ELASTIC BUCKLING BEHAVIOR OF STEEL FRAMES WITH CORRUGATED STEEL SHEAR WALLS

F. M. El –Amin[1], M. F. Abdel–Khalek[2], K. Hassan[2], E. E. Asham [3]

ABSTRACT

The search for light weight and efficient structural elements is a continuing process. Reducing the structural weight and improving the load carrying capabilities of steel frames will allow designers to add additional capabilities while reducing cost. Corrugated steel plate shear walls offer several advantages when used to resist both vertical and lateral forces in a building. This paper is devoted to the buckling behavior of steel frames with corrugated steel shear walls. The system consists of two parts, the first one is the envelope steel frame and the second is the infill corrugated steel shear wall. The corrugated steel plates are connected to the surrounding frame beam and columns. The buckling load factor of this system depends on the interaction of many factors including the corrugation configuration of steel plate, the angle of corrugation, the depth of corrugation, the thickness of corrugated plate and spacing of the fasteners used to attach the corrugated sheets to the steel frame, in addition to the restraint conditions and the external load action. In the present paper, the effects of corrugation configuration of steel shear wall, and the depth of corrugation on the buckling behavior of such frames subjected to vertical or vertical and horizontal loads were investigated. Steel frames with trapezoidal or triangular corrugated steel shear panel were modeled using the finite element software COSMOS/M 2.8 and a linear analysis was performed.

KEYWORDS: steel frames, corrugated steel shear walls, buckling factor

INTRODUCTION

Steel plate shear walls offer several advantages when used to resist lateral forces in building. These walls are lighter and more ductile than reinforced concrete shear walls. An increase in the speed of the erection and usable space are often realized. Steel savings as much as 50% have been achieved in structures employing steel-plate shear walls rather than a comparable moment-resisting frame. They are also useful in upgrading existing steel buildings since their light weight can often forgo the necessity of extensive modification of the substructure.

Several researches have been published on steel plate shear walls. Most of the researches about steel plate shear walls concentrated on the flat plate shear wall with or without stiffeners. In this paper a new-innovative system, which uses corrugated plate shear walls is presented. The use of corrugated steel plate shear walls has both an aesthetic and structural function. It helps to increase the out of plane stiffness and buckling behavior without the use of vertical stiffeners, as well as it overcomes the

(1) Prof. of structural Eng., (2) Assistant Prof. of structural Eng., (3) Demonstrator, Civil Eng. Dept., Faculty of Eng., Assiut University, *Assiut, Egypt*

problem of flatness in flat plate.

The early study on the corrugated plate was reported by Easley [1]. He tested small-scale carefully constructed corrugated diaphragms to determine the most accurate formula to calculate its shear buckling strength. He further compared the test results with calculated results using three sets of formulas and found that the Easley-McFarland [2], formulas agrees better with the experimental results than that of Hlavacek [3], formulas, the results of which had a 20% difference from the test results.

Timler and Kulak [4] tested a single-story steel flat plate shear wall to verify the analytical technique established by Thorburn et al. [5]. The specimen was loaded statically, with three complete cycles of loading to a service load deflection limit of $h_s/400$, or 6.25 mm. During these cycles, it was assumed that the test specimen behave elastically. Subsequently, the shear wall was loaded in one direction to its ultimate capacity. Later, Tromposch and Kulak [6] tested steel flat plate shear wall similar to that previously tested by Timler and Kulak [4]. However, important modifications were introduced including the use of bolted shear connections to the frame, a thinner infill panel, and very stiff beam members. The stiff beams were intended to provide anchorage to the infill panel similar to that expected in a multistory condition with panels above and below.

Recently, an experimental program conducted by Elgaaly and Caccese [7] investigated the behavior of ten one-quarter-scale steel flat plate shear wall models subjected to cyclic loading. Caccese et al. [8] and Elgaaly et al. [9] tested one quarter-scale steel flat plat shear walls, three stories high and one bay wide, under cyclic lateral load. Two series of 24-load cycles were conducted using a single in plane horizontal load at the top of the shear wall. The test specimens were modeled by the finite element method using shell elements to represent the infill plate in one case and by a perpendicular grid of truss elements oriented in the directions of the principle tensile and compressive stresses in other case.

A general method of dynamic analysis of thin-panel steel flat plate shear walls was developed by Sabouri-Ghomi and Roberts [10]. The method uses a time-stepping finite-difference technique to solve the governing differential equation of motion. The structure is idealized as a vertical cantilever beam with masses lumped at each floor level. Xue and Lu [11] carried out an analytical study on four thin steel flat shear panels. In each case, a 12-story, and three-bay frame with moment-resisting beam-to-column connections in the exterior bays and with steel infill flat plates in the interior bay was used. Later, Xue and Lu [12] studied the effect of the width-to-thickness ratio of the panel and the panel aspect ratio on the load versus deflection behavior of a single-story, single-bay steel plate shear wall with pinned beam-to-column connections and with the infill panel connected only to the beams.

More recently, a large-scale four-story steel plate shear wall was tested by Driver et al. [13] in order to evaluate the performance of this type of structure under severe cyclic loading. The test specimen had unstiffened panels and moment-resisting beam-to-column frame connections. Both vertical and lateral loads were applied, and the maximum deflection achieved in the lowest story was nine times the yield deflection. The test specimen showed great ductility, considerable energy dissipation, and robust resistance to degradation.

To the authors' knowledge, this is the first investigation concern with the buckling behavior of steel frames with corrugated steel plate shear wall. Therefore, a great need to study the effect of these shear panels with different corrugation configurations on the elastic buckling behavior of one bay steel frames.

PROBLEM DEFINTION

Figure 1-a, shows the geometrical dimensions of the rectangular hinged base steel frame with corrugated steel plate considered throughout the present study. The frame was assumed 6.0 m. wide and 6.0 high with constant moment of inertia. The S.I.B. No. 20 was assumed for the frame components. The detailed dimensions for this cross section are shown in figure 1-b. For the steel shear wall, two different corrugation profiles were selected, namely trapezoidal and triangular corrugated plates. The corrugation parameters are as shown in figure 1-c. The depth of corrugation was assumed as 25 mm., 50 mm., and 80 mm., for both the trapezoidal and triangular shear walls while the thickness of these plates was assumed constant and equal 2.0 mm. The dimensions of the triangular and trapezoidal corrugation profiles are summarized in tables 1 and 2, respectively. The corrugated panel is fastened to the frame members allover its perimeter and the bottom edge of corrugated panel is fully blocked by RC strap. It is assumed that the bottom edge is fastened to the RC strap footing where all nodes in this edge are restrained in the three directions. This model was analyzed under the effect of two cases of loading, which are referred to as "FCV" and "FCVH". In the first case "FCV", the model is subjected to two vertical loads "P" at its top corners. In the second case of loading "FCVH", the model is subjected to two vertical loads at its top corners in addition to single horizontal force "0.1P" acting at the left hand top corner of the steel frame.

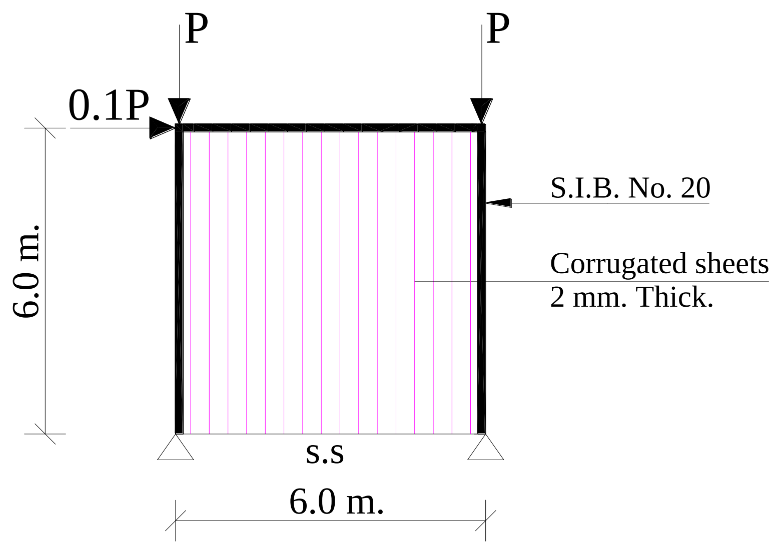


Fig.1-a: Dimensions and geometry of the steel frame

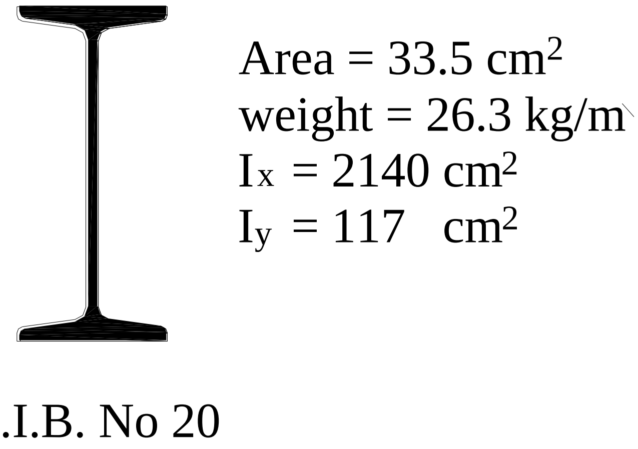


Fig.1-b: Cross section of steel frame elements.

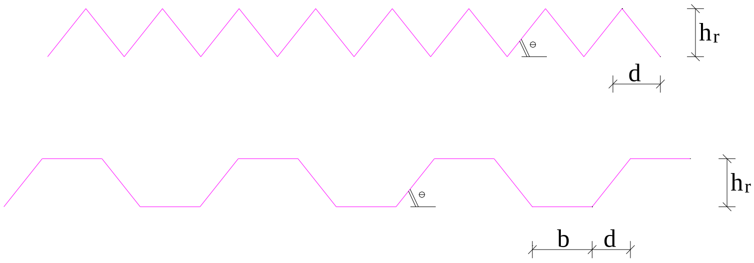


Fig.1-c: Dimensions of corrugation profiles.

Table 1: Dimensions of triangular corrugated in-filled panels

Panel type	h_r (mm)	D (mm)	θ Degrees	Thickness "t" (mm)
Profile 1	25.0	25.0	45	2.0
Profile 2	50.0	50.0	45	2.0
Profile 3	80.0	80.0	45	2.0

Table 2: Dimensions of trapezoidal corrugated in-filled panels

Panel type	h_r (mm)	b (mm)	d (mm)	θ degrees	Thickness "t" (mm)
Profile 1	25.0	25.0	25.0	45	2.0
Profile 2	50.0	50.0	50.0	45	2.0
Profile 3	80.0	80.0	80.0	45	2.0

The effect of corrugated plate height to the frame span ratio (h_c/L) on the model critical buckling load was also considered. For each case of loading four models, with (h_c/L) ratios of 1.0, 0.5, 0.333, and 0.25 were analyzed. In these models the corrugated steel plates were provided with a horizontal steel strap 20.0 mm. thick and 100.0 mm. wide as shown in figures 2-a, and 2-b, and consequently, this steel strap attached to the steel frame columns at the desired height h_c according to the above mentioned (h_c/L) ratios. These models are identified in the text as FCVS1, FCVHS1, FCVS2, FCVHS2, FCVS3, and FCVHS3 as show in figure 3. Thus, a total of 48 steel frames with trapezoidal and triangular corrugated shear plates were considered in this investigation.

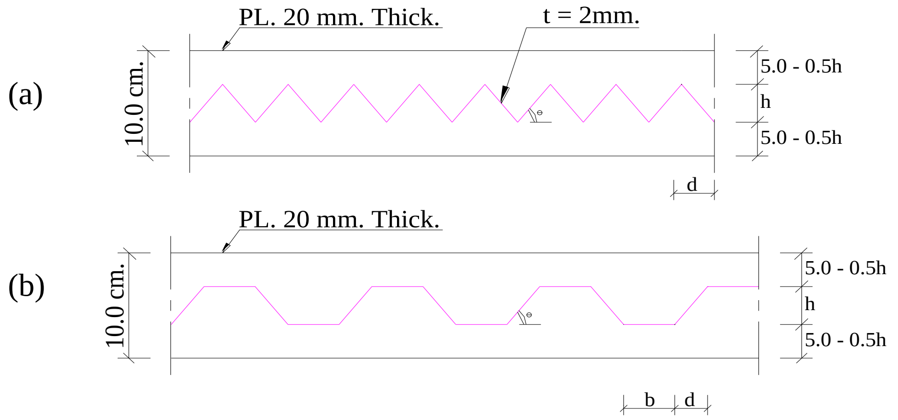


Fig.2: Dimensions of triangular and trapezoidal corrugated plates and horizontal stiffener.

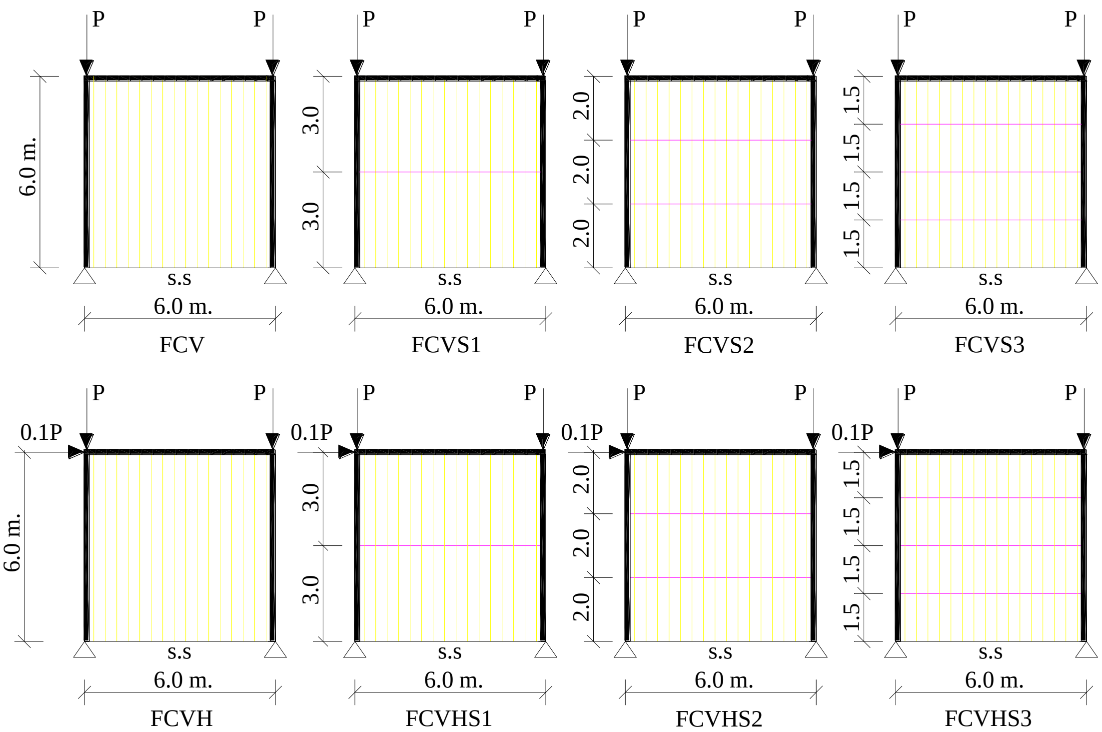


Fig. 3: Geometry and cases of loading for the models under study.

FINITE ELEMENT ANALYSIS

The finite element method has been used in the present study to determine the critical elastic buckling load for the steel frames with corrugated shear wall. In the finite-element linear elastic buckling analysis, the method is based on solving an eigenvalue problem that describes the behavior of the model at buckling. The lowest eigenvalue corresponds to the critical buckling load, and the eigenvector defines the model buckled shape. The analytical tool used in this study is a commercially 3-D finite element code Cosmos/m 2.8 [14]. The framing members, which are located in the *z=0*, *x-y* plane, are modeled using two-dimensional elastic beam elements (BEAM2D). On the other hand, the corrugated plate model is discretized using A 4-node "QUAD4" plate/shell finite element with 6 degrees of freedom at each node that are available in the computer program. For the trapezoidal corrugated steel plates two elements were used across the width of each horizontal and inclined fold of the corrugations. Similarly, two elements were used across the width of each inclined fold of the triangular corrugated steel plate. The vertical edges of the corrugated plates were divided to 60 elements. This mesh was chosen based on an extensive preliminary convergence tests. The elastic modulus E of 2100 t/cm^2, and Poisson's ratio ν = 0.3 are used for both the steel frame elements and the corrugated shear wall materials. All geometry, boundary conditions, and loading were modeled in the Cartesian coordinate system. A typical finite element model generated and the first buckling mode shape are shown in figures (4) and (5), respectively.

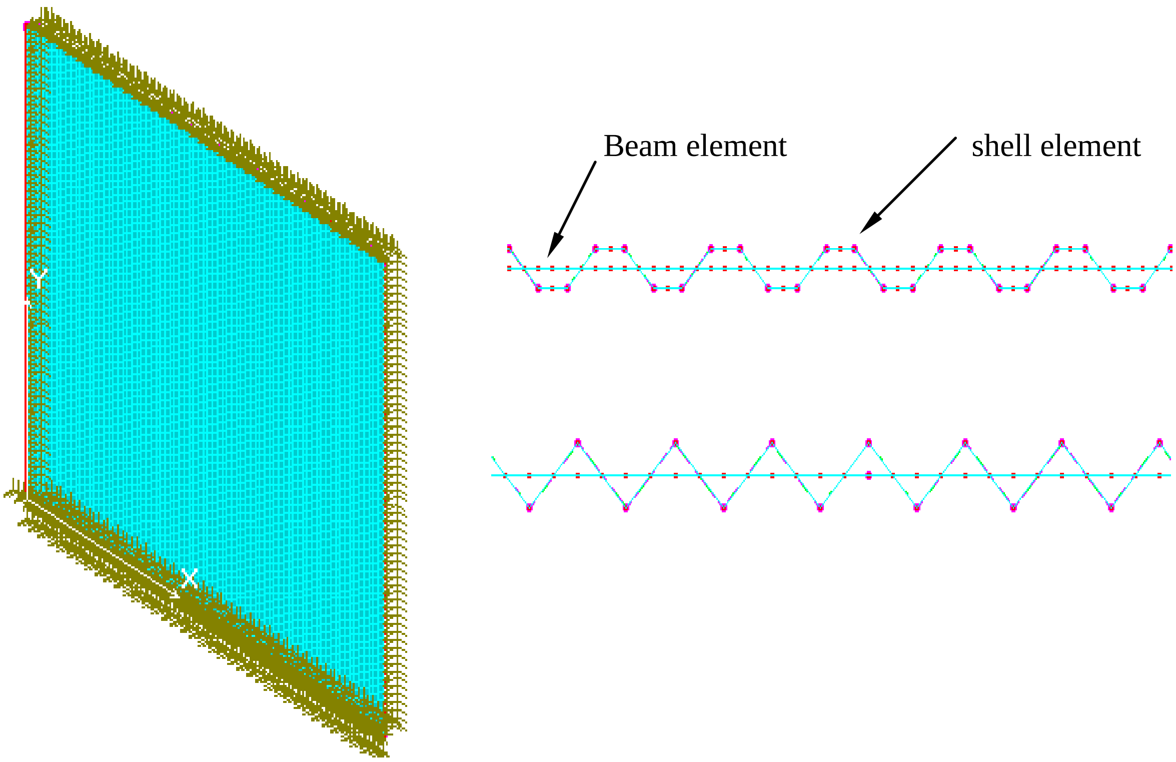
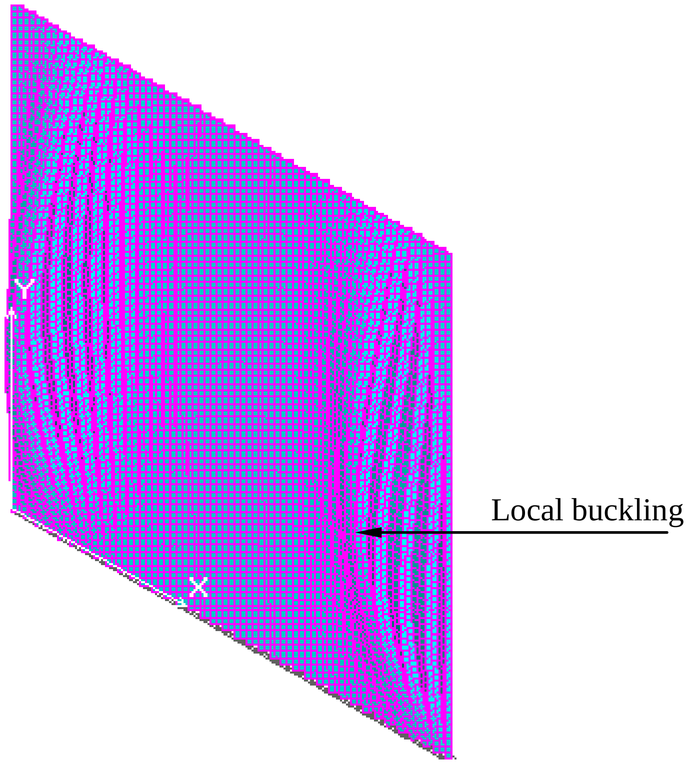
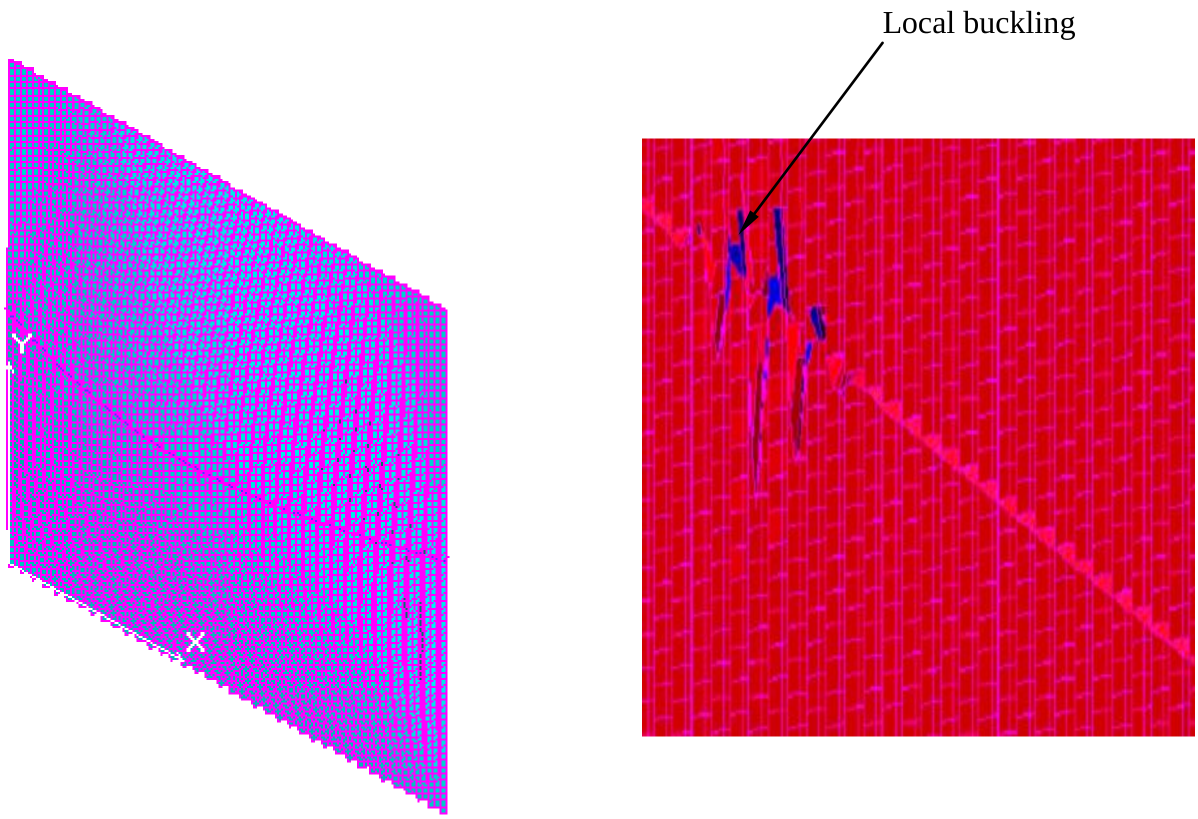


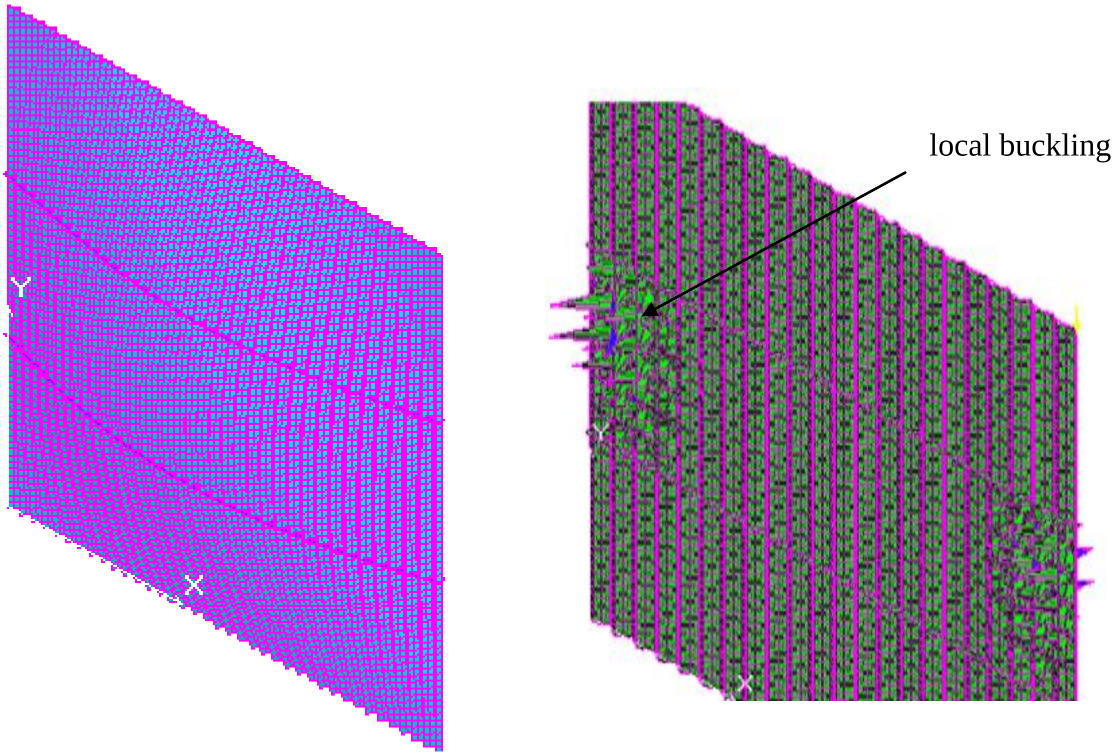
Figure 4: Finite element model and boundary conditions as given by COSMOS/M.2.8



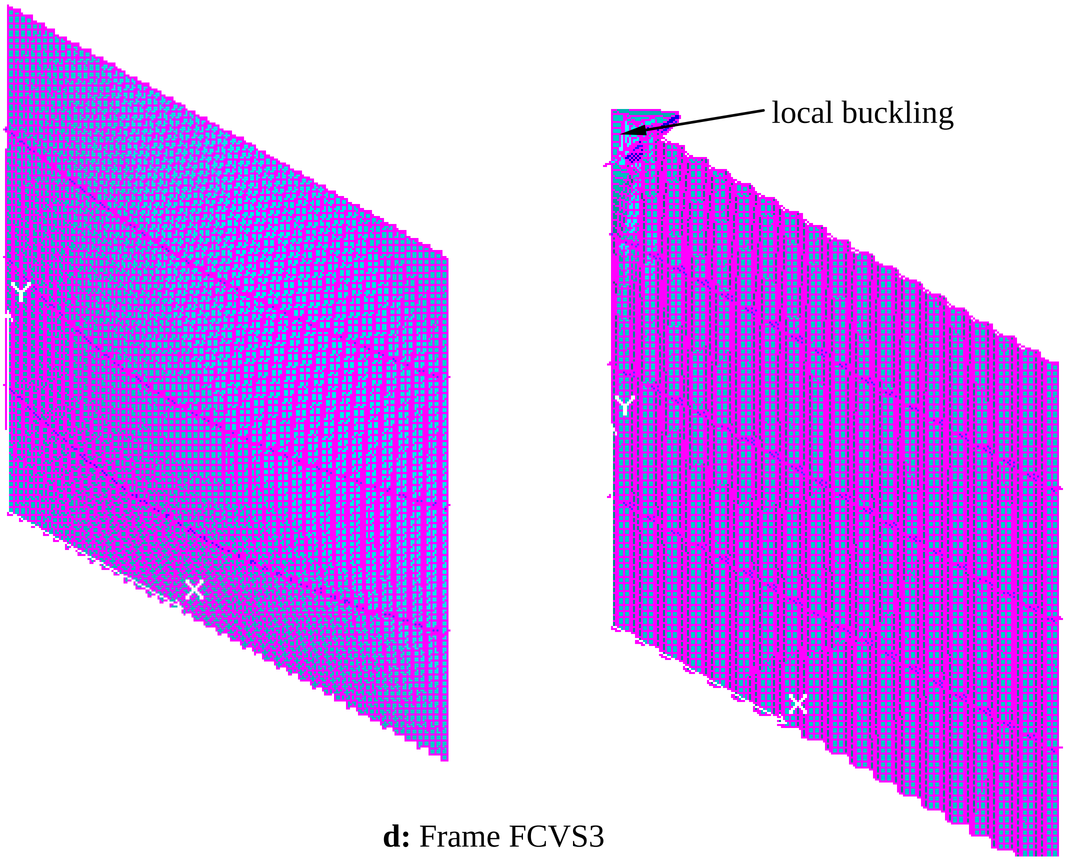
a: Frame FCV



b: Frame FCVS1



c: Frame FCVS2



d: Frame FCVS3

Figure 5: Buckled shapes of the steel frame with corrugated plates.

PARAMETRIC STUDY

As mentioned earlier, the Cosmos\M software has been used in the present study. This investigation was commenced with the analysis of open steel frame without shear plates in order to establish the adequacy of the present finite element analysis. Therefore, bifurcation-buckling analysis was carried out for a two hinged steel frame subjected to two vertical loads "P" at its top corners. The difference between the exact solution [15] and the critical buckling load from the finite element analysis was only -1.67 %.

In order to investigate the effect of the steel shear plates on the linear elastic buckling of the steel frame, the cross section of the steel frame is kept constant throughout this study. The corrugation configuration, depth of corrugation, and the presence and location of horizontal steel stiffeners are among the parameters considered in this study. The trapezoidal and triangular corrugation profiles with different depth and constant plate thickness were considered. In all cases the angle of inclination on inclined fold "θ" was assumed 45^o.

RESULTS AND DISCUSSION

A total of 48 finite element models were created and tested using Cosmos\M software package. All models were assigned with different geometry for the corrugated shear wall to explore the influences of each variable on the critical buckling load factor "B.L.F" for the steel frame with corrugated steel wall under the effect of the previously mentioned two cases of loading. The results obtained are summarized in table 3. For the benefit of comparison, the ratio between the critical buckling load factors for the steel frames with corrugated shear walls "K_{Fc}", and the corresponding values for the steel frames without shear wall "K_F" were also calculated as shown in table 3. This ratio is identified in the text as the normalized load factor (N.L.F= K_{Fc}/K_F). The relationships between the panel corrugation depth and the normalized load factors are illustrated in figures 6 and 7 for the steel frames with triangular and trapezoidal corrugated shear walls, respectively. At the same time, the results obtained when the same models were subjected to a single horizontal load "0.1P" in addition to two vertical point loads "P" are also shown in figure 6 and 7. In this case, it is clear that the values of the normalized load factor are lower than the corresponding values for the frames subjected to two vertical loads. On the other hand, it is interesting to note the effect of the horizontal stiffeners on the normalized load factor for these models. When the depth of corrugated shear panel were assumed 25 mm., and 50mm., it can be seen that the normalized load factor is the highest for model "FCVS3" followed by FCVS2, FCVS1, and FCV. The normalized load factor increases with a decrease in the h_c/L ratio. However within the parameters considered here, as the depth of the corrugated shear wall is increased to be 80 mm, it is clear that the values of normalized load factor are constant for model FCVS1, FCVS2 and FCVS3, this mean that the effect of horizontal stiffeners diminishes, this refer to local buckling phenomenon of corrugated plate between the stiffeners as shown in finger 5. The present results also showed that the normalized load factor for models FCV, and FCVH, was lower than unity when the depth of corrugated shear wall was assumed

25 mm this refer to the local buckling of the corrugated plate only without any buckling in the steel frame. Evidently, as the corrugated shear wall height to frame span ratio "h_c/L" equals unity the behavior of the model approaches that of a separate panel.

In the sake of comparison, the results obtained for the steel frames with triangular shear wall along with the corresponding values for the frames with trapezoidal one, are given in table 3. It appears that the normalized load factor for steel frames with trapezoidal corrugated shear walls is higher than the corresponding values for steel frames with triangular corrugated panels. For example, as shown in table 3 and Fig, 6, the FCV, and FCVH models with trapezoidal shear walls could sustain 61% and 96.7% higher load compared to the same models with triangular corrugated shear walls when the depth of corrugation was assumed 50 mm, and 80 mm, respectively. At the same time, the percentage of differences obtained for the models FCVS1, FCVS2 and FCVH3 in the range of 25.97 % to 44.96 % when the depth of corrugation was assumed 50 mm. On the other hand, these differences decrease when the depth of corrugation becomes 80 mm.

The weight of steel parts is a pronounced factor in design and constructing a steel structure, this refers to the cost of the project. For this reason and for the benefit of comparison, the authors utilized the terms normalize weight which is defined as the weight of corrugated shear wall to weight of steel frame ratio "W_c/W_f" and the normalized load factor "K_{FC}/K_C" and the results obtained from this study are summarized in table 4 and are illustrated in figures 8 and 9. These figures show the relationships between the normalized load factor and the corresponding normalized weight factor for different models used in this study. It could be seen from these graphs (Figs. 8 and 9) that for using corrugated plate with depth of corrugation 25mm, and 50mm, the normalized load factor "K_{FC}/K_C" of the model increases as its normalize weight "W_c/W_f" increases. On the contrary for using corrugated plate with depth of corrugation 80mm, there is no change on the value of normalized load factor when increased the normalized weight for trapezoidal configuration. However, from the economical point of view, the buckling capacity of the model "FCVS1" with triangular or trapezoidal corrugated shear wall and corrugation depth of 80 mm, has been increased between 6.53 and 6.80 times the buckling capacity of the steel frame without shear wall

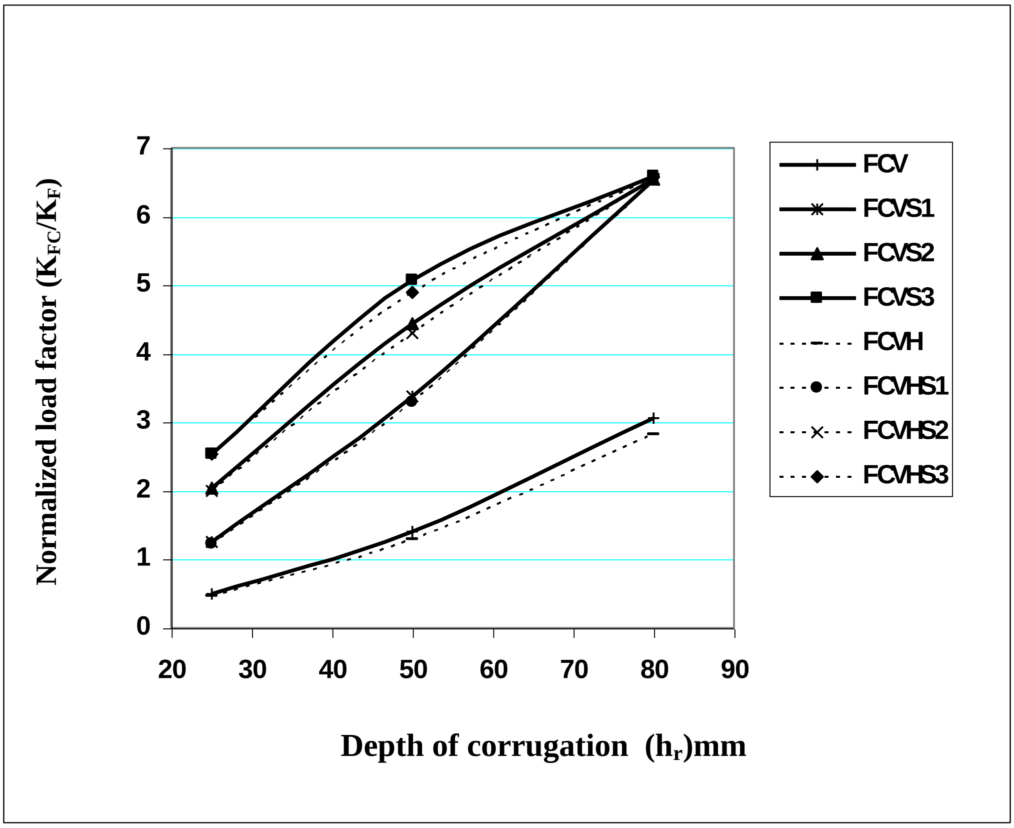


Fig. 6: Relationship between normalized load factor and depth of triangular corrugated shear wall.

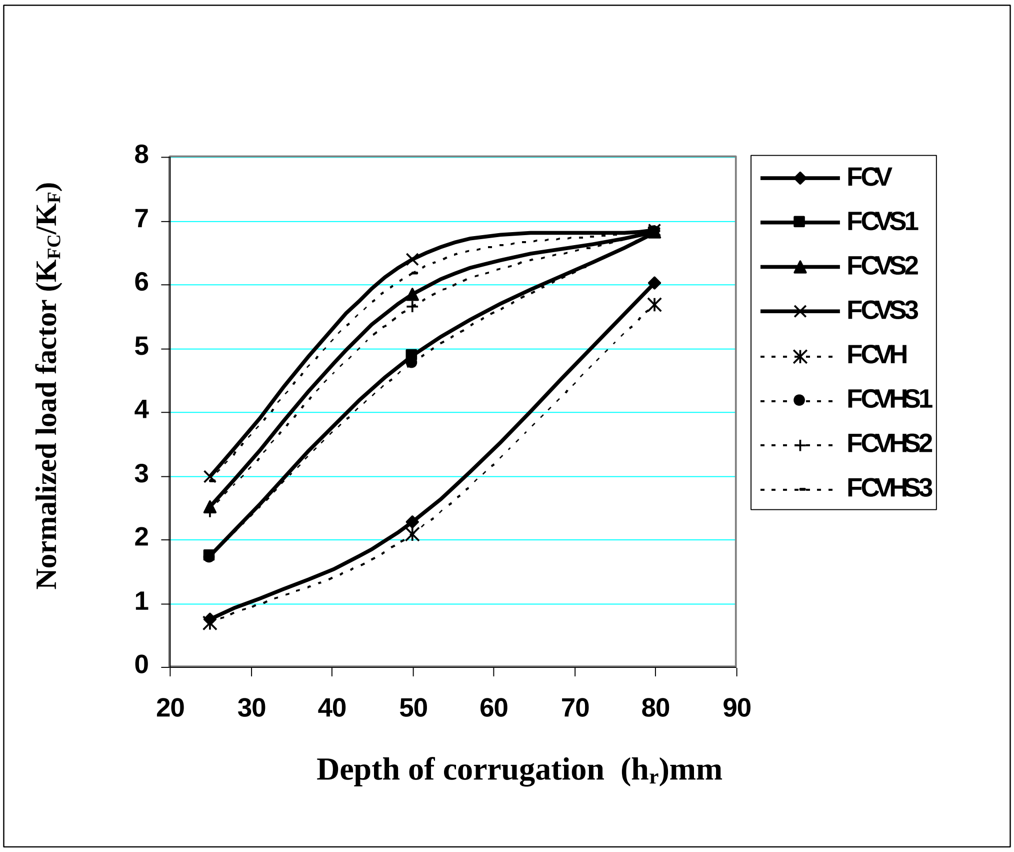


Fig. 7: Relationship between normalized load factor and depth of trapezoidal corrugated shear wall.

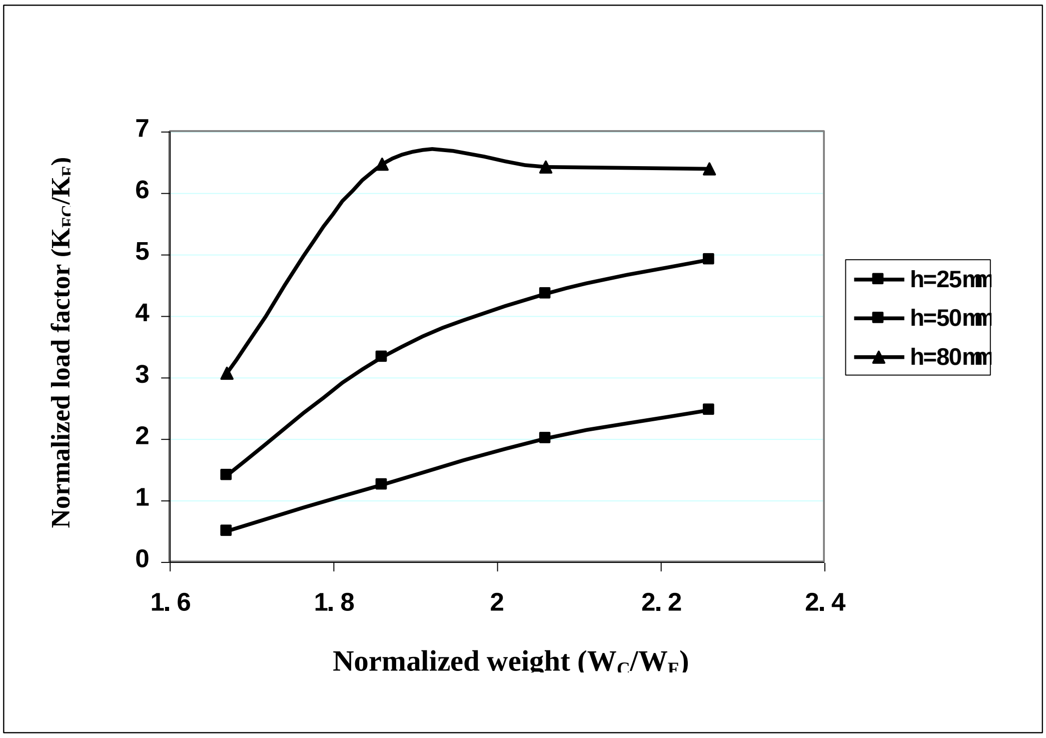


Fig. 8: Relationship between normalized weight and normalized load Factor K_{FC}/K_F for steel frame with triangular corrugated shear wall.

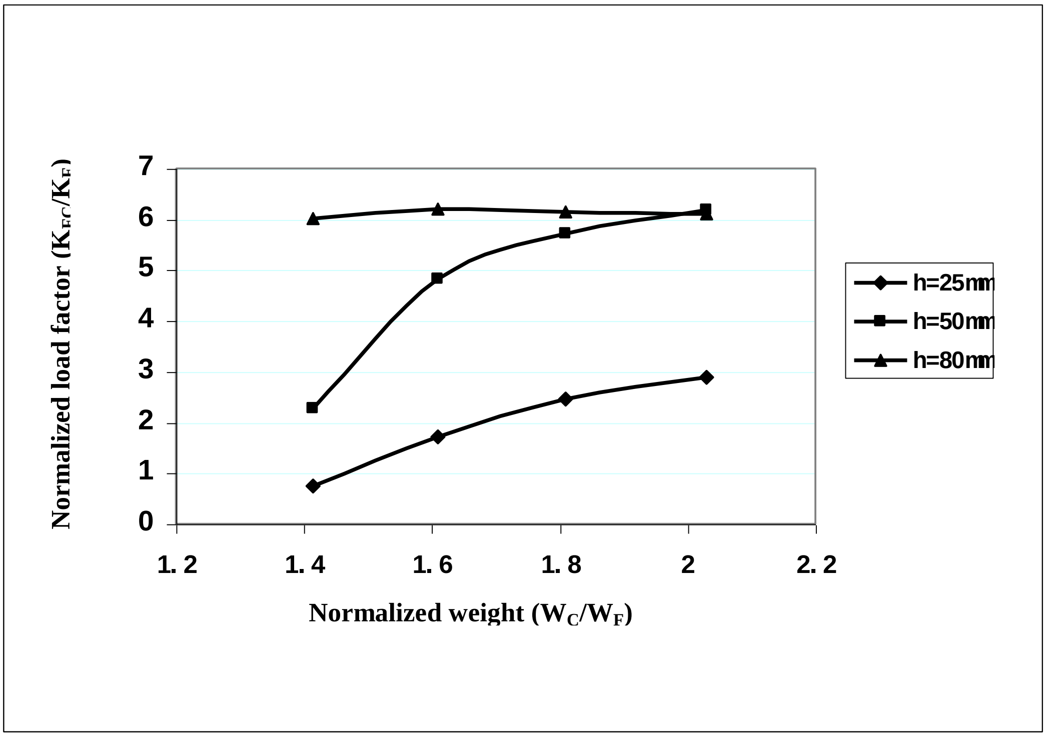


Fig. 9: Relationship between normalized weight and normalized load Factor K_{FC}/K_F for steel frame with trapezoidal corrugated shear wall.

CONCLUSIONS

In this investigation, the elastic buckling behavior of steel frames with corrugated steel shear wall has been studied. A finite element software Cosmos/M. was used for this purpose. Parametric studies on variables such as corrugation configuration of steel shear wall, strengthening corrugated shear wall by horizontal stiffeners, and the depth of corrugation on the buckling behavior of such frames subjected to vertical or vertical and horizontal loads were investigated. Based on results obtained, the following conclusions are drawn:

1- The steel frame with vertical trapezoidal corrugated steel shear wall could sustain between 2.08 and 5.67 times the critical buckling load of identical steel frame without shear wall, when the depth of corrugation was assumed 50 mm. and 80mm, respectively. When the trapezoidal corrugated shear walls were replaced by triangular one, lower buckling load is achieved. The corresponding increments obtained were 1.293 and 2.82. for that the authors recommended not using corrugated plate with depth of corrugation 25mm for local buckling phenomenon on the plate, but recommended by using corrugated plate with depth of corrugation 80mm.

2- Higher buckling capacity is achieved when the vertical trapezoidal or triangular corrugated steel shear wall was strengthened by horizontal stiffeners. However, the ideal ratio of h_c/L, which gave the maximum value of buckling load, is 0.5, this mean that there is one stiffener in the middle of the height.

3- The steel frame with trapezoidal corrugated shear wall contributed higher buckling load capacity than the corresponding values for the model with triangular shear wall. Besides that, the saving in the models weights are within the range of 10.18 to 15.27% in comparison with the same models with triangular shear walls.

REFERENCES

1. Easley, J.T., “Buckling formulas for corrugated metal shear diaphragms”, Journal of the Structural Division, ASCE, St.7, pp. 1403-1417, July, (1975).
2. Easley, J., and McFarland, D., “Buckling of light-Gage corrugated shear Diaphragms”, Journal of the Structural Division, ASCE, Vol.95 No. ST 7, pp.1497-1516, July, (1969).
3. Hlavacek, V., “Shear Instability of Orthotropic Panels” Acta Technica CSAV, Prague Czechoslovakia, No. 1, (1968).
4. Timler, P. A., And Kulak, G. L. “Experimental study of steel plate shear walls.” Struct. Eng. Rep. No. 114, Dept. of civil Eng. University of Alberta, Edmonton, alta., Canada, (1983).
5. Thorburn, L.J., Kulak, G. L., and Montgomery, C.J. “Analysis of steel plate shear walls.” Struct. Engrg. Rep. No. 107, Dept. of Civil Engrg., University of Alberta, Edmonton, Alta., Canada, (1983).

6. Tromposch, E. W., and Kulak, G. L. “Cyclic and static behavior of thin panel steel plate shear walls” Struct. Eng. Rep. No. 14. Dept. of Civil Eng. University of Alberta, Edmonton, Atla., Canada, (1987).
7. Elgaaly, M., and Caccese, V. “Steel plate shear walls ” proc., AISC Nat. Steel Constr Conf., American institute of steel Construction (AISC), Inc., Chicago, III. 4-1-4-28, (1990).
8. Caccese, V., Elgaaly, M., and chen, R. “Experimental study of thin steel-plate shears walls under cyclic load” J Structure Eng. ASCE, Vol. 119, No. 2, pp. 573-587, February, (1993).
9. Elgaaly, M., Caccese, V., and Du, C. “Post buckling behavior of steel-plate shear walls under cyclic loads. ” J. Structure Eng. ASCE, Vol. 119, No. 2, pp. 588-609 , February, (1993).
10. Sabouri-Ghomi, S., and Roberts, T. M. “Nonlinear dynamic analysis of thin steel plate shear walls” Comp. And Struct., 39 (1/2), pp. 121-127, (1991)
11. Xue, M., and Lu, L., -W. “Interaction of infilled steel shear walls panels with surrounding frame members” proc. Struct. Stability R.Council Annu. Tech. Session, Bethlehem, Pa., pp.339-354, (1994a).
12. Xue, M., and Lu, L.,-W. “Monotonic and cyclic behavior infilled steel shear panels. ”Proc., 17th Czech and Slovak Int. Conf. On steel Struct. And Bridges, Bratislava, Slovakia, (1994b).
13. Driver, R. G., Kulak, G. L., Elwi, A. E., and Kennedy, D. J. L. “Cyclic test of four-story steel plate shear wall” J. Strct. Eng., ASCE, Vol. 124, No. 2, pp. 112-120 , February, (1998).
14. Cosmos/M 2.8 “A computer program for nonlinear static and dynamic analysis”, Structural Research and Analysis Corporation, California, USA, (2002).
15- Timoshenko, S. P., and Gere, J. M. “Theory of Elastic Stability”, 2nd Ed., McGraw-Hill Book Co., Inc., New York, N.Y. (1961).

Notation

The following symbols are used in this paper:

b = width of horizontal field of corrugation.
d = width of inclined field of corrugation.
h_r = depth of corrugation
h_c = spacing between horizontal stiffeners.
h_s = steel frame height
θ = angle of corrugation.
N.L.F ≡ normalized load factor
K_{FC} ≡ buckling load factor of the steel frame with corrugated shear wall
K_F ≡ buckling load factor of the steel frame
N.W ≡ normalized weight
Wc ≡ weight of the corrugated steel panel.
W_F ≡ weight of the steel frame