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Cyclic Loading Tests for Cold-Formed Steel Wall Frames with Lightweight Concrete

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ABSTRACT: Lightweight steel framing is a method in housing and construction that have been widely used in lightweight steel construction. In this method, the structure is built by cold formed steel elements. They are cost-effective, light, and easy to assemble. However, the performance of lateral load resisting systems in cold-formed steel structures specially the behavior of cold-formed steel shear walls filled with lightweight structural concrete under seismic loads has not been studied in detail. In this study, an experimental investigation on cold-formed steel frames filled with lightweight structural concrete has been conducted and the results are presented. Six full-scale cold-formed steel frames filled with lightweight structural concrete with two different configurations were studied. The test was performed under a standard cyclic loading regime. This study is focused on the ultimate lateral load capacity and seismic response modification factor of cold-formed steel walls filled with lightweight concrete subjected to cyclic loads. Based on the test observation, detailed discussions on the failure modes of cold-formed steel wall specimens are given. Finally, shear load resistance, seismic response modification factor, failure modes, energy dissipation and stiffness of tested shear walls are proposed and discussed. The results show that although lower height to width ratio leads to a greater shear load resistant, energy dissipation, and stiffness for shear wall filled with lightweight concrete, its seismic modification factor is lower than those shear walls, which have higher height to width ratio.

Keywords: Cold-Formed Steel, Lightweight Concrete, Seismic Response Modification Factor, Steel Shear Walls, Ultimate Lateral Load Capacity.

INTRODUCTION

The use of Cold-Formed Steel (CFS) structures has been significantly developed in the building construction industry because of its advantages like easy and fast assembly, high strength, and low cost. There are many experimental and numerical studies that have been conducted on the effect of using different lateral load resistant systems in CFS structures. In the most of these lateral load resisting systems, CFS frame is combined with steel sheathing plates, plywood, steel strap bracing, Oriented-Strand Board (OSB) and gypsum board sheathing that the lateral design information of these lateral resisting

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systems in CFS structures are available in the AISI S213 standard (2007). Javaheri Tafti et al. (2014) conducted an experimental investigation on cold formed steel frames sheathed by thin galvanized steel plates. The experiment involved 24 full-scale steel plated walls tested under cyclic loading with different configurations of studs and screws. Based on their obtained results, they concluded that decreasing the screw spacing from 150 mm to 100 mm enhanced the shear resistance capacity by around 16-18% but in the case of single end studs only. Finally, they recommended increasing the AISI R value of 6.5 to 7. Javaheri Tafti and Behnamfar (2013) presented a detailed research on seismic characteristics of CFS shear panels and effect of the number of chord studs on the mentioned parameters. They studied six full-scale 1200×2400 mm shear walls sheathed by steel plates under a cyclic loading. They resulted that a larger enclosed area in the hysteretic curves will result a more desirable lateral load resisting response and higher seismic response modification factor for these panels. They also found that the number of chord studs does not affect the R-factor substantially. Lin et al. (2014) investigated the structural strength and seismic properties of cold-formed steel shear wall sheathed with calcium silicate board under shear loading. They used two different thicknesses of sheathing, 9 mm and 12 mm, with one-side or two-side of attachment in the assembly of test specimens. Nine cold-formed steel wall specimens that were tested in their study, subjected to monotonic shear load and cyclic loads. Finally, based on the test results, for design purpose, they recommended the value of 4.2 for response modification factor (R) of the steel framing wall sheathed by calcium silicate board. They also reported that the wall specimens with two-side sheathings provide higher ultimate strength, stiffness, and energy absorption as compared with those having one-side sheathing. DaBreo et al. (2014) carried out a loading test research on steel sheathed CFS shear walls with different configurations. Their aim was studying the performance of specimens to institute a general database for developing a pursuant design method. Totally 68 specimens differed in framing thickness, sheathing thickness, screw fastener detailing, aspect ratio and framing reinforcement were tested. Based on their results, the use of closely spaced sheathing panel fasteners and thicker panels leads to a higher shear resistance if the stud members are designed to carry the additional force. Yu (2010) carried out a research investigation to add some missing and required shear strength values in design codes. There are many numerical studies that have been conducted on CFS shear load resistant systems. Dai (2012) studied the performance CFS shear walls under in-plane monotonic loads by a new numerical modeling method. He concluded that the shear performance of mentioned shear walls depends on the mechanical properties of affixed sheets and connectors. Gerami and Lotfi (2014) carried out a numerical study using Finite Element Nonlinear Analysis on 112 CFS shear walls with various configuration in bracing arrangement, aspect ratio and sheathing plate thickness. Also, cyclic and monotonic loading were utilized in the test program. A nonlinear monotonic analysis carried out by Esmaeili Niari et al. (2013) on some cold formed steel shear wall frames sheathed by steel plate. The study was performed by utilizing Finite Element method. Based on the results, using steel sheathing plate with higher thickness and also thicker stud will not effect on shear load resistant of CFS steel plate sheathed shear walls. Recently, Mohebbi et al. (2016) investigated the seismic behaviour of CFS shear walls sheathed by steel sheet using gypsum and fiber cement board claddings under cyclic loading. Liu et al. (2014) carried out a research aimed to investigate the effect of practical construction details on the cyclic performance of CFS shear walls sheathed with Oriented Strand Board. For evaluating the performance of cold-formed steel shear walls sheathed by steel plate under dynamic loading, Shamim and Rogers (2013) performed a numerical study to describe the numerical modelling using OpenSees of shear walls constructed of cold-formed steel framing and flat steel sheathing. For evaluating the seismic response modification factor of shear walls, Abdollahzadeh and Malekzadeh (2013) carried out a study for investigating the ductility, over-strength and seismic response modification factors of coupled steel shear wall frames. In addition, Mahmoudi et al. (2016) studied the seismic response modification factor of concrete coupled shear wall structures with various length/depth ratios of spandrel beams.

Although there are many studies on lateral load-resisting systems, most of these studies have focused on some particular lateral load-resisting systems (as CFS walls with CFS sheet sheathing). On the other hand, CFS structures with these kinds of lateral load resisting systems are facing with height limitations. Hence, it is required to research on the innovative lateral load resisting systems with more shear resistance to increase the allowable height in these structures. In addition, in cold-formed steel buildings, owners often need to connect and install especial implements in their buildings. Hence using lightweight and structural concrete to fill the hollow spaces of some CFS panels and utilize them as the shear wall in cold-formed steel structures can meet both structural and non-structural aims. Using cold-formed steel shear walls filled with lightweight concrete as lateral load resisting system in cold-formed steel structures is a new method of strengthening in CFS structures. However, there is no adequate and practical information about lateral performance of these kinds of shear load resisting systems. In addition, there is no design information about these systems in design codes. Hence, an experimental study for investigating the seismic and load-deformation behavior of CFS wall frames filled with lightweight concrete has been conducted. Totally, six cold-formed steel shear wall filled with lightweight concrete have studied. Two different types of frames with different aspect ratios were used in the assembly of test specimens. Cyclic loading regime was utilized for testing specimens. The shear resistance, seismic response modification factor, energy dissipation, stiffness and failure modes of each test specimen are presented in this paper.

MATERIALS AND METHODS

Seismic Modification Factor (R)

In Javaheri Tafti and Behnamfar (2013) study and based on Uang (1991), FEMA 450-2 (2003), a method for estimating the response modification factor is presented that is also adopted in this study for computing the response modification factor (R). Based on Javaheri Tafti and Behnamfar (2013) study, “the response modification factor generally is expressed in terms of its two main components: ductility reduction factor (R_d) and structural over-strength factor (Ω_0)” (Uang, 1991; FEMA 450-2, 2003). The R factor is described as (Javaheri Tafti and Behnamfar, 2013):

$$R = R_d \times \Omega_0 \quad (1)$$

In Figure 1, which is adopted from Javaheri Tafti and Behnamfar (2013) study which is based on FEMA concepts, the actual and the elastic performance of a structural system and the idealized bilinear force-displacement curve are shown. In addition, the constitutive parts of the response modification factor are evaluated using

Figure 1, as (Javaheri Tafti and Behnamfar, 2013):

$$R_d = \frac{V_e}{V_y} \;,\; \Omega_0 = \frac{V_y}{V_s} \tag{2}$$

Moreover, the *R* factor can then be reproduced as:

$$R = R_d \times \Omega_0 = \frac{V_e}{V_y} \times \frac{V_y}{V_s} = \frac{V_e}{V_s} \tag{3}$$

where V_e, V_y and V_s represent the structure's elastic response strength, the idealized yield strength and the first "significant yield" strength respectively.

For idealizing a force-displacement curve (actual performance curve of a structural system), the method based on FEMA 356 (2000), has been utilized. Based on this method, the idealized bilinear curve is formed by two lines. These two lines should be placed in such a way that the area of the top and bottom of the curve are equal. In addition to mentioned condition, for idealizing actual curve two conditions must be fulfilled: 1) The first portion line must cross the actual curve at $0.6V_y$ point; 2) The second portion line must be placed so that crosses the actual curve at the target displacement (or Δ_t). Based on FEMA 450-2 (2003), target displacement is the maximum allowable inter-story drift (2.5%) or 20% reduction in the load whichever occurred earlier. In this study target displacement is equal to 60 mm (Δ_t = 60 mm) (Javaheri Tafti et al., 2014).

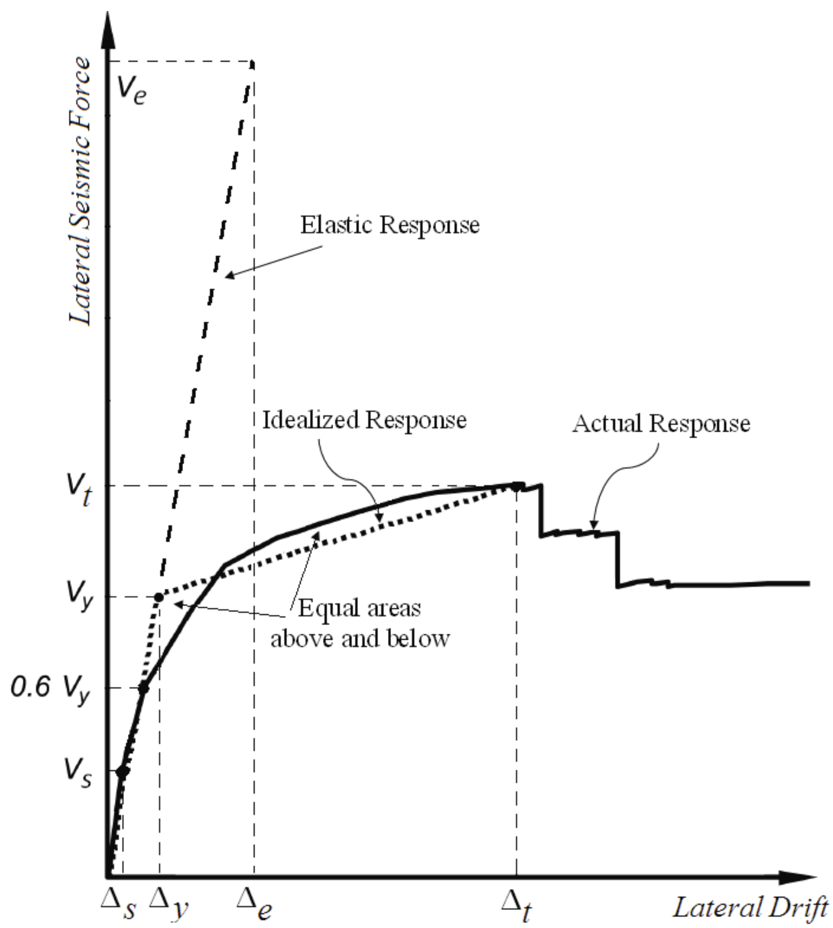


Fig. 1. General structural response curve adopted from Javaheri Tafti and Behnamfar (2013) study which is representing FEMA's concepts (FEMA 356, 2000)

Testing Rig and Instrumentation

The diagram of a specimen on testing rig is shown in Figure 2. For replacing each specimen on the testing rig, each wall panel is fixed on the rig between the upper and bottom beams. The top beam is fixed and has supporting duty. Cyclic loading is also applied to specimens by rigid loading bottom beam. Four Hold Downs devices at the four corner of each specimen are used to prevent any movement and slippage between the specimens and the beams. In addition, an accurate Horizontal Drift (HD) transducer is used for measuring the horizontal displacement of the bottom track (Javaheri Tafti and Behnamfar, 2013). In addition, one load-cell is utilized to measure the amount of shear resistance. Computer using Lab View Signal Express software (LabVIEW, 2007) analyzes measured amounts of horizontal displacement and shear resistance. By utilizing the analyzed data gathered from computer, the load-displacement curve of each specimen is then sketched (Javaheri Tafti and Behnamfar, 2013).

Loading Protocol

For testing all specimens, the cyclic loading regime has been utilized in this research. The cyclic loading regime is based on the Method B of ASTM E2126-07 (ASTM E2129, 2007), which was originally developed for International Organization for Standardization (ISO) standard 16670 and was used in Javaheri Tafti et al. (2014) recently. Method B of ASTM E2126-07 is composed of one full cycle at 0.5, 1, 2, 3, and 4 mm and three full cycles at 8, 16, 24, 32, 40, 48, 56, 64, and 72 mm, except if failure or a considerable reduction in the load resistance happens earlier (Javaheri Tafti et al., 2014).

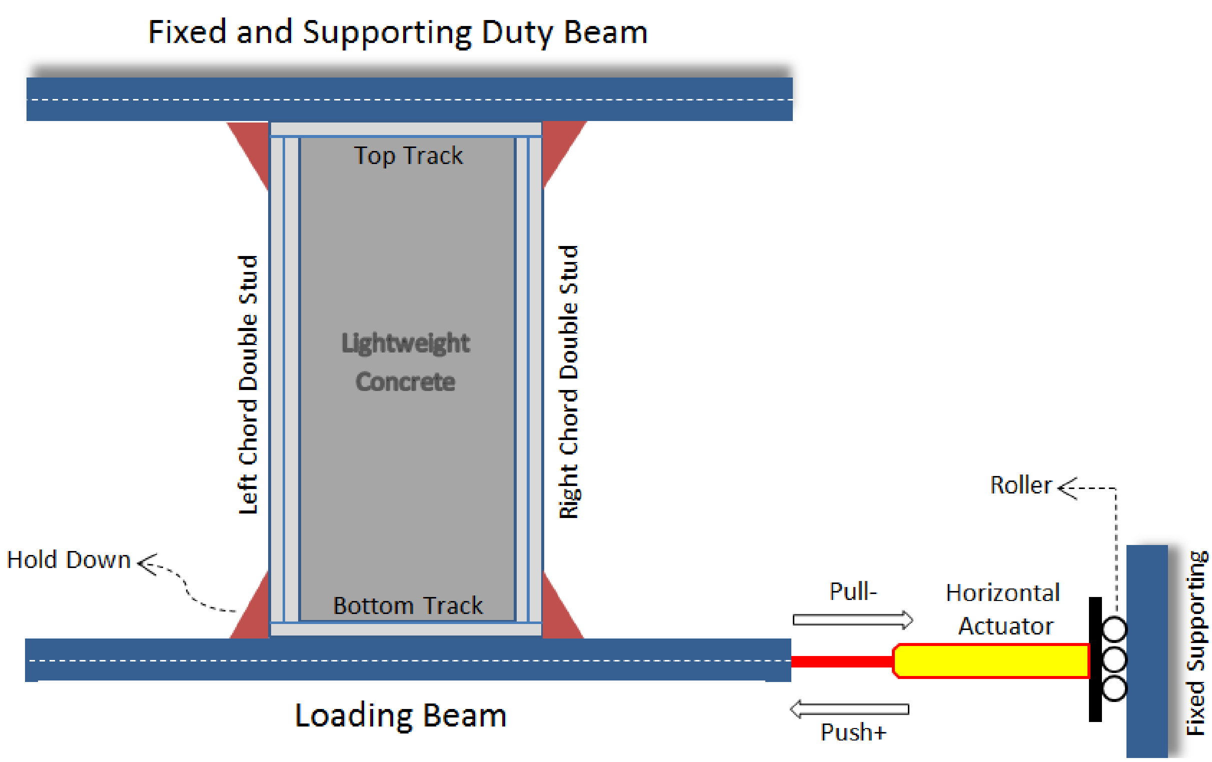


Fig. 2. The schematic of a specimen on testing rig

These lateral ranges are corresponding to 1.25%, 2.5%, 5%, 7.5%, 10%, 20%, 40%, 60%, 80%, 100%, 120%, 140%, 160%, and 180% of the final lateral displacement of the specimen. It should be noted that, based on Method B of ASTM E2126-07, the amplitude of cyclic displacements has to be selected based on fractions of monotonic ultimate displacement. As each specimen had its own ultimate displacement, if the amplitude of cyclic displacements were to be selected based on fractions of monotonic ultimate displacement in this study, the loading regime would have varied for each specimen. Since two types of shear walls with different configurations are compared in this study, it was necessary to use identical cyclic amplitudes for different walls. Hence, in this experimental research Method B is utilized with the lateral range independent of the monotonic test result (Javaheri Tafti et al., 2014). In addition, because of some laboratory limitations in maximum amplitude of testing rig actuator, all specimens were tested to 120 mm lateral displacement (ASTM E2129, 2007).

Test Specimens and Material Properties

To investigate the seismic characteristics of steel shear panels filled with lightweight structural concrete, two types of specimens were studied. Specimen type A with the dimension of 600×2400 mm and type D with the dimension of 900×2400 mm were built (Figure 3). For enhancing the accuracy of the test results, three panels of type A wall and three panels of type D wall were built. It should be noted that all type A and type D specimens have double back-to-back end studs as chord. All elements of the frames have been made from the same structural material. The C-section structural material properties and the detailed section geometry are presented in Table 1 and Figure 4, respectively. All the studs on the edge have been connected to the tracks by size 8 mushroom head screws. Tensile tests have been used in order to determine these specifications. Mechanical properties of the screws are provided in Table 2.

The hollow spaces between walls are filled with Lightweight Structural LECA (lightweight expanded clay aggregates) concrete. The concrete has been made by mixing of LECA aggregates, cement, natural sand and water. For LECA aggregates, the structural lightweight expanded clay aggregates were used. Type II Portland cement has been used. The stress – strain relationship of concrete was obtained via compressive tests on concrete samples (cylinder and square) that the used concrete mechanical properties are presented in Table 3 (ACI Committee 318, 2008).

Table 1. Mechanical properties of the C section stud

Fu/Fy	**Elastic Modulus**	**Ultimate Stress, *Fu***	**Yield Stress, *Fy***	**Nominal Thickness**
1.22	200 GPa	415 MPa	341 MPa	0.93 mm

Table 2. Mechanical properties of the screw

Tilting and Hold Bearing Capacity	**Tensile Strength Capacity**	**Shear Strength Capacity**
0.9 kN	12.1 kN	7 kN

Table 3. Concrete properties

Bending Strength	**Elastic Modulus**	**Slump**	**Compressive Strength of the Concrete, *F'c***	**Mass per Unit Volume**
3.6 MPa	18 GPa	110mm	24 MPa	1650 kg/m^3

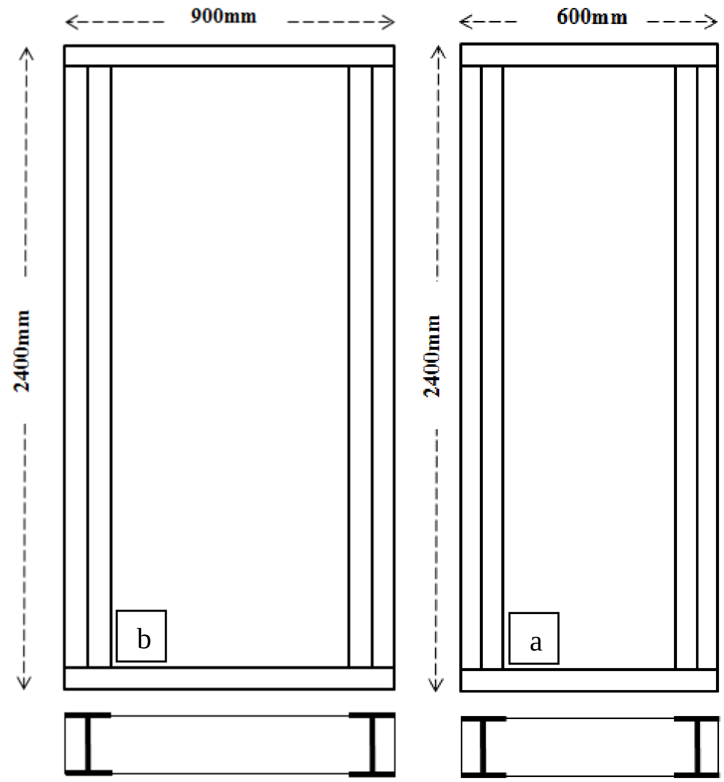


Fig. 3. Schematic representation of the tested panels, a) Type A and b) Type D

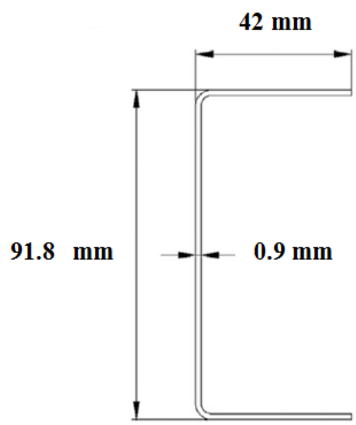


Fig. 4. Detailed dimensions of C-section stud

RESULTS AND DISCUSSION

Type A Wall

In type A wall's specimens, chord studs were made of double studs and the dimension of the type A walls was 600×2400 mm. Three walls of the same type were built for this type. The first mode that was appeared during the test was deformation in attachments zone near the supports, elastic buckling in the flange of bottom tracks followed by screw tilting and hole bearing at the top and the bottom tracks in the first cycle of +48 mm lateral displacement. In the most cycles of lateral displacements to 72 mm, distortional buckling at the middle of the top and bottom tracks and near the corners, screw pull-out of the track to chord connections were observed. In the first cycle of lateral displacements to +96 mm of wall A2 a remarkable decrease in the lateral load resistance (about 20%) occurred, due to tearing and bending of the bottom track's web at the location where the bottom track is connected to the base supporting beam. Hysteretic curves are shown in Figure 6. The combined hysteretic envelope curve of all three specimens of this type is shown in Figure 7. Failure modes for this type of specimens are shown in Figure 8. The nominal shear strengths are calculated as the peak load of load-displacement curve and are given in Table 4.

Fig. 5. CFS shear wall filled with lightweight concrete on testing rig

At each displacement amplitude, the racking resistance associated with the first cycle was significantly higher than those of the second and third cycles. The difference between the ultimate displacement in positive and negative directions confirms that the behavior of filled CFS walls with Lightweight Structural LECA concrete is reliable. After the tests, no damage was observed in the concrete.

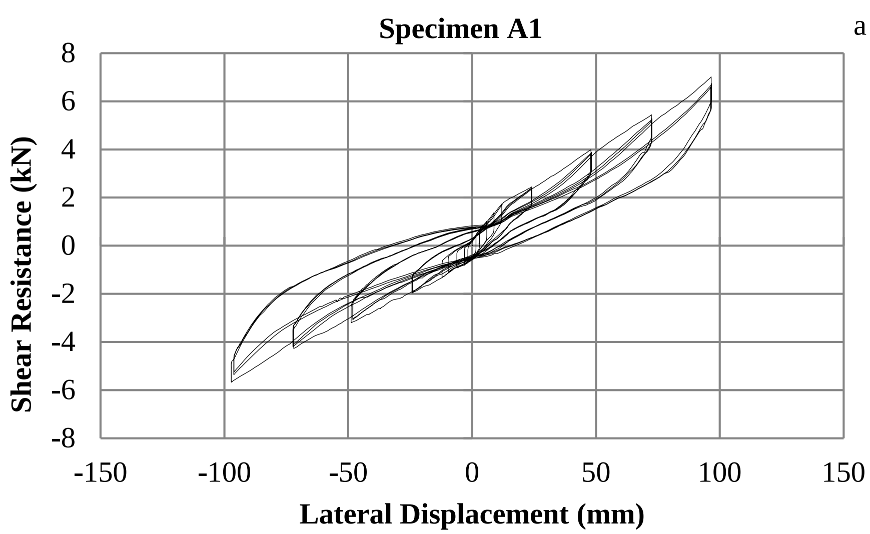
Type D Wall

Wall panels type D were then tested which were similar to wall panels type A with the only difference having width 900 mm instead of 600 mm. In addition, here a double back-to-back stud section was used as the boundary stud (like wall type A). In Figure 9, the hysteretic curves for the specimens of wall type D are presented. In addition, In Figure 10, the hysteretic envelope curves for mentioned specimens are presented. Failure modes for these panels are also shown in Figure 11. It is noted that because of technical problems during testing process, two of three specimens for type D walls were tested and the results were considered in this study. Primary failure mode in primary lateral displacements was chord to track connector screws deforming, especially near the hold-downs, that by increasing the lateral displacement amplitude, this failure mode was extended and polling out of the screws, (about half of the screws) was happened in the first cycle of 72 mm lateral displacement. In the last cycles of the 72 mm lateral displacement, the screw shear ruptures occurred mainly on the end of the top and bottom tracks at the loading side due to its concentrated force on the screws produced from the uplift loads of the chord studs. It was observed that, when the test load was increasingly applied to the chord studs, the screw gradually failed by shear rupture. Because of the shear ruptures in stud to track screws, two ends of top and bottom tracks were observed to be deformed after they were subjected to tension and/or compression forces induced from the wall panel in 96 mm lateral displacement. When the lateral loads were gradually applied, top and bottom channels were found to deform inward or outward for specimens under cyclic. Because the applied lateral loads have an instant large movement for specimens under cyclic load, the bottom channels were torn off at connected parts for some specimens. In 96 mm lateral displacement, distortional buckling in bottom track flange at the middle of the wall was also observed. At the end of the test, all screws had pulled over the boundary stud to track connections. Most of the decrease in the lateral load resistance capacity (about 15-20% decrease in shear resistance in 120 mm lateral displacement) was associated with local failure in the panel (bending in tracks web and bearing in stud-track connection) which is similar to the failure of test specimen A. In addition to mentioned failure modes of type D wall, separating out the concrete from top track in effect of rupture in chord to track screws and tearing failure at the end of top track, was observed. The ultimate strengths of two specimens in the same panel type D were almost identical as shown in Table 4. It should be noted that based on the assumption that the shear walls are designed just for lateral loads (the vertical and lateral loads are separately apply for design), the efficacies of vertical loading during these tests are neglected (Javaheri Tafti and Behnamfar, 2013).

Based on Table 4, type D shear walls with average ultimate load capacity of 13.9 kN have the higher shear resistance compared with type A shear walls with average ultimate load capacity of 7.2 kN. It seems that the 300 mm increase in the width of type D frame to type A frame can be the reason of this difference between shear resistance of type D and type A walls.

For evaluating the response modification factors, a procedure, which was used by Javaheri Tafti et al. (2014), is applied. According to the procedure, the specimens' hysteretic envelope curves are used by the following steps:

At first, the idealized bilinear curve is evaluated using the method presented in FEMA 356 (2000). Javaheri Tafti et al. (2014) expressed that the ductility reduction factor, R_d, is determined by using the first part of Eq. (2). Based on Eq. (2), for evaluating R_d, two parameters, V_e and V_y, are required. By utilizing Figure 1, V_e and V_y are calculated based on the concept of equal energy and the idealized bilinear curve, respectively.



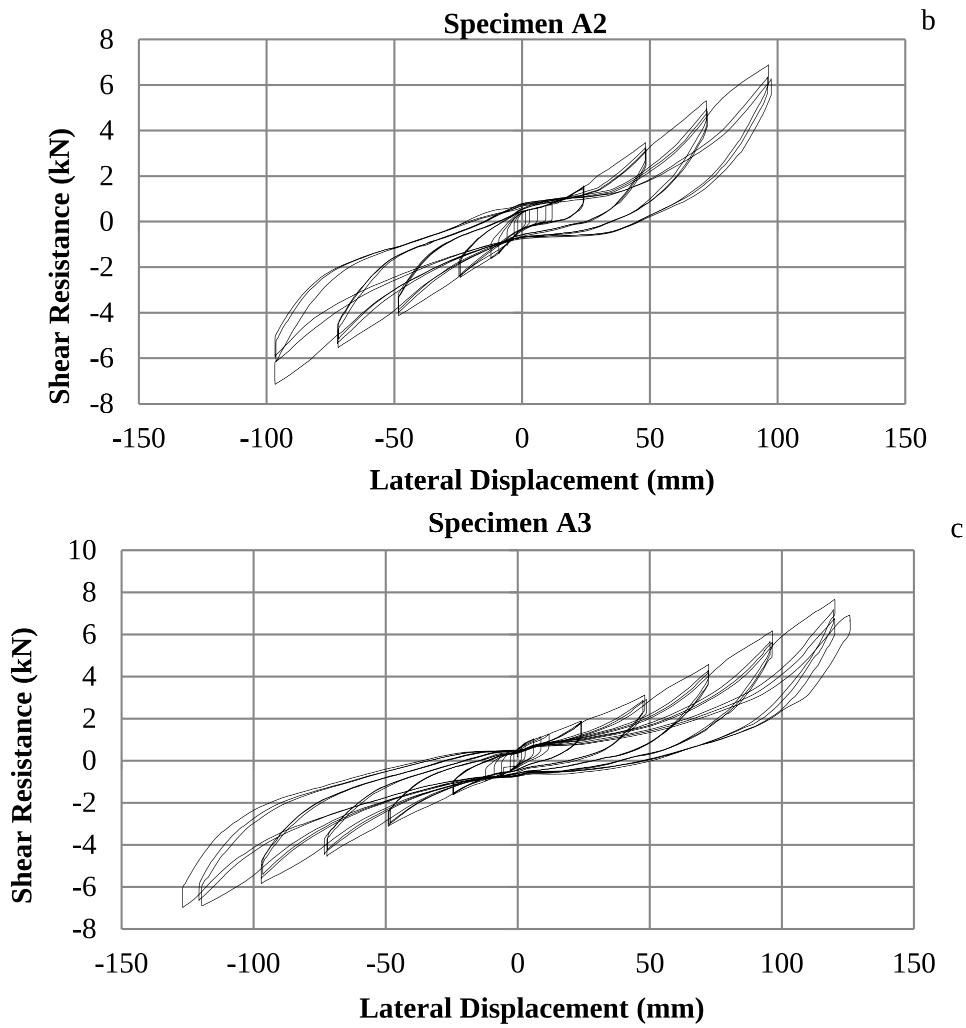


Fig. 6. Hysteretic curves for wall A specimens

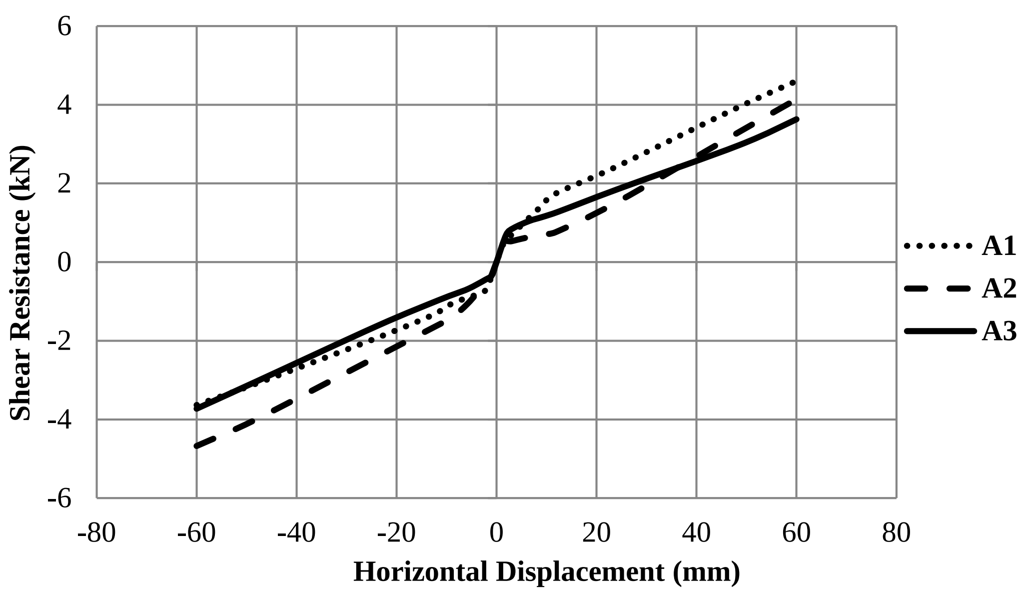
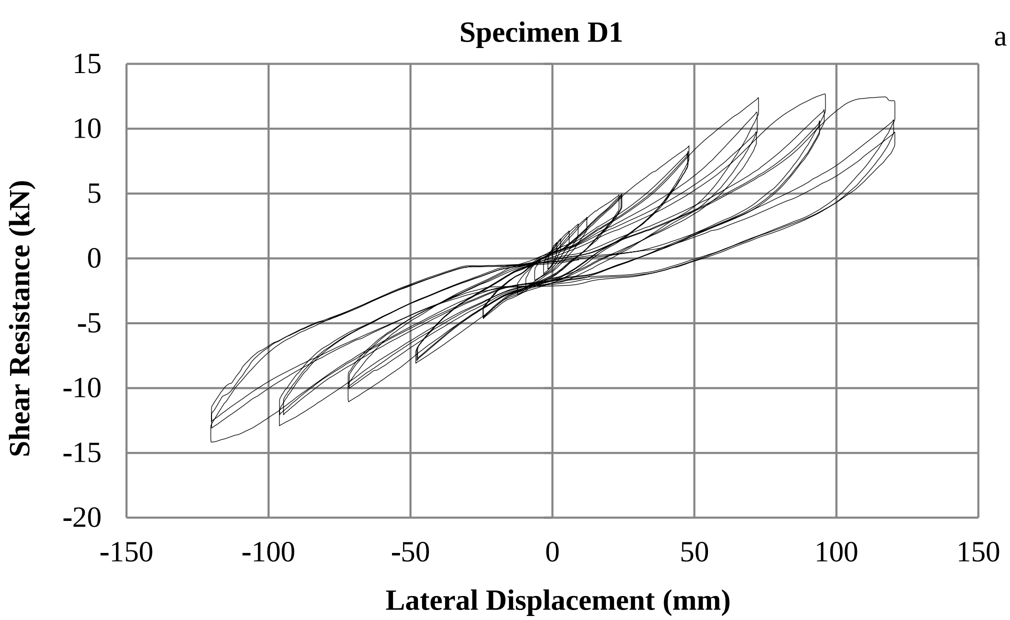


Fig. 7. Hysteretic envelope curves for wall A specimens

Fig. 8. Failure modes of type A wall

In addition to R_d, for estimating the seismic response modification factor (R), over-strength factor (Ω_0) is required, too. Over-strength factor (Ω_0) can be estimated by utilizing the second part of Eq. (2) and assessing V_y and V_s parameters. For evaluating V_s, design capacity of specimens by utilizing the reverse calculations and specifying the mechanism with the smallest failure load is assessed. It should be noted that the mechanism with the smallest failure load during tests was the screw pull-out as observed in the tests (Javaheri Tafti et al., 2014).



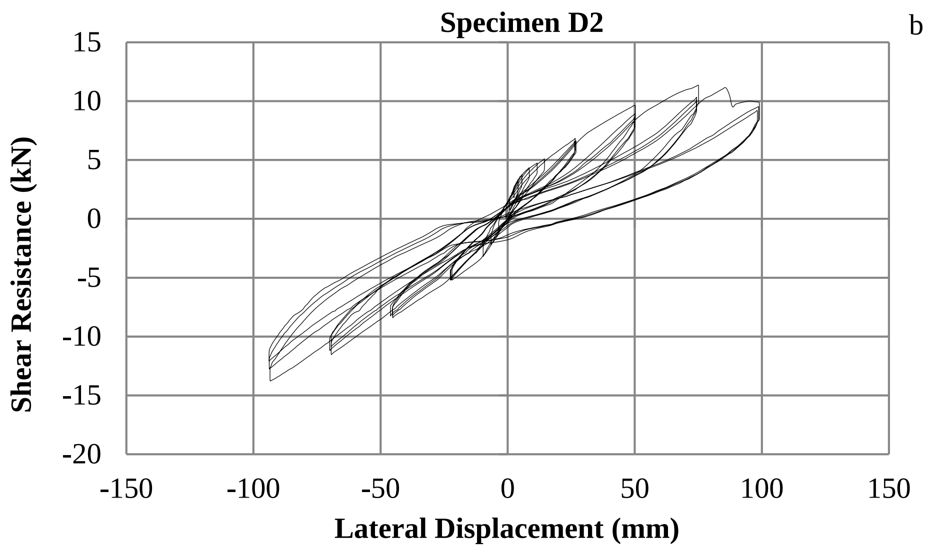


Fig. 9. Hysteretic curves for wall D specimens

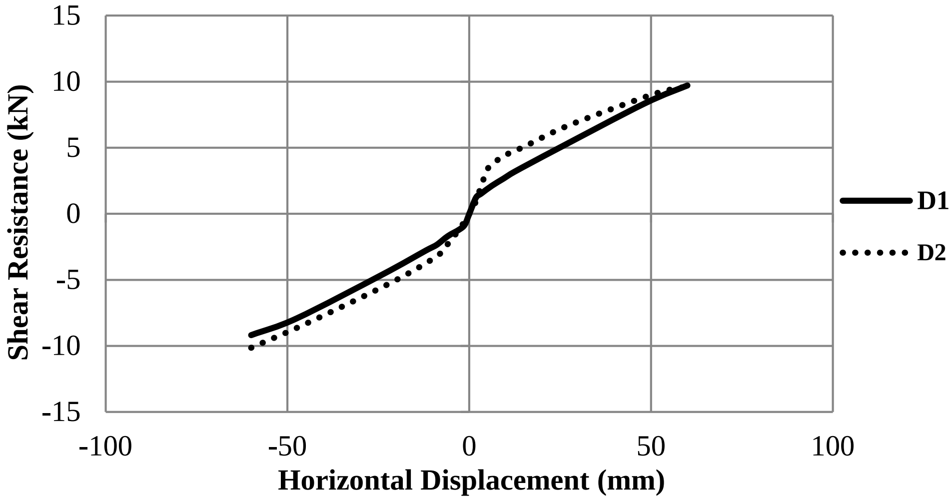


Fig. 10. Hysteretic envelope curves for wall D specimens

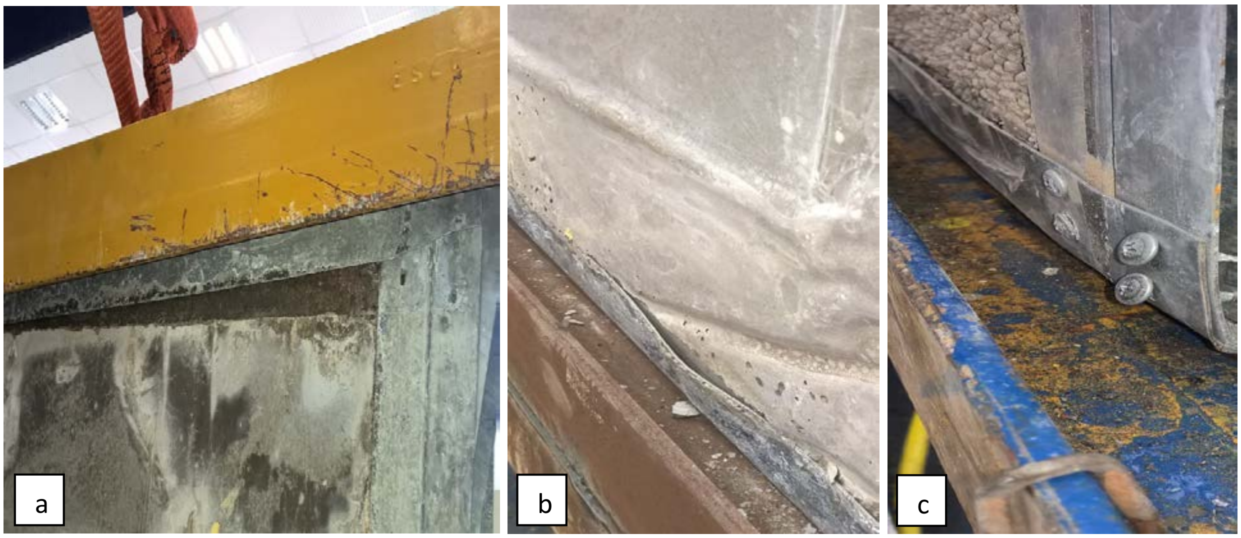


Fig. 11. Hysteretic envelope curves for wall D specimens

After evaluating the required parameters for seismic response modification factors (R) determination of tested specimens, R-values for all tested specimens, using Eq. (3), are then calculated (Javaheri Tafti et al., 2014). R-values with relative parameters for all tested shear walls are presented in Table 5. Based on Table 5 and obtained values, type D shear walls with average response modification factor of 5.5 has higher response modification factor than type A shear walls with the value of 5.2.

One of the most effective parameters for studying the seismic performance of a structure is energy dissipation. Energy dissipation can be calculated by computing the average both positive and negative enclosed area under the load-displacement envelope curve (Lin et al., 2014). It is worth noting that the energy dissipation values in cases under cyclic loads (like present study), are resulted from the average values of positive and negative directions computed from corresponding curve (Lin et al., 2014). Calculated energy dissipations of tested shear walls in this study, are presented in Table 6 at location of 60 mm displacement (which is Δ_t, target displacement). According to Table 6, type D specimens have the most energy dissipating compared with type A specimens, and it can be due to the higher frame width in type D specimens compared with type A specimens.

In addition to seismic parameters mentioned in previous paragraphs, the stiffness of tested shear walls is presented in Table 7. For evaluating the stiffness of tested specimens by using secant stiffness, the method proposed in AISI Manual standard is utilized (AISI, 2003; Lin et al., 2014). Based on this method, the shear stiffness of framed wall can be calculated by using Eq. (4) (AISI, 2003; Lin et al., 2014):

Table 4. The ultimate shear load resistant of types A and D walls

Wall Type	Specimen	Positive		Negative		Ultimate Load (kN)	Average
		Maximum Load (kN)	**Displacement (mm)**	**Maximum Load (kN)**	**Displacement (mm)**		
A	1	7.02	96.53	-5.67	-97.18	7.02	7.27
	2	6.88	96.33	-7.14	-96.66	-7.14	
	3	7.66	119.97	-6.89	-119.48	7.66	
D	1	12.45	117.11	-14.17	-120.16	-14.17	13.97
	2	11.13	88.21	-13.78	-93.42	-13.78	

Table 5. Values of R factor

Wall Type	Specimen Number	R_d	Ω_o	R	Average R
A	1	5.54	1.94	5.6	5.1
	2	3.67	1.2	5.8	
	3	1.36	2.98	4	
D	1	1.26	2.98	3.7	4.65
	2	4.95	1.14	5.6	

Table 6. Values of energy dissipation for tested shear walls

Specimen No.	60 mm (kN-mm)	Average (kN-mm)
A1	148	139
A2	144	
A3	124	
D1	329	354
D2	380	

Table 7. Calculated stiffness for tested shear walls

Shear Wall Type	Specimen No.	*K* (kN/mm)
A	1	0.34
	2	0.32
	3	0.25
D	1	0.5
	2	0.88

$$G' = \left[\frac{P}{\Delta}\right]_{0.4P_n} \times \frac{a}{b} \quad (4)$$

where P and Δ are load and displacement, correspond to the load at $0.4P_n$ (which P_n is the ultimate load). Also $\frac{a}{b}$ is the aspect ratio of tested shear walls.

According to Table 7, type D specimens have higher stiffness than type A specimens.

CONCLUSIONS

Six full-scale cold-formed steel frames filled with lightweight structural concrete were considered, and the responses were studied under cyclic loading regime. Seismic parameters including ductility reduction factor, over-strength factor and seismic response modification factor for these systems were evaluated. In addition, the shear load resistance and failure modes of tested shear walls were studied. Based on the obtained results and discussions in current research, the following conclusions are presented:

- The average R factor in specimens type A is 5.1 and is about 10 percent higher than the average R factor of type D specimens (that is 4.65). This 10 percent decrement can be due to the higher dimension.
- The highest ultimate load capacity is 13.9 kN, which is related to type D shear wall. Type A shear wall with the ultimate load capacity of 7.2 kN has the lowest shear load resistance. It seems that the 300 mm increase in the width of type D wall to type A wall can be the reason of this difference between shear load resistance of type D and type A shear walls.
- Type D shear walls have higher values of energy dissipation, shear strength and stiffness as Compared with type A shear walls. In addition, response modification factor of type A shear walls are more than the same in type D shear walls. These results show that there is no direct relation between energy dissipation, shear strength, stiffness, and response modification factor of these types of lateral load resisting systems.
- Primary failure modes occurred during tests was the same in both types A and D wall specimens. Crippling of the track's web near the corners due to the failure of connections, screw pulling-out in stud to track connection, local buckling of top and bottom track's flange are the primary failure modes that was common in all tested specimens.
- In each two types A and D walls no damages were observed in concrete.

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