively. These infill walls were designed based on carrying the entire shear load by steel plate, without any prior-to-yield buckling of this plate and then concrete thickness was estimated based on providing initial stiffness or ultimate strength (whichever results in thicker cover) equal to a metal-stiffened steel wall, with no elastically-buckled subpanels. I-section *IPB600* was used for columns and also *IPE550* for beams, which were designed based on satisfying the b/t requirements for buckling, according to AISC seismic provision [15]. However, since the unilateral inelastic buckling would happen inevitably in steel plate, the frame of the model should be checked for carrying the tension field action of buckled plate, similar to a thin steel plate shear wall [16], although using same procedure for stiffened case would be conservative, since buckling of steel plate is postponed and consequently, the effect of tension field action would be weakened.

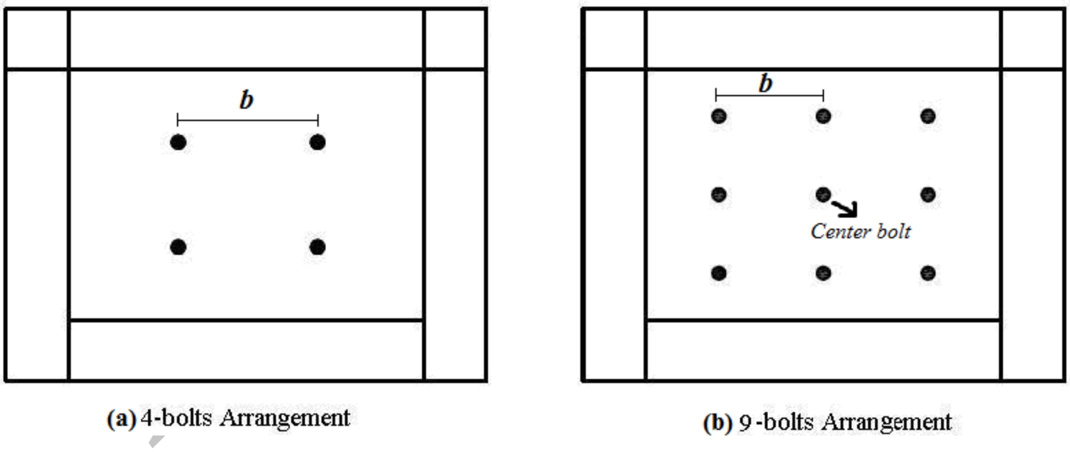

Figure 7. Different arrangements of bolts studied in this paper

In analyzing each arrangement for different distances, it is important to have stiff bolts that do not reach the yield limit state in order to achieve a ductile desirable mode of failure. For this purpose, the bolt diameter required for resisting the minimum capacity of steel and concrete walls was first calculated and then a higher commonly-used bolt diameter was used. In evaluating the capacity of concrete cover, shear resistance of reinforcing bars was neglected, since they were only expected to distribute stress uniformly through the panel and control the crack propagation. The shear capacities of steel and concrete walls were calculated from parts 17.2 of AISC seismic provision [15] and 11.9.6 of ACI-318 code [17], respectively. From these provisions, values of 37*ton* and 288*ton* were obtained for shear capacities of concrete cover and steel plate, respectively. The minimum required diameter would be 14*mm* and 10*mm* for 4-bolt and 9-bolt arrangements respectively. Therefore, the 24mm diameter high-strength bolts were selected to be used in the studied specimens.

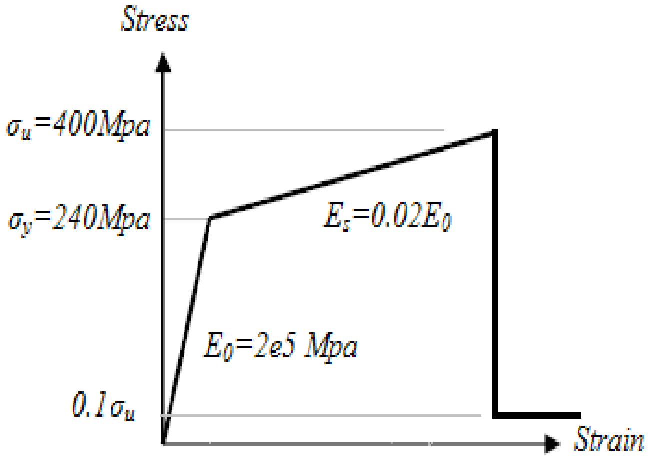

Figure 8. Stress-strain curve for steel parts of the models (frames and steel walls)

4. RESULTS AND DISCUSSIONS

4.1 Models with 4-bolt arrangement

The distance between bolts considered in this arrangement (parameter ″b″ in Figure 7-a) contains 8 values of (in *mm*) 200, 500, 667, 1333, 2000, 2667, 3000 and 3200. Results show that these specimens have had nearly same elastic behavior up to base shear of about 370*ton* with almost same initial stiffness equal to 70 *ton/mm*, but their differences appeared in the inelastic behavior. Maximum values of lateral displacement and base shear in different specimens of 4-bolt arrangement have been compared in Figure 9. It can be seen from the figure that specimen with distance of 500*mm* between bolts has obtained the most desirable behavior, since it has reached the largest inelastic deformation and most ductile behavior, while achieving same ultimate strength than others. Generally, concrete covers in specimens of this arrangement experienced two different phases of behavior: first and at initial steps of loading, they carried the shear load through diagonal compression. But second, out-of-plane lifting of concrete cover at high levels of loading has resulted in widely-extended smooth distribution of stress throughout the cover, with some stress concentrations at the location of bolts. More or less, steel plate buckles unilaterally toward its free side for all of the specimens. Furthermore, since unilateral buckling of steel plate was occurred, the bolts are under relatively large tension, which was caused by preventing steel plate from free out-of-plane displacement at the location of bolts. The maximum tensile stress of the surrounding bolts in different specimens of this arrangement is compared in Figure 10. It can be concluded from this figure that increasing the distance between bolts up to a specified limit (i.e. distance of 2667*mm*) would increase the maximum tension on the bolts, but no more increase is observed by more increase of the distance. Seeking for the reason, results of the different specimens show that unilateral buckling of steel plate first occurred at free corners rather than interior subpanels, especially when the distance between bolts are very small (such as 200 or 500*mm*) that facilitates buckling in the corners, but simultaneously stiffened the interior subpanels. The corner buckling tends to spread in to central parts, while high-strength bolts make a stiff restriction for out-of-plane displacement of the plate and therefore, prevent buckling from spreading into the interior subpanels. This prevention leads to 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 other hand, too-large interior space between bolts (like what occurred in specimens with 3000 or 3200*mm* spacing) would make buckling of plate possible just in the interior subpanels with weak (if not say impossible) out-of-plane displacement in edges, so that restriction at the locations of bolts and consequently, tension in the surrounding bolts would decrease.

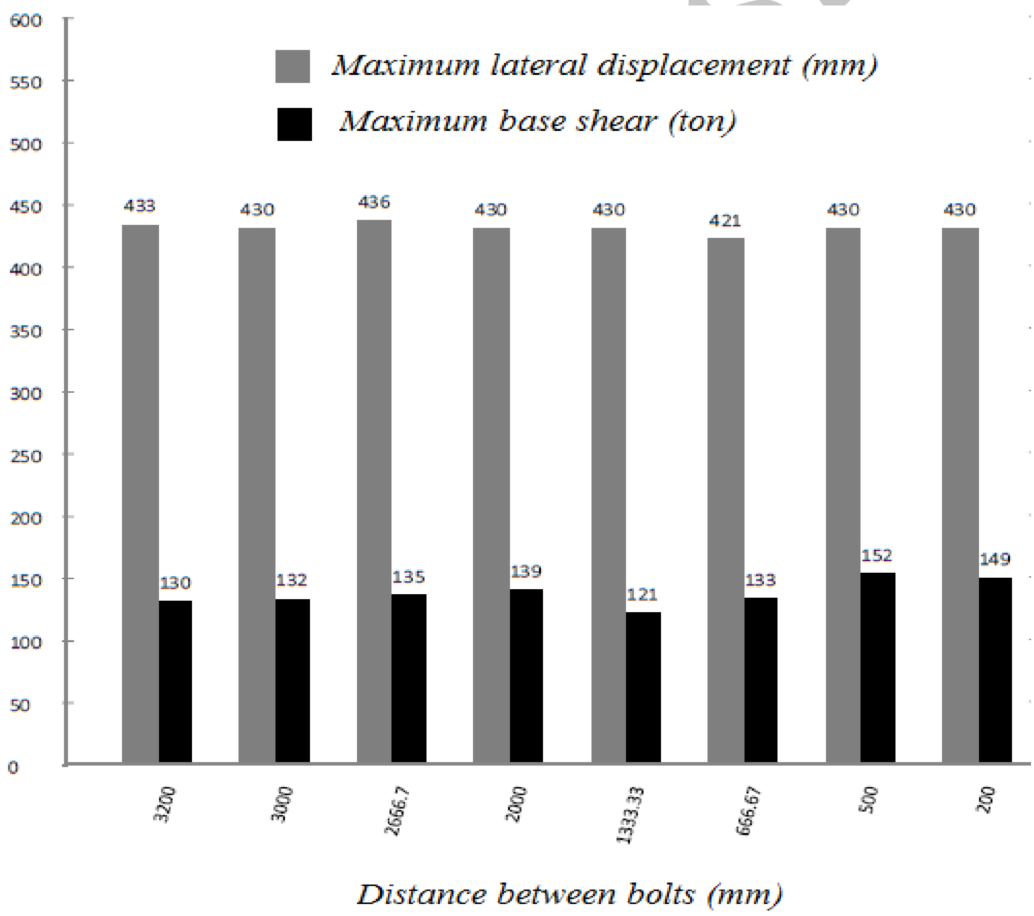

Figure 9. Comparison of maximums of lateral displacement and base shear for different distance between bolts in 4-bolt arrangements

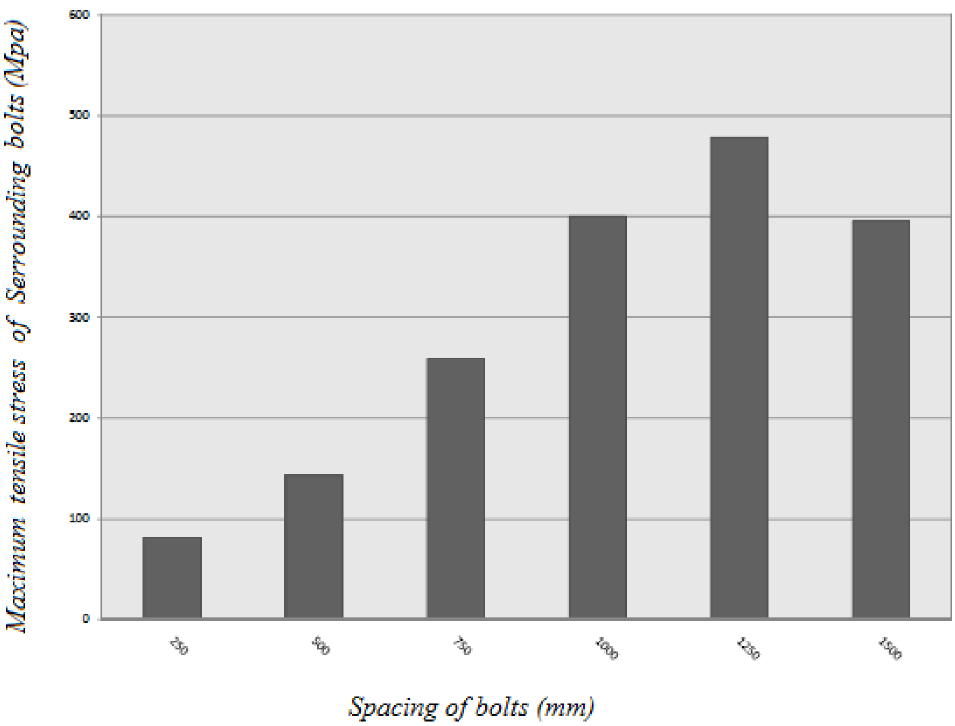

Figure 10. Comparison of maximum tensile stress of the corner bolts in different specimens of 4-bolt arrangement model

4.2 Models with 9-bolt arrangement

The distance between bolts considered in this arrangement (parameter ″b″ in Figure 7-b) contains 6 values of (in *mm*) 250, 500, 750, 1000, 1250 and 1500. In Figure 11, the pattern of steel plate buckling for specimen with distance of 1000*mm* is compared to a specimen with distance of 1333*mm* in 4-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 easily demonstrates that for a certain panel, increasing the number of bolts would increase the buckling constraint caused by bolts, since smaller free subpanels would form. Similar to 4-bolt arrangement, result indicates that we have same elastic behavior with similar initial stiffness for different distances between bolts, but different values of ultimate strength and inelastic deformation were obtained.

In Figure 12, Maximum values of lateral displacement and base shear are compared for different specimens of 9-bolt arrangement. The comparison of tensile stress in corner bolts for different specimens is also compared in Figure 13. Figure 12 demonstrates that specimen with distance of 1250*mm* between bolts behaved more ductile than others. Furthermore, Figure 12 is in agreement with the conclusion mentioned and justified in case of 4-bolt arrangement about tensile stress of surrounding bolts. One of the interesting results observed in 9-bolt arrangement is small value of tension in center bolt (introduced in Figure 6-b), compared to its surrounding bolts. In Figure 14, the ratio of tension created in center bolt, respect to average of tension in other bolts is compared for different distances of 9-bolt arrangement, which shows that this ratio has never exceeded about 50% for the studied specimens. This indicates that the interior subpanels are stiffer in preventing buckling than surrounding subpanels, which is warranted by using stiff surrounding bolts that do not allow corner buckling to spread into central subpanels. This stiff behavior is more severe, when the distance between bolts is small enough (such as specimens with distances of 250 and 500mm) to completely avoid buckling at the interior subpanels.

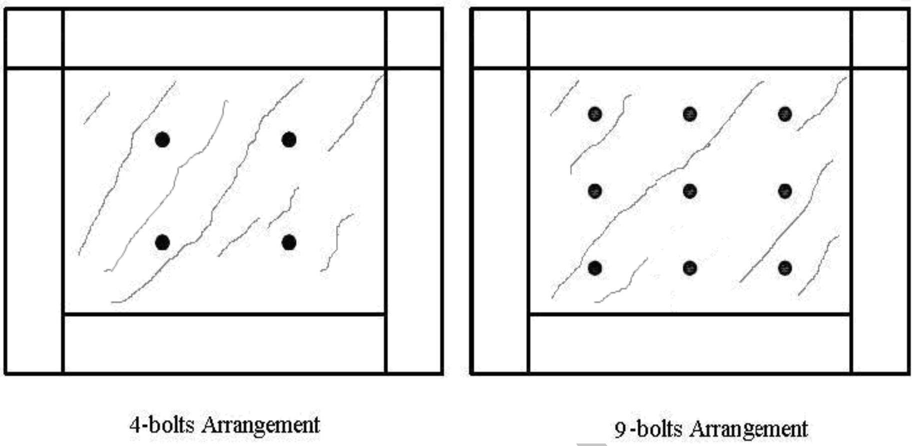

Figure 11. Patterns of unilateral buckling in different arrangements studied in this paper (Specimen with distance of 500*mm* in 4-bolt arrangement & one with distance of 1250mm in 9-bolt arrangement)

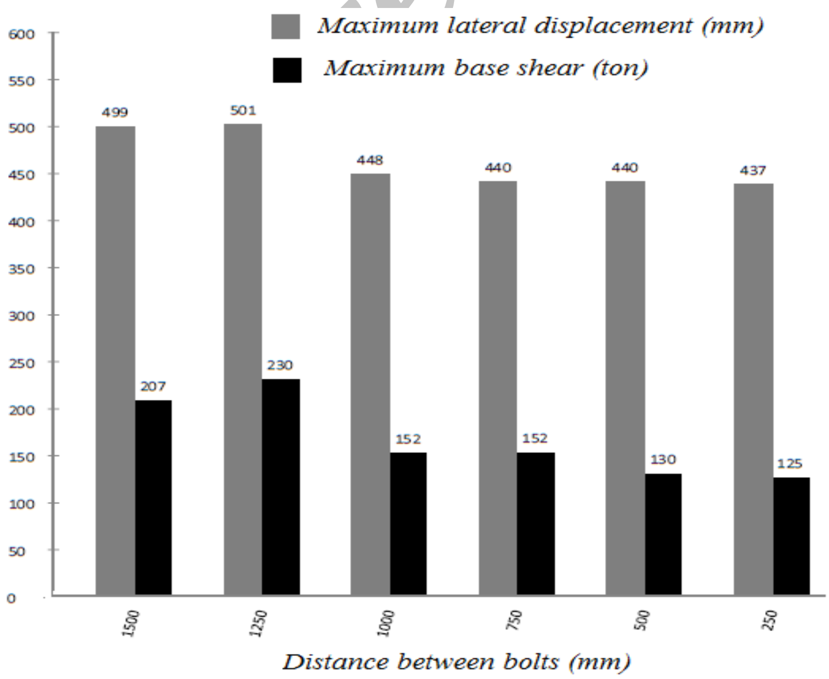

Figure 12. Comparison of maximums of lateral displacement and base shear for different distance between bolts in 9-bolt arrangements

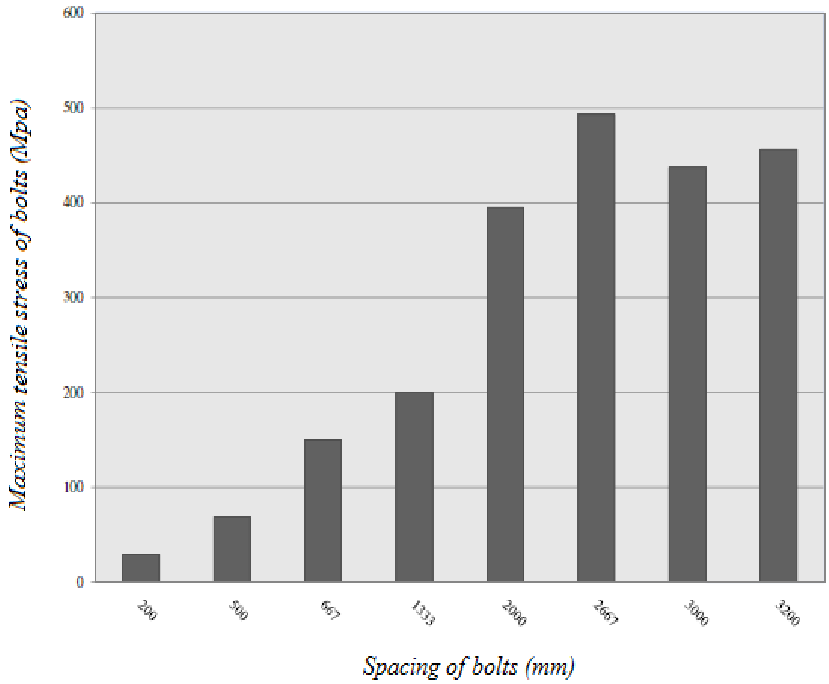

Figure 13. Comparison of maximum tensile stress of the corner bolts in different distance between 9-bolt arrangement models

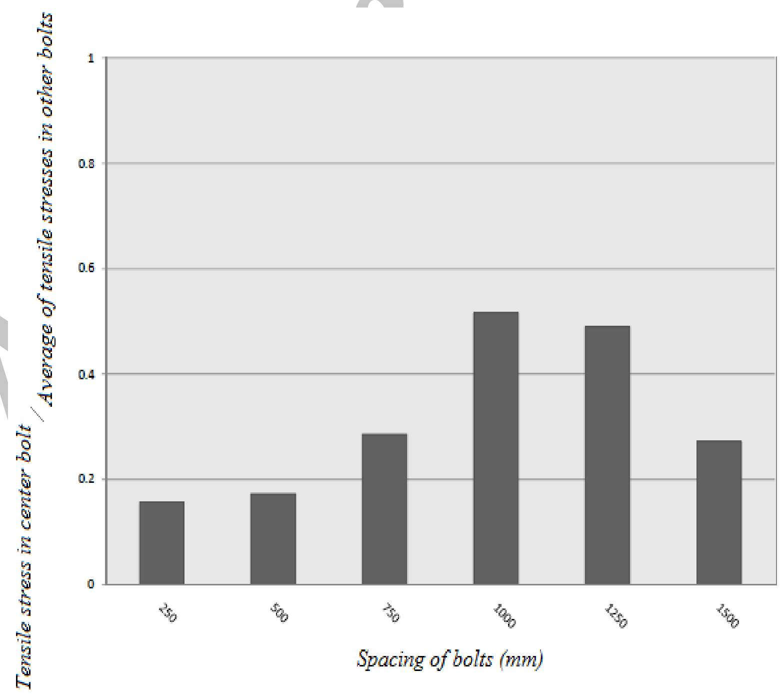

Figure 14. Comparing the ratio of tensile stress in center bolt divided to average of tensile stresses obtained in other bolts

5. CONCLUSIONS

In this paper, the numerical model of composite steel plate shear wall with two different arrangements of bolts, namely 4-bolt and 9-bolt arrangements, and different distances between bolts for each of them, has been analyzed using finite element analysis Software ABAQUS and following remarks were concluded:

1. Results of the studied arrangements show that increasing distance between bolts up to a specified limit (i.e. 500*mm* in 4-bolt and 1250*mm* in 9-bolt arrangement) would improve the behavior of system, since it makes the surrounding subpanels more stiffened, while simultaneously ensures the prevention of buckling at interior subpanel(s). But further increase of spacing would make the interior subpanels too large to facilitate buckling of steel plate in this area, so no more desirable behavior would be expected.
2. Increasing distance between bolts up to a specified value (i.e. 2667*mm* in 4-bolt and 1250*mm* in 9-bolt arrangement) would increase the maximum tension of the surrounding bolts. This is because when the distance between bolts is lower than above-mentioned values, buckling of plate is easily occur at free corner subpanels. The bolts located at corners should prevent this outer buckling from spreading to central part of the panel. The more the distance up to the mentioned limits, the more restraint is expected from corner bolts and more tension would be obtained. But beyond that, the corner subpanels are small enough to makes buckling impossible at the corners and instead, buckling would occur in central subpanel. As a result, less constraint is needed at the location of bolts, which decreases tension in bolts.
3. Regardless of the distance between bolts, the bolt located at the center of panel in 9-bolt arrangement is under a relatively small value of tension in comparison to its surrounding bolts. This shows that interior subpanels are anticipated to less unilateral buckling than corner ones, which highlights the important rule of high-strength bolts in stiffening the central subpanels and therefore, decreasing the tensile restriction needed in center of panel for preventing out-of-plane displacement. This stiffening becomes more severe, when the distance between bolts is small enough (such as 250 or 500mm) to make buckling of steel plate at interior subpanels completely impossible. In this paper, an upper limit of 0.5 is obtained for the ratio of tension in center bolt, relative to average of tension in other bolts.
4. Same initial stiffness is obtained in different specimens with different distances between bolts, either in 4-bolt arrangement or in 9-bolt one. This seems reasonable due to the fact that in elastic region, the relative movement between steel and concrete walls is small enough to help high-strength bolts to enforce different specimens behave with similar stiffness.
5. Finite element modeling is able to predict the behavior of the system accurately. This is clear from comparing load-displacement curves and locations with high stress in Astaneh experimental specimen with similar numerical model analyzed by the authors.

REFERENCES

1. Astaneh-Asl A. Seismic behavior and design of composite steel shear walls. *Steel TIPS Report, Structural Steel Educational Council*, 2002, Moraga, California.
2. Hatami F, Rahai A. Performance evaluation of composite shear wall behavior under cyclic loadings. *Journal of Constructional Steel Research*, **65**(2009)1528-37.
3. Hatami F. *Performance evaluation and optimization of composite shear wall*. Ph.D. Thesis, Amirkabir University of Technology, 2008, Tehran, Iran.
4. Hatami F, Sehri SM. Evaluation of Steel Plate Thickness on composite steel shear wall. *Journal of Structure & Steel, Iranian Society of Steel Structures,* Fall 2008; 26-36 [In Persian].
5. Ma XB, Zhang SM, Guo LH, Guan N. Simplified model of steel-concrete composite shear wall with two-side connection. *Journal of Xi'an University of Agriculture and Technology*, **41**(2009) 352–7.
6. Arabzadeh A, Soltani M, Ayazi A. Experimental investigation of composite shear walls under shear loadings. *Journal of Thin-Walled Structures,* **49**(2011) 842–54.
7. Seide P. Compressive buckling of a long simply supported plate on an elastic foundation. *Journal of the Aeronautical Sciences,* **25**(1958) 382–395.
8. Ma X, Butterworth J, Clifton C. Shear buckling of infinite plates resting on tensionless elastic foundations. *European Journal of Mechanics A/Solid.* **30**(2011) 1024–7.
9. Shahwan KW, Waas AM. A mechanical model for the buckling of unilaterally constrained rectangular plates. *International Journal of Solids and Structures,* **31**(1994) 75–87.
10. Ma X, Butterworth JW, Clifton C. Compressive buckling analysis of plates in unilateral contact. *International Journal of Solids and Structures,* **44**(2007) 2852–62.
11. Jian Cai, Yue-ling Long. Local buckling of steel plates in rectangular CFT columns with binding bars. *Journal of Constructional Steel Research,* **65**(2009) 965–72.
12. Arabzadeh A, Moharrami H, Ayazi A. Local elastic buckling coefficients of steel plates in composite steel plate shear wall. *Journal of Scientia Iranica A,* No.1, **18**(2011**)** 9-15.
13. Salmon C, Johnson J. *Steel Structures Design and Behavior, Emphasizing Load and Resistance Factor Design*, 4th Edition, Harper Collins College Publisher Inc., New York, 1996.
14. Zhao Q. *Experimental and analytical studies of cyclic behavior of steel and composite shear wall system*. Ph.D. Thesis, Civil and Environmental Engineering, University of California, Berkeley, 2006.
15. AISC. *Seismic Provisions for Structural Steel Buildings*. American Institute of Steel Construction, Chicago (IL), 2005.
16. Sabouri-Ghomi S. *Lateral Load Resisting Systems: An Introduction to Steel Shear Walls,* Anghizeh publishing Ltd, 2002, Tehran, Iran [In Persian].
17. ACI 318-08. Building code requirements for structural concrete and commentary. American Concrete Institute, Farmington Hall, MI, 2008.