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Nonlinear finite element modeling of steel-sheathed cold-formed steel shear walls

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Abstract. Cold formed steel shear panel is one of the main components to bearing lateral load in low and mid-rise cold formed steel structures. This paper uses finite element analysis to evaluate the stiffness, strength and failure mode at cold formed steel shear panels whit steel sheathing and nonlinear connections that are under monotonic loading. Two finite element models based on two experimental model whit different failure modes is constructed and verified. It includes analytical studies that investigate the effects of studs and steel sheathing thickness changes, fasteners spacing at panel edges, one or two sides steel sheathing and height-width ratio of wall on the lateral load capacity. Dominant failure modes include buckling of steel sheet, local buckling in boundary studs and sheet unzipping in the bottom half of the wall.

Keywords: cold formed steel shear wall; finite element analysis; steel sheathing; maximum lateral resistance; nonlinear connections

1. Introduction

The cold formed steel family consists of two types' lateral load bearing system, light gauge steel frame and load bearing walls system. Cold-formed steel shear walls with various sheathing are main member in lateral load bearing wall system. This walls frame is configured by vertical and horizontal cold formed steel sections that sheeted by steel straps bracing, gypsum sheathing boards, plywood, OSB boards or steel sheets but history of steel sheathing is about last decade.

A few experimental studies on cold formed steel shear walls with steel sheathing have been done. The main experimental studies on cold formed steel shear walls, in 1997 by Serrette at the Santa Clara University have been done. Part of his research included a monotonic and cyclic test on steel sheet sheathing with thickness 0.457 mm and 0.686 mm (Serrette 1997). These studies continued in 2002 (Serrette 2002). Cheng Yu at the University of North Texas conducted experimental studies in this area in 2007. Comprehensive tests on 60 samples steel sheeted cold formed steel shear walls to provide shear strength values of different walls was performed to

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expand the design information (Yu 2007). In 2009, Cheng Yu performed further studies. Due to the lack of complete agreement with the results obtained in 2007 against the values given in the AISI standard conducted a series of new experiments was necessary. So the first part of 2009 to scrutinize and provide revised values of shear strength were assigned (Yu and Chen 2009). Few experimental studies in steel sheeted cold formed steel shear walls relatively make a few finite element studies in this area.

In 2006, Telue and Mahendran used finite element method to investigate and design a stud that connected to plasterboard under axial compressive load. In The finite element model, stud and plasterboard are modeled by shell elements. In addition, a beam element is used for screws. The interaction surfaces and the residual stresses were obtained in the model to achieve real results. Axial load-deflection curve of the finite element model showed good agreement with the experimental model. The design was based on the in plan coefficients of the effective length, and the ultimate load values obtained (Telue and Mahendran 2004). Esmaeili et al at the nearest study in 2012 examined the finite element and numerical steel-sheeted cold formed steel shear walls. Numerical modeling by ABAQUS software were taken and analyzed to evaluate the effects of height-width ratio of wall, studs and steel sheathing thickness changes and fasteners spacing at panel edges under monotonic loading. Verify the modeling is done on Balh MsC thesis (Balh 2010). The main failure modes of steel-sheathed cold formed steel shear walls were the buckling of the steel sheathing, flange distortion of the boundary studs and pullout of sheathing screws. The results were expressed higher levels of thickness for steel sheet and other sections have little effect in increasing the lateral load bearing. Also reducing the fasteners spacing at panel edges, increases the maximum shear strength of wall (Esmaeili Niari *et al*. 2012). In 2012 Besevic conducted an experimental investigation of residual stresses in cold formed steel sections. Two methods of residual stresses investigation is presented and compared with results of other reaserch in this case (Besevic 2012). In 2013, Kripka *et al*. by simulated annealing method tried to maximum performance of Cold-formed steel channel columns with low use of material (Kripka and Martin Chamberlain Pravia 2013). In 2013, a Study on the effect of ties in the intermediate length Cold Formed Steel columns presented by Anbarasu et al. Non-linear numerical analysis using ANSYS is performed to simulate the experimental results. Results show using ties in intermediate length of column not only improve the failure mode but also increase the final strength of columns (Anbarasu *et al*. 2013). In 2014, Kwon *et al*. conducted Compression tests of cold-formed channel sections with perforations in the web. The results indicate that the slits in the web had significant negative effects on the ultimate strength of thin-walled channel section columns and buckling mode (Kwon *et al*. 2014). In 2014 Rosario-Galanes and Godoy presented modeling of wind-induced fatigue of cold-formed steel sheet panels. An analytical model developed and validated for the fatigue life prediction and failure mechanism of different connection types and thicknesses of cold-formed steel cladding (Rosario-Galanes and Godoy 2014).

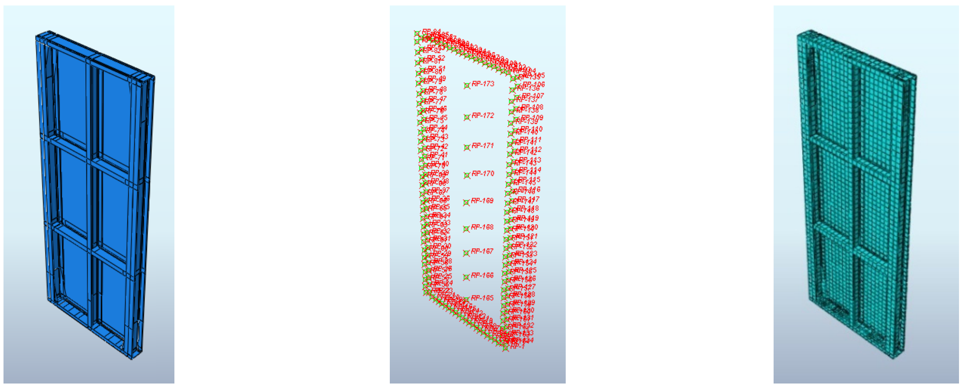
In this paper, the effects of stud's and steel sheathing thickness changes, fasteners spacing at panel edges and one or two sides steel sheathing under monotonic loading, on lateral load capacity is investigated.

2. Finite element modeling

The best way to study the behavior of structures is full- scale testing. High number of these experiments has been done on cold formed steel structures and codes and standards provide design equations are based on the same results tests (AISI Lateral Standard 2004). But full scale testing always has its costs. Finite element method with high accuracy and overcome the costs of full scale experiments provide the desired response. Finite element analysis software used in this study is, ABAQUS/ standard.

2.1 Geometry and material

Models built based on experimental studies at Tehran University by Mohebbi *et al*. (2015). The wall Included double studs at boundary, and single stud in the middle of the wall. The height-width ratio of wall is 2.5 and height of the wall is 3 meters. Dimensions of studs are: web 150 mm, flange 50 mm, and lip on the edges of the flange is 12 mm. Two horizontal elements at levels 1 m and 2 m from the ground surface are applied to the walls configuration. Steel sheet fasteners to the frame are screws No. 8. You can see the geometric details of the wall in Fig. 1.
For accurate modeling, material behavior is modeled as a homogeneous and nonlinear. Frame and steel sheet material are the same. The stress-strain curve is based on tensile coupon test (Fig. 2). Input table for ductile behavior (large inelastic strains) in ABAQUS inelastic analysis. Stress and strain measures are “true” stress (Cauchy stress) and logarithmic strain. If the nominal (engineering) stress-strain data for a uniaxial test and the material is isotropic, a simple conversion to true stress and logarithmic plastic strain is



(a) Geometry and components (b) Screws (c) Meshing

Fig. 1 Details of the model

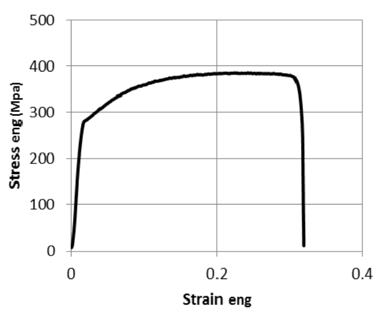


Fig. 2 The engineering stress-strain curve for steel material

$$\sigma_{true} = \sigma_{eng}(1 + \varepsilon_{eng}) \tag{1}$$

$$\varepsilon_{true} = \ln(1 + \varepsilon_{eng}) - \frac{\sigma_{true}}{E} \tag{2}$$

Where σ_{eng}, ε_{eng}, ε_{true} respectively are engineering stress, engineering strain, true stress, true strain and E is the Young's modulus (Phama and Hancock 2010). The true yield stress is 284 Mpa and the true ultimate tensile stress, is equal 494 Mpa.

2.2 Analysis and meshing

Due to the low-speed testing, static analysis is used for finite element model. Static analysis is for problems in which we can ignore the effects of inertia. This analysis is also able to solve linear or nonlinear problems. The analysis also considered the effect of large deformations. There is not a restriction on the time steps increment size while many local buckling due to the low thickness of the sections, the analysis does not stop. However, this increases the analysis time. Shell elements family is used in the model. Selected element S4R, element has four nodes, the linear Geometric order is used for thick and thin shells. In addition to these elements, reduced integration techniques have also been used. Size of meshes considered equal to 50 mm.

2.3 Interactions and connections

The properties defined in the interaction between two steel plates such as damping, tangential and normal behavior. For calculating the elastic sliding friction in tangential behavior, the penalty coefficient used and the amount equal to 0.2 is considered. In normal behavior selected the hard contact. The damping coefficient is 0.05 used for contact between the two steel sheets. Connections modeled by the point-based fasteners and Cartesian and Cardan used for connector sections. This type of connection provides connectivity between two nodes where response is defined along six local axis. Nonlinear behavior of connectors has been assigned (Table 1).

2.4 Loading and boundary conditions

At below and above the wall, two relatively rigid plates are used, as well as experimental conditions. Loading as a one-side displacement along the walls width on the upper plate, is applied. Displacement rate, 120 mm in line with the load applied to the wall that the incremental displacement is ramp and support forces at each step is recorded as base shear. All parts of the bottom plate in the six degree of freedom for the transitional and rotational constrained and anchor bolts have been modelled in the bottom of the wall.

Load values are varied for each model according to its thickness. These values are obtained based on the following empirical relationship.

$$[\text{Load values}] = \begin{bmatrix} 0 \\ 47.7 \\ 45.96 \\ 0 \end{bmatrix} \times 1000 \times t \times (1.2) \tag{3}$$

Table 1 Connector's nonlinear behavior

	Slip (mm)	Load (N)
Model 1: F 1.25- Sh 0.6	0	0
	1.1	88245
	1.2	85000
	4.7	0
Model 2: F 1.25- Sh 0.8	0	0
	0.9	117342
	1	113027
	1.7	0

Load values = empirical base values × kN to N × total thickness of the steel sheet and frame × (just for buckling models).

3. Verifying finite element model

To validate the finite element model presented in the previous section, curves of lateral strength versus lateral drift was obtained for two models with different specifications. AISI defined the minimum thickness for steel sheet and frame members respectively 0.457 and 0.838 mm (AISI Lateral Standard 2004). Thickness ratio of the frame members to steel sheet is 1.834. This ratio effects on the lateral behavior and failure modes of the wall. If thickness ratio of the frame members to steel sheet is less than this value then due to the ultimate capacity of the steel sheet, the compressive axial force in stud will arise. The result is the local buckling in boundary compressive stud. With stud local buckling and steel sheet unzipping of frame in the buckling area, lateral load bearing of wall reduce suddenly. On the other hand, there is a different kind of behavior at a higher proportion of the thickness of the frame members to steel sheet. In this type of configuration with increasing horizontal displacement at top of wall And sheet weaknesses relative to the frame, in the lower half of the walls and the bottom corners, Steel sheet in the surrounding area of screws were decayed and in some cases were torn. In these conditions the reciprocating motion of the walls, unzip steel sheet of frame.

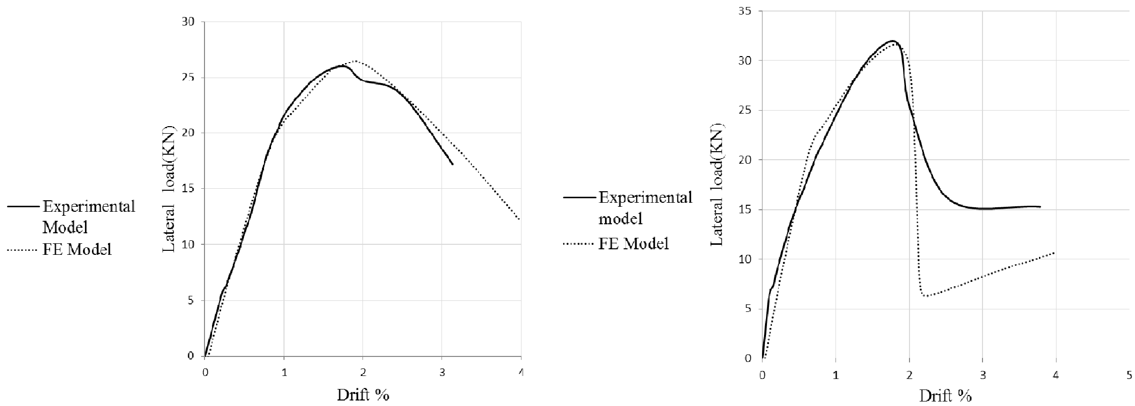
For these reasons, two various model conducted for less or more than 1.834 ratio. First model for steel sheet unzipping (more than 1.834 ratio) and second model for stud local buckling and steel sheet unzipping (less than 1.834 ratio).

In order to validate the finite element model that described in previous sections compared its lateral load to drift curve with experimental model. These curves and failure modes investigated. Results show Accuracy of the finite element model (Figs. 3 and 4). The following nomenclature is used in the models.

F1.25- Sh0.6- 5 cm- OF or BF

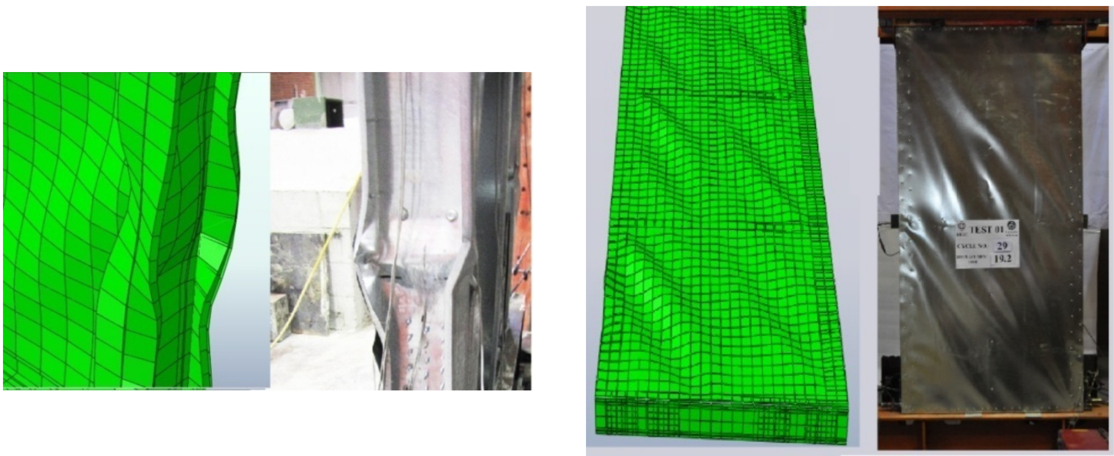
These means as:

- F: Frame thickness
- Sh: Sheet thickness – fasteners spacing at panel edges
- OF or BF: One face sheathing or both face sheathing



(a) F 1.25- Sh 0.6 verifying (b) F 1.25- Sh 0.8 verifying

Fig. 3 FE model verse experimental model



(a) Stud buckling (b) Steel sheet buckling

Fig. 4 Comparing failure modes of finite element model with experimental model. F 1.25- Sh 0.6

The model F 1.25- Sh 0.6 dominant failure mode, is buckling of steel sheet. With the increase in buckling, unzipping sheet occurs at the bottom half of the wall. Results show Steel sheet in the surrounding area of screws were decayed and in some cases were torn. The Model F 1.25- Sh 0.8 dominant failure mode, is stud local buckling and steel sheet unzipping of frame in the buckling area. Local buckling in stud Occurs at the above the hold-down. These failure modes have been seen in both finite element and experimental models. Failure procedures at both first and second finite models match to their experimental models.

4. Analytical study

The frame members in shear wall load bearing path must bear maximum forces caused by steel sheet stretch-diagonal performance. The weaknesses of each member of this pathway disturb wall performance and the ultimate lateral resistance is reduced.

4.1 The effect of changing the thickness of the steel sheet or frame on the lateral load capacity

The analysis was done based on the finite element model for steel thicknesses, 0.6, 0.8, 1 and 1.25 mm and the thickness of the frame 1.25, 1.5, 2 and 2.5 mm is regarded. Sixteen permutations of this thickness composition are investigated. In Table 2, the ultimate lateral strength with corresponding drift percent and failure mode and in Fig. 5 lateral-load to drift curves for theses are shown.

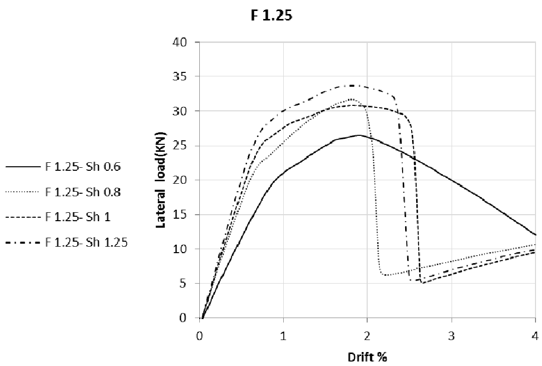
4.2 The effect of changing spacing between screws at panel edge on the lateral load capacity

Studies show that only 8% of the ultimate strength of cold formed steel wall is by frame without sheathing. Most of shear capacity depends on the sheet resistance and fasteners. Wall nonlinear behavior depends on the nonlinear behavior steel frame screws. Even in serrette experimental study connections spacing, is effective. In a series of his tests, spacing between screws in the middle and at the edges of walls respectively 12 in. and 6 or 2 in. has been used. The results showed that by reducing spacing between screws at edges the wall lateral strength could raise. This section examines the effect of the spacing between screws at panel edge on the ultimate shear strength. Diagrams obtained for these studies are provided at Fig. 6. The results showed that

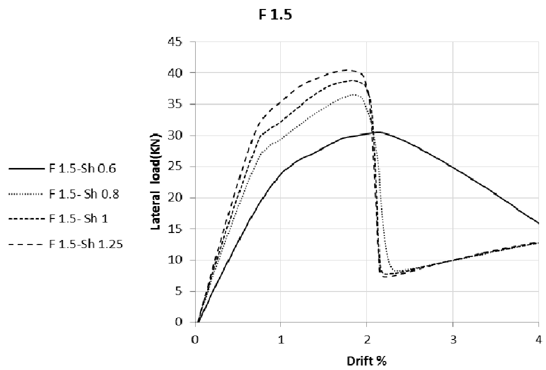
Table 2 Results for 16 samples of different thickness

		Drift %	The ultimate shear strength	Failure mode
1	F 1.25- Sh 0.6	2.83	26.49	U
2	F 1.25- Sh 0.6	2.05	31.69	S
3	F 1.25-SH 1	2.55	30.86	S
4	F 1.25-SH 1.25	2.36	33.68	S
5	F 1.5-SH 0.6	3.06	30.47	U
6	F 1.5-SH 0.8	2.09	36.57	S
7	F 1.5-SH 1	2.07	38.84	S
8	F 1.5-SH 1.25	2.05	40.52	S
9	F 2- SH 0.6	3.60	38.56	U
10	F 2- SH 0.8	2.88	41.35	U
11	F 2- SH 1	2.56	43.81	U
12	F 2- SH 1.25	2.02	51.63	S
13	F 2.5- SH 0.6	4.00	47.73	U
14	F 2.5- SH 0.8	3.24	50.03	U
15	F 2.5- SH 1	2.70	52.71	U
16	F 2.5- SH 1.25	2.36	55.66	U

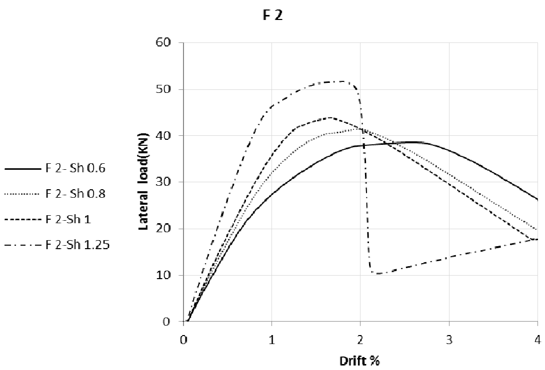
U: Unzipping steel sheet failure mode
S: Stud local buckling and steel sheet unzipping of frame in the buckling area failure mode



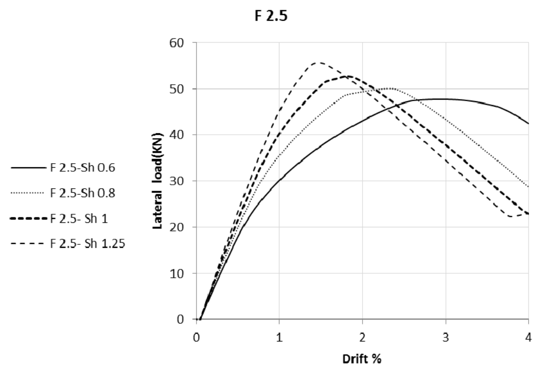
(a) F 1.25



(b) F 1.5

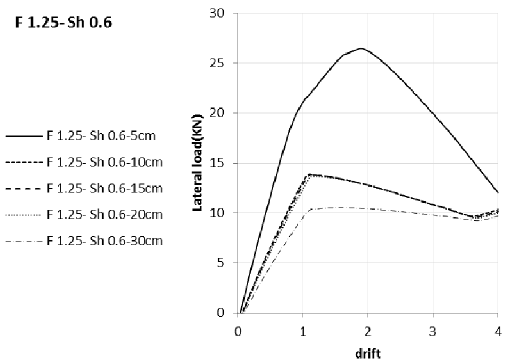


(c) F 2

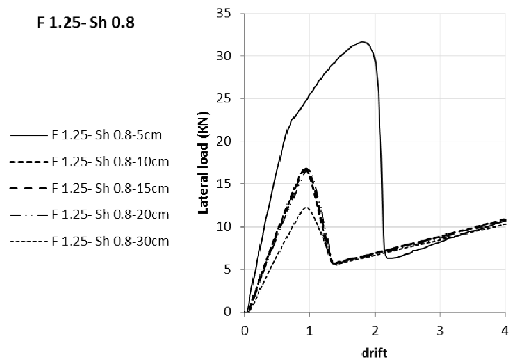


(d) F 2.5

Fig. 5 The effect of changing the thickness of the steel sheet or frame on the lateral load



(a) F 1.25- Sh 0.6



(b) F 1.25-Sh 0.8

Fig. 6 The effect of changing spacing between screws at panel edge on the ultimate shear strength

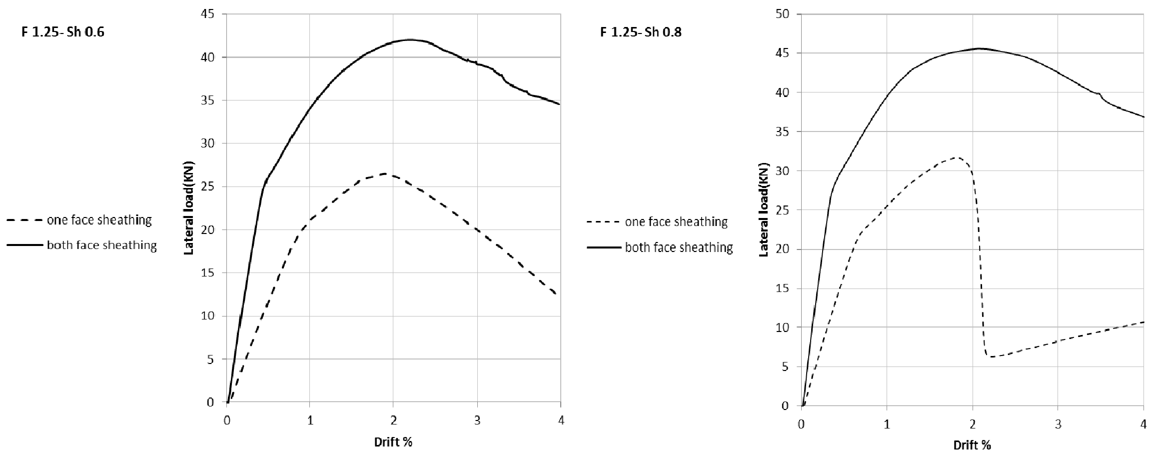
by reducing spacing between screws at edges the wall lateral strength could raise. This is consistent with experimental results.

4.3 The effect of one or two sides steel sheathing on the lateral load capacity

Using steel sheet sheathed on both sides for achieving greater strength and stiffness for per panel. This section examines the effect of one or two sides steel sheathing on lateral load capacity. Diagrams obtained for these studies are provided at Fig. 7. With two sides steel sheathing all parts of bearing route must be strengthened. In two sides sheathed wall than one side lateral load capacity rise about 60%. If boundary element of walls does not buckle under the pressure load, the lateral strength amount to 1.8 to be increased (Serrette 1997).

4.4 The effect of height-width ratio of wall on the lateral load capacity

Considering the constant height of the wall, 2.4 m, for the three height-to-width ratio of 1, 2 and 4 is discussed. As shown in the Fig. 8 the height-width ratio, 2, has more energy absorption capacity and ductility than the other cases. Also in the comparison between the height-width ratio of 2 and 2.5, the ratio of 2 has better response parameters. For the height-width ratio of 2, ductility



(a) F 1.25- Sh 0.6 (b) F 1.25-Sh 0.8

Fig. 7 The effect of one or two sides steel sheathing on the lateral load capacity

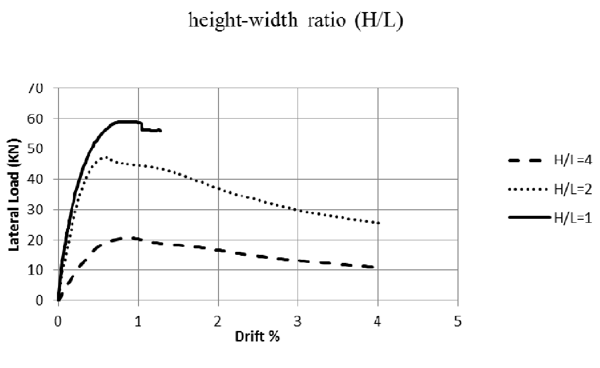
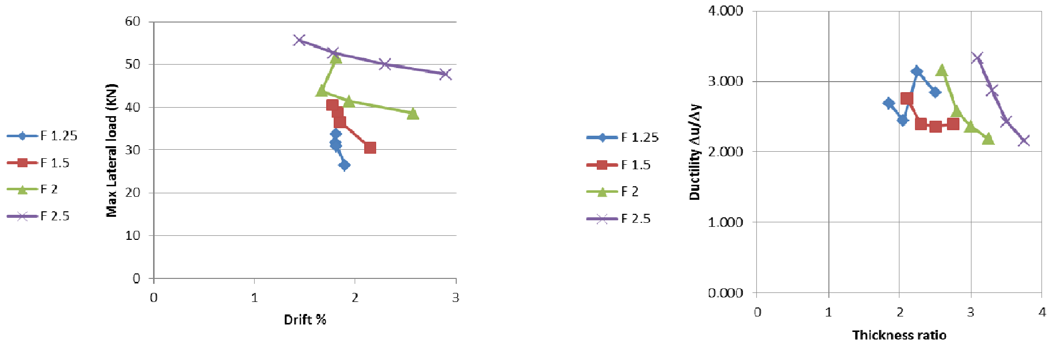
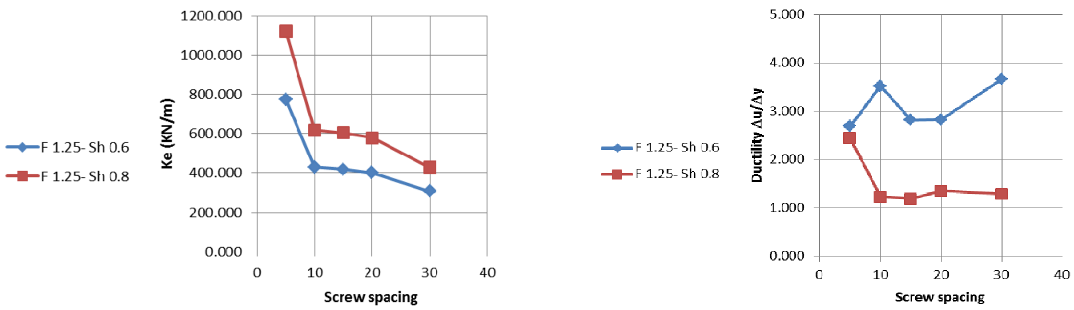


Fig. 8 The effect of height- width ratio of wall on the lateral load capacity



(a) On the maximum lateral load capacity (b) On the ductility

Fig. 9 The effect of studs and steel sheathing thickness changes, on the maximum lateral load capacity



(a) On the lateral stiffness (b) On the ductility

Fig. 10 The effect of fasteners spacing at panel edges

and maximum lateral load than height-width ratio of 2.5 is respectively 2.51 and 1.78 times increased.

4.5 The parametric analysis graphs

In this part, effects of studs and steel sheathing thickness changes and fasteners spacing at panel edges, on the maximum lateral load capacity and ductility in Figs. 9 and 10 is investigated. In each category of thickness of the studs with increasing in steel sheathing thickness, the maximum lateral load is increased and the drift amount is decreased as shown in Fig. 9. Increasing the fasteners spacing has significant effect in reducing the lateral stiffness and ductility. As shown in Fig. 10 the ductility is increased by increasing the thickness of the studs and Increasing in the thickness difference between studs and steel sheathing.

5. Conclusions

With increasing steel sheet or frame thickness increases the wall lateral strength and stiffness. By doubling thickness of the frame, the ultimate strength of the wall is increased about 94%. In results is observed when the frame members are not damaged during bearing, with increasing thickness, as well as the ultimate strength of the shear wall increases. The results indicate an important effect on the ultimate lateral strength amount is due to space between screws. Wall lateral strength increases with decreasing spacing between the screws. Whatever the distance is less than or greater than their influence shows greatly. If you change the spacing between the screws that are leading to changes in the stretching-diagonal process of wall, then the resistance values will be affected by the spacing between the screws, otherwise spacing changes does not affect the ultimate lateral strength. Two sides steel sheathing for wall increase in lateral stiffness is about twice. However, the resistance is increased about 60%. The height-width ratio of 2 has better response parameters than other cases.

References

AISI Lateral Standard (2004), AISI Standard for Cold-Formed Steel Framing; (Lateral Design 2004 Edition), American Iron and Steel Institute, Washington, DC, USA.

Anbarasu, M., Bharata Kumar, S. and Sukumar, S. (2013), "Study on the effect of ties in the intermediate length Cold Formed Steel (CFS) columns", *Struct. Eng. Mech., Int. J.*, **46**(3), 323-335.

Balh, N. (2010), "Development of seismic design provisions for steel sheathed shear walls", Ms.C. Thesis; Department of Civil Engineering and Applied Mechanics, McGill University, Canada.

Besevic, M. (2012), "Experimental investigation of residual stresses in cold formed steel sections", *Steel Compos. Struct., Int. J.*, **12**(6), 465-489.

Esmaeili Niari, S., Rafezy, B. and Abedi, K. (2012), "Numerical methods finite element analysis of steel sheathed cold-formed steel shear walls", *Proceedings of the 15th World Conference on Earthquake Engineering* (*15WCEE*), Lisbon, Portugal, September.

Kripka, M. and Martin Chamberlain Pravia, Z. (2013), "Cold-formed steel channel columns optimization with simulated annealing method", *Struct. Eng. Mech., Int. J.*, **48**(3), 383-394.

Kwon, Y.B., Kim, G.D. and Kwon, I.K. (2014), "Compression tests of cold-formed channel sections with perforations in the web", *Steel Compos. Struct., Int. J.*, **16**(6), 657-679.

Mohebbi, S., Mir Ghaderi, S.R., Farahbod, F. and Bagheri Sabbagh, A. (2015), "Experimental work on single and double-sided steel sheathed cold-formed steel shear walls for seismic actions", *Thin-Wall. Struct.*, **91**, PP 50-62.

Phama, C. and Hancock, G. (2010), "Numerical simulation of high strength cold-formed purlins in combined bending and shear", *J. Construct. Steel Res.*, **66**, 1205-1217.

Rosario-Galanes, O. and Godoy, L.A. (2014), "Modeling of wind-induced fatigue of cold-formed steel sheet panels", *Struct. Eng. Mech., Int. J.*, **49**(2), 237-259.

Serrette, R.L. (1997), "Additional Shear Wall Values for Light Weight Steel Framing", Report No. LGSRG-1-97; Santa Clara University, CA, USA.

Serrette, R.L. (2002), "Performance of Cold-Formed Steel-Framed Shear Walls: Alternative Configurations", Report No. LGSRG-1-97, Santa Clara University, CA, USA.

Telue, Y. and Mahendran, M. (2004), "Behavior and design of cold-formed steel wall frames lined with plasterboard on both sides", *Eng. Struct.*, **26**(5), 567-579.

Yu, C. (2007), "Steel Sheet Sheathing Options for Cold-Formed Steel Framed Shear Wall Assemblies Providing Shear Resistance", Report No. UNT-G76234; submitted to American Iron and Steel Institute, Washington, DC, USA.

Yu, C. and Chen, Y. (2009), "Steel Sheet Sheathing Options for Cold-Formed Steel Framed Shear Wall Assemblies Providing Shear Resistance– Phase 2", Report No. UNT-G70752; submitted to American Iron and Steel Institute, Washington, DC, USA.

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