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Research Article

Analytical Analysis of Seismic Behavior of Cold-Formed Steel Frames with Strap Brace and Sheathings Plates

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Cold-formed steel frames (CFS) are popular all over the world. In this study, we have investigated 112 frames with different bracing arrangements and different dimensional ratios with different thicknesses of sheathing plates under cyclic and monotonic loading using Finite Element Nonlinear Analysis. We also evaluated seismic parameters including resistance reduction factor, ductility, and force reduction factor due to ductility for all specimens. On the other hand, we calculated the seismic response modification factor for these systems. The maximum modification factor among shear wall panels with sheathing plates related to GWB (gypsum wall board) specimen with thickness of 15 mm was 5.14; among bracing specimens in bilateral bracing mode related to B sample was 3.14. The maximum amount of resistance among the specimens with bilateral (2-side) bracing systems belongs to the specimen C (2-side double X-bracing) with the dimension ratio of 2 (4.8 m × 2.4 m) and resistance of 305.60 kN and also among the shear wall panels with sheathing plates, it belongs to DFP (douglas fir plywood) with a thickness of 20 mm and resistance of 371.34 kN.

1. Introduction

Nowadays, in many countries, the use of LSF system has been developed due to many advantages including high speed, quality, and suitable seismic performance. This system which is made of steel sections with rolled cold members came into the construction industry in 1946 [1] but its usage was limited due to noneconomic feasibility. Since 1990, LSF systems have been developed extensively for many reasons such as the rising price of wood and its limited supply of resources, environmental problems, the need to fast and mass production of housing, and the necessity of using prefabricated buildings. As today, this system has large usage in the long term and medium term construction of commercial and residential buildings in America, Canada, Australia, Japan, and many other countries [2]. One appropriate strategy to improve the seismic performances of these buildings is the use of structural sheathings or bracings. The bracings transmit the horizontal forces from the floor and ceiling levels to foundation.

Zeynalian and Ronagh performed three series of full-scale shear wall tests with aspect ratios of 1 : 1 or 2 : 1 and fibre cement boards sheathed panels. Each series consisted of identical wall panels tested using a cyclic loading regime. They reported that the performance of this kind of CFS lateral resistant systems under cyclic loads is satisfactory and can be considered as a reliable system even in high seismic regions [3]. Scrutinizing the obtained results and comparing the results to other experiments performed by other researchers show that X strap braced system is considered as a ductile system with a satisfactory shear strength; and as such the use of this kind of CFS structure can be preferable particularly in low to medium seismic regions [4–6]. Zeynalian et al. studied the lateral performance of K-braced cold-formed steel structures and their response modification coefficients of R factor. A total of 12 full-scale 2.4 × 2.4 m specimens of different configurations were tested under a standard cyclic loading regime. Of particular interest are the specimens' maximum lateral load capacity and deformation behavior as well as a rational estimation of the seismic response modification factor. The results showed that using washers in the K-elements to studs rivet connections improves the lateral performance of the walls considerably including both strength and ductility as it eliminates pull-out of the rivets;

using double studs for those vertical elements, which are next to K-braced spans, improves neither the ultimate strength nor the R factor. Hence, the use of a K-stud bracing system is possible only in low seismic regions where the earthquake loads and thus the required lateral resistance capacity are not high [7].

Pan and Shan are focused on the experimental study of the structural strength of cold-formed steel wall frames with sheathing under monotonic shear loading. Totally, 13 wall specimens including 5 walls with gypsum board sheathed, 5 walls with calcium silicate board sheathed, 2 walls with oriented-strand board sheathed, and 1 wall frame without sheathing were tested. The ultimate strength, stiffness, and ductility ratios were studied for each test specimen. Based on the test result analysis, the ductility ratio of the specimen with one-side sheathing was greater than that of the specimen with two-side sheathing. Compared to the ultimate strength for the same type of specimens, the wall frame with gypsum board sheathing had the greatest value, the wall frame with calcium silicate board had the secondary value, and the wall frame with gypsum boards had the smallest value. And the ultimate strength of wall frame with sheathing increased as the thickness of board increased [8].

Fülöp and Dubina performed six series of full-scale wall tests with various types of cladding arrangements including X-strap braced frames, corrugated sheathed walls, gypsum board sheathed panels, and oriented strand board (OSB) sheathed panels. Each series consisted of identical wall panels tested using both monotonic and cyclic loading regimes. They found that in most specimens, strengthening the walls' corners is fundamental as the failure starts at the bottom track in the anchor bolt region. Thus, the corner detail should be designed so that the uplift force is directly transmitted from the brace or corner stud to the anchoring bolt, so that it does not induce bending in the bottom track. Also they reported that the seam fastener represented the most sensitive part of the corrugated sheet specimens; damage is gradually increased in seam fasteners, until their failure causes the overall failure of the panel [9].

Kawai et al. conducted a series of full-scale experimental tests on different CFS lateral bracing systems which again included steel sheets. Of particular interest was the in-plane shear resistance of the specimens as well as their ductility. They concluded that while the strap-braced frame was very ductile with remarkable pinching behaviour, the walls with thin steel sheets, plywood, and gypsum board showed less ductility and moderate pinching. They also claimed that the behaviour of walls with a combination of two different lateral bracing systems was reasonably close to the behaviour of the two superimposed ones [10]. In 1998, Elgaaly studied steel shear walls and, due to the very high strain at the end of the corresponding plate, replaced the plate with virtual strips as well as a gusset plate at the ends, and examined the stress and strain in the strip and gusset plate. The computational modeling introduced by him showed a good agreement with the experimentally bolted and welded specimens [11].

In another research, Moghimi and Ronagh introduced new strap-bracing systems that comply with code provisions and satisfy ductility criteria. The program consisted of nine full-scale specimens to evaluate the performance of four different strap-braced walls. The first strapping scheme had four brackets at the four corners of the wall. Strength, stiffness, and ductility of this system depended mostly on the bracket's shape and size and to a lower extent on the chords. The second scheme investigated direct screw connection of straps to the four outermost corners of the wall panel. A similar study was conducted for the connection of straps to the interior frame joints. Finally, investigation was on the lateral performance of a strap-braced wall panel with solid strap connected to gusset plates at four corners [12].

Al-Kharat and Rogers presented an experimental overview of the inelastic performance of sixteen 2.44 m × 2.44 m cold-formed steel strap-braced walls that were not designed following a strict capacity-based design. Using monotonic and reversed cyclic loading protocols, they showed that if capacity design principles are not considered, it is possible for the performance of the walls to be affected by the hold-down detail, which in many cases did not allow the test specimens to reach or maintain a yield capacity and severely diminished the overall system ductility [13].

Gad et al. performed some surveys about the situation of the gypsum board walls on seismic performance of rolled cold steel structural walls braced by crossed straps. The modification factors of the response related to these systems were evaluated by shaking tables and numerical studies trials. Due to these surveys, a wide range of tests, between 4 and 29, were obtained for the modification factor of the response. These results are impractical and misleading and more studies are required to get a reliable response [14, 15].

Yu also presented a research project aimed to evaluate shear strength values for 0.686 mm, 0.762 mm, and 0.838 mm steel sheet sheathed CFS shear walls with aspect ratios of 2 : 1 or 4 : 1. The project consisted of two series of tests in a displacement control mode. The first series was monotonic tests for determining the nominal shear strength for wind loads. The second series was the cyclic tests to obtain the shear strength for seismic loads. The sheathing was only attached to one side of the frame. The test parameters also included three steel sheathing thicknesses and three fastener spacing configurations on the panel edges. Test results indicated that a linear relationship could be assumed between the nominal shear strength and the fastener spacing at panel edges. The buckling of the steel sheathing and pullout of sheathing screws were the primary failure modes for sheet steel CFS shear walls. This project also showed that CFS framed shear walls with large aspect ratios had relatively low stiffness but yielded a significantly large drift capacity [16].

The regulation NEHRP has recommended seismic provisions FEMA 450, FEMA P750, and technical guidelines TI 809-07 and considered the modification factors of the seismic response for some framing systems. In this regulation, the modification factor of the seismic response is considered 4 for braced systems with the diagonal straps, 6.5 for the shear wall, and 3 for other steel systems such as braced order k [17–19]. The standards AISI, as a leading center of treating the performances of rolled cold steel frames, have introduced one performance factor R between 2 and 7 for lateral resistant systems under the base seismic force and have

recommended the use of a performance factor greater than 3 in certain cases [20–22]. The standard AS/NZS 4600-05 has recommended that when rolled cold steel components are used as main seismic resistant elements in structures, the modification factor of the response should not be greater than 2. However, because Australia is located in a region with low seismic hazard, often, the wind is a dominant element in designing short term rolled cold steel buildings. Hence, if the factor R is considered low, it has no effect on designing. Thus, in Australia, there is little attention to some surveys on the evaluation of factor R in rolled cold steel buildings [23].

The laboratorial surveys of the researchers somewhat revealed the performances of shear wall panels and braced frames; however, these costly investigations can be useful when their results are accurately adapted to suitable software analyses. Then, alongside previous research, the effects of these elements on the performances of shear wall panels can be investigated via changing some parameters such as the thickness of the sheathings and the change of distances with lower costs and without laboratory activities as well. Also, after performing the suitable software analysis and comparing laboratorial results with cross bracing samples, we decided to go one step ahead as well as investigating several systems with different bracing configurations and changing distances. The models investigated in this research were examined accurately by the finite element software; however, there is not a general agreement about the value of the response modification factor in rolled cold steel systems and there is no clear and credible source in the regulation for the response modification factor in bracing systems with the configuration K yet. Therefore, to clarify this issue, more studies are required. To clarify this important issue, we decided to model and analyze bracing systems with different configurations such as cross braces, Chevron, and K form and investigate nonlinear responses in these systems by considering different aspect proportions of height to length of the walls in order to remove the uncertainties.

2. Basic Concepts

Investigating the parameters of nonlinear behavior and ductility is of utmost importance. These issues are addressed under the rubric of response modification factor.

2.1. Stiffness and Strength. The position of the predicted strengths, S_{yn} (nominal lateral yield strength), and S_{yp} (predicted lateral yield strength), with respect to S_y (lateral yield strength) may vary from what is illustrated depending on the particular wall being analyzed. The predicted nominal lateral yield strength, S_{yn}, of the wall was based on the tension yield. The strength of the braces was determined using their nominal area (width × thickness) as well as minimum specified (nominal) yield stress. The nominal tension yield capacity of the brace was adjusted for the inclined position of the strap members horizontally. The predicted nominal lateral shear stiffness of the wall, k_n (nominal lateral shear stiffness), was calculated based on the axial stiffness of the two tension brace members. It was also adjusted for their inclined position horizontally. The predicted values S_{yn} and k_n represent the nominal (not factored) design parameters so that an engineer will typically be able to determine that using minimum specified member sizes and material properties without the aid of test results and measurements. S_{yp} is the predicted lateral yield strength of the wall, which is typically reached when the strap braces yield in tension. K_p (predicted lateral shear stiffness) is the predicted lateral shear stiffness of the wall, again obtained from the initial elastic axial stiffness of the strap braces alone. The maximum load level reached by each braced wall regardless of the failure mode was defined as the measured yield strength, S_y. The measured initial elastic shear stiffness K_e (initial elastic shear stiffness) was defined as the secant stiffness from the zero load level to the 40% of maximum load level, $S_{0.4}$, as recommended in ASTM E2126 [24] (Figure 1).

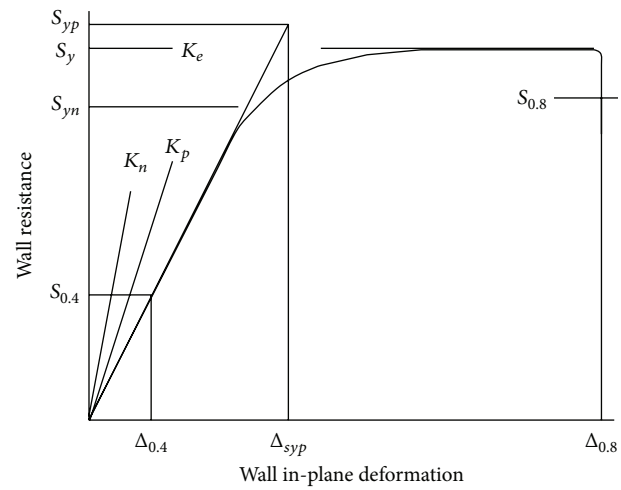

Figure 1: Measured and predicted wall strength and stiffness.

2.2. Ductility. The main factor influencing the behavior factor is the ductility factor. Ductility is, in fact, the ability of bearing nonlinear displacements of the system so that when the system reaches its yielding capacity, it can still withstand forces until it reaches substantially nonlinear lateral displacement without the structure being collapsed. The ductility of the system has been calculated according to the following equation:

$$\mu = \frac{\Delta_{0.8}}{\Delta_{syp}}. \tag{1}$$

In this equation, Δ_{syp} is the elastic yielding ductility calculated by measuring elastic stiffness (K_e) and lateral yielding wall resistance (S_{yp}). $\Delta_{0.8}$ is the rate of displacement failure continuing until there is no resistance (80% of the ultimate resistance is reduced).

2.3. Response Modification Factor. The concept of response modification factor is based on the ductile behavior of the structure to absorb seismic energy as well as the delay in structure failure. In fact, benefiting from the reality that

any structure has a bit additional resistance and ductility, Earthquake Regulations allow designing structures with less power and they pay the fine of this reduction in force by accepting larger displacements. Research has shown that the two factors of additional resistance factor (R_0) and force reduction factor have the greatest impact on the behavior factor at the nonelastic stage due to ductility (R_d). Behavior response modification factor is written as follows:

$$R = R_0 \times R_d. \tag{2}$$

Additional resistance factor (R_0) is the ratio of total yielding limit of structure during the formation of failure mechanism to force corresponding to the formation of the first plastic hinge:

$$R_0 = \frac{S_y}{S_{yn}}. \tag{3}$$

To apply the effect of ductility factor parameter, a factor called force reduction factor due to ductility (R_d) has been introduced and when the fundamental period of the structure is between 0.1 and 0.5 second, Newmark and Hall force equation (1982) is calculated as follows [25]:

$$R_d = \sqrt{2\mu - 1}. \tag{4}$$

3. Modeling

In the present study, a finite element numerical method has been used to model the frames and evaluating the nonlinear response of belt braces has been applied as a lateral bracing system.

3.1. Validation of the Analytical Model with Experimental Model. One way to achieve higher confidence in all numerical modeling techniques is to adapt the numerical results with experimental ones. Hence, due to the similarity of the finite element model with the actual conditions and the possibility of simulating the complexity of potential failure in members, connections, and loading conditions, a reasonable model with the lowest error rate can be achieved. In estimating the monotonic behavior of cold-formed steel frames, LSF frame laboratory specimen with tape cross brace by Al-Kharat and Rorger and the laboratory specimen of shear wall panels done by Fulop and Dubina has been used [9, 13]. Next, using finite element program and software MSC PATRAN-NASTRAN, it is modeled and the results of experimental analysis are compared [26].

3.1.1. The Profiles of Sections and Materials Used in the Model. Test samples were braced walls with crossed straps and shear wall panels with the sheathings. The height and length of the frames were 2.44 meters. Lateral studs made from double C-shaped sections were fused together from the front and the middle studs made from single C-shaped sections were installed with a nominal spacing of 406 mm. Modeling was performed using the same sections. Section Profiles and behavior of materials used in double and single studs, sheeting, tracks, and belts are presented in Tables 1, 2, and 3.

Table 1: Dimension sections and material properties.

Member	Thickness (mm)	Dimensions (mm)	Nominal grade F_y (MPa)
Chord studs	1.91	152 × 41 × 12.7	345
Interior studs	1.22	152 × 41 × 12.7	230
Tracks	1. 91	152 × 31.8	345
Strap bracing	1.91	152	230
Connection plate	1.91	300 × 300	230

Table 2: Material properties of sheeting.

Sheath	Module of elasticity (MPa)	Yield stress F_y (MPa)[5]	Poisson ratio
OSB[1]	9917	3.50	0.30
CSP[2]	7376	3.20	0.25
DFP[3]	10445	3.80	0.30
GWB[4]	1290	2.00	0.20

[1]Oriented strand board.
[2]Canadian softwood plywood.
[3]Douglas fir plywood.
[4]Gypsum wall board.
[5]All sheathings (cover plates) are wooden panels.

A desired failure mode in a system of cold-formed steel wall structures in a ductile behavior is that the entire section surfaces of the braces which play the role of fuse element reach yielding. In this case, all elements and connections in seismic-resistant systems are expected to handle the forces at the capacity of the braces. Braces must be able to enter nonlinear behavior region, scatter the energy resulted from seismic excitations, and continue to desired nonlinear ductility without any failure for the tracks, studs, and their connections. According to this and the fact that belt braces do not tolerate compression loads, for all members of the wall except for belt braces, materials with linear behavior models were defined and for belt braces, materials with nonlinear behavior models (with tensile behavior) were defined.

3.1.2. Components Used in the Analytical Model. The basis of numerical finite element methods, which is used for solving a variety of engineering problems, is the discretization of complex geometric models to easier and smaller elements to facilitate the analysis. A finite element model was formed by applying geometrical characteristics, mesh, material properties, and supporting and loading conditions and the geometry of frame was created by many surfaces of triangular and rectangular elements. For modeling the finite element, standard 4-node elements (CQUAD 4) have been used. To converge the analysis, the number of elements was selected in a way that a proper correspondence is established between the time of analysis and the accuracy of the results. The type of lateral behavior analysis of the walls is a nonlinear pushover analysis. In this analysis, the stress-strain relationship is nonlinear and each point of sentences having the second derivative

Table 3: Matrix of strap-braced wall tests (nominal design dimensions and material properties).

Member	Nominal thickness (mm)	Thickness (mm)	Yield stress F_y (MPa)	Ultimate stress F_u (MPa)	F_u/F_y	Elng. %	F_y/F_{yn}
Chord studs	1.91	1.91	352	489	1.39	35	1.02
Interior studs	1.22	1.23	336	398	1.19	35	1.46
Tracks	1.91	1.94	348	474	1.36	37	1.01
Strap bracing	1.91	1.83	262	346	1.32	38	1.14

of ductility is also considered in strain calculations. So, the possibility of examining the behavior of the structure and ductility was calculated with higher accuracy.

3.1.3. Loading. The cyclic loading regime that has been used in this study is based on Method B of ASTM Standard [24], which was originally developed for ISO (International Organization for Standardization) standard 16670. This loading methodology consists 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, unless failure or a significant decrease in the load resistance occurs earlier. The mentioned lateral amplitudes are corresponding to 1.55%, 3.125%, 6.25%, 9.35%, 12.5%, 25%, 50%, 75%, 100%, 125%, 150%, 175%, 200%, and 225% of the ultimate monotonic lateral displacement of the walls, which was evaluated to be equal to 32 mm. It is worth noting that Method B of ASTM E2126-07 stipulates that the amplitude of cyclic displacements has to be selected based on fractions of monotonic ultimate displacements. If this was applied here, since each specimen had its own ultimate displacement, the loading regime would vary for different specimen types. However, as set out earlier, one of the current research objectives is the comparison of different types of K-braced configurations of the shear walls, which would necessitate using identical cyclic amplitudes for different walls. Hence, Method B is used in this study with a lateral amplitude independent of monotonic testing. Moreover, although 75 mm, or 3.125%, interstorey drift ratio was the maximum amplitude of the actuator, it was considered adequate, since the maximum allowable storey drift ratio specified by the Standard FEMA450 is 2.5% [17]. The average loading velocity was about 2 mm/s, which is compatible with the ASTM E2126-07 recommendation that the loading velocity must be in the range of 1–63 mm/s [24].

3.2. Results Obtained from Evaluating the Validation of the Modeling. Results of the modeling panel of straps and shear walls with finite element software and laboratory works [9, 13] have been presented in Tables 4 and 5. According to the results, a proper corresponding is observed between lateral resistance and drift obtained from numerical and the results of the experimental specimens. The significance level of 5% between results of the numerical and experimental samples causes an increase to confidence in the obtained results in the evaluation.

3.3. Parametric Study. After ensuring the accuracy of analytical model with experimental results, several braced frames with different configurations such as the cross bracing, Chevron, K form, and some shear wall specimens with 4 different sheathing types were modeled in the finite element software. The height of all frames fixed (2.44 m) with variable length was assumed. Each frame included upper and lower fractures, lateral and median parts, and also steel strap braces with wooden sheathings. Sections and materials used in all parts were similar in all specimens. Their properties and specifications are shown in Tables 1, 2, and 3 in previous sections.

Table 4: Comparison of results, experimental and analytical.

Specimen	Analysis results of lateral braced walls (2.44 m × 2.44 m)	
	Experimental [13]	Analytical
Lateral resistance (kN)	103.40	108.17
Difference percent of resistance	4.60%	
Maximum lateral drift (%)	1.70	1.78
Difference percent of drift	4.70%	

Table 5: Comparison of results, experimental and analytical.

Specimen	Analysis results of sheeting walls (3.60 m × 2.44 m)	
	Experimental [9]	Analytical
Lateral resistance (kN)	78.76	74.93
Difference percent of resistance	5%	
Maximum lateral drift (%)	0.175	0.184
Difference percent of drift	5%	

There are considered four different sheathing types to investigate the behavior of the shear walls including OSB, DFP, CSP, and GWB. To consider the effect of the thickness on the seismic performance, every type of these sheathings was modeled with the thicknesses 10, 12.5, 15, 17.5, and 20 mm by considering the one-sided and double-sided sheathings. Since the least thickness of the base for the sheathing DFP is equal to 12.5 mm, there are considered different thicknesses for this kind of the sheathing from 12.5 mm. Also, to observe the effect of the aspect proportions of the shear walls, there were modeled four aspect proportions 0.5, 1, 1.5, and 2 in the constant thickness 12.5 mm of every four sheathing types by considering the one-sided and double-sided sheathings. There were investigated 9 main types of braced frames as shown in Figure 2. With this description, there were studied 68 braced frames and 44 frames with different sheathings,

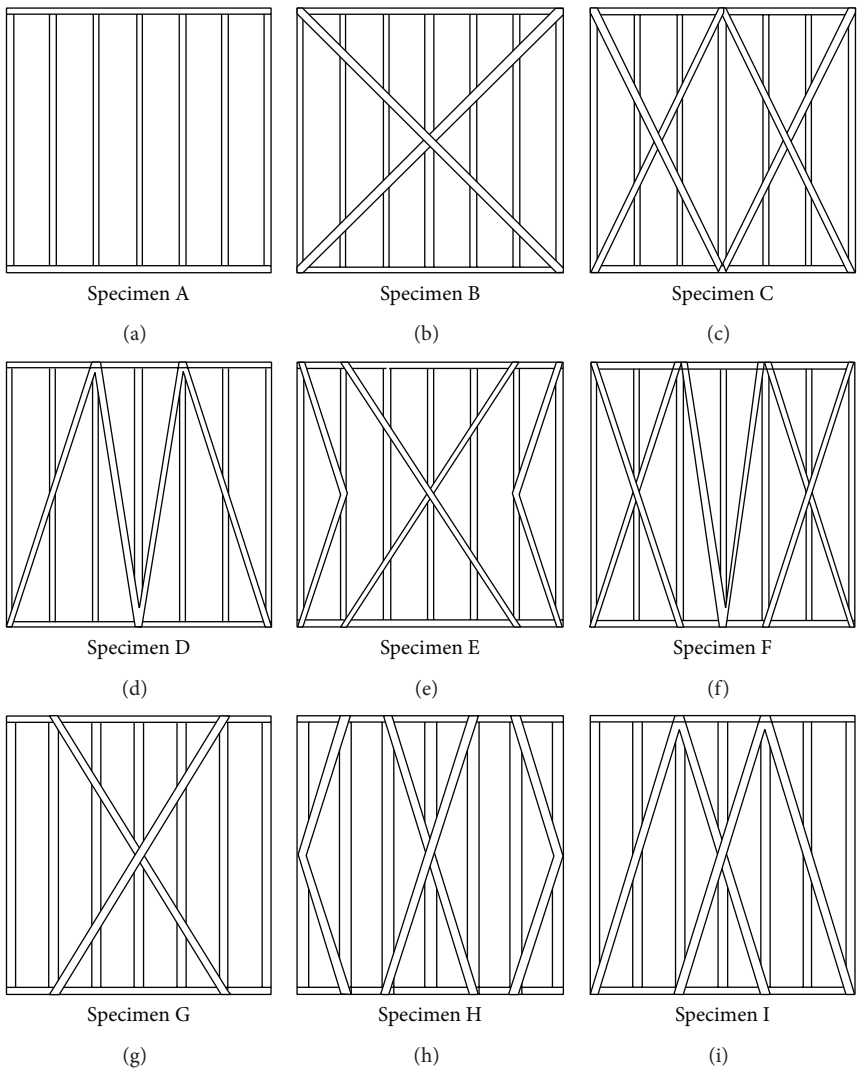

Figure 2: General configuration of specimens.

after modeling in the finite element software and under cyclic and monotonic loading.

4. Discussion on Results

In this part, results of seismic parameters for each model and diagrams from cyclic and monotonic loading of each specimen will be discussed. Values of (R_0) and (R_d) were calculated according to (3) and (4), respectively, and finally, for each specimen, values of response modification factor were calculated using (2).

4.1. Numerical Analysis of Frame of Specimens A. Under monotonic loading, the mean value of yielding capacity for these walls was obtained equal to 6.07 kN, which is equivalent to 78.36% of the value of the predicted yielding capacity. Under cyclic loading, the mean yielding resistance was obtained as 6.29 kN and the ratio of (S_y/S_{yp}) was obtained equal to 79.13%. For monotonic and cyclic tests, the mean value of $\Delta_{0.8}$ was assessed as 119.83 and 124.91 mm, respectively, and the mean value of ductility was equal to 4.39 and 4.97, respectively.

To calculate (R_d) from (3) for wall without brace from a value of mean ductility, equal to 4.67 has been used and the value of R_d was evaluated as 2.89 for walls without brace. Since the value of (S_y/S_{yp}) is less than 1, there is no additional resistance; thus, the value of (R_0) is considered equal to 1. Finally, the value of the response modification factor was obtained as 2.89 (Figure 3).

4.2. Numerical Analysis of Frames of Specimen B. The mean value of yielding capacity under monotonic loading of specimens B with lateral and bilateral braces was obtained as 106.64 and 169.53 kN, respectively, that has been predicted equivalent to 73.85% and 75.50% of capacity values. The mean value of yielding capacity under cyclic loading of specimens B with lateral and bilateral braces was obtained as 114.80 and 183.58 kN, respectively, and also the ratio of (S_y/S_{yp}) was

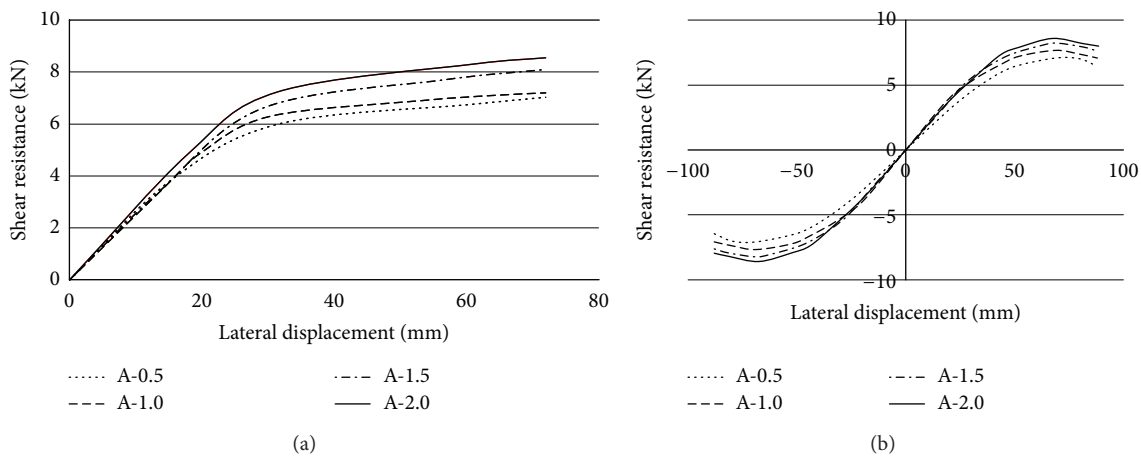

Figure 3: Curves of specimens A. (a) Monotonic curves. (b) Hysteretic envelope curve.

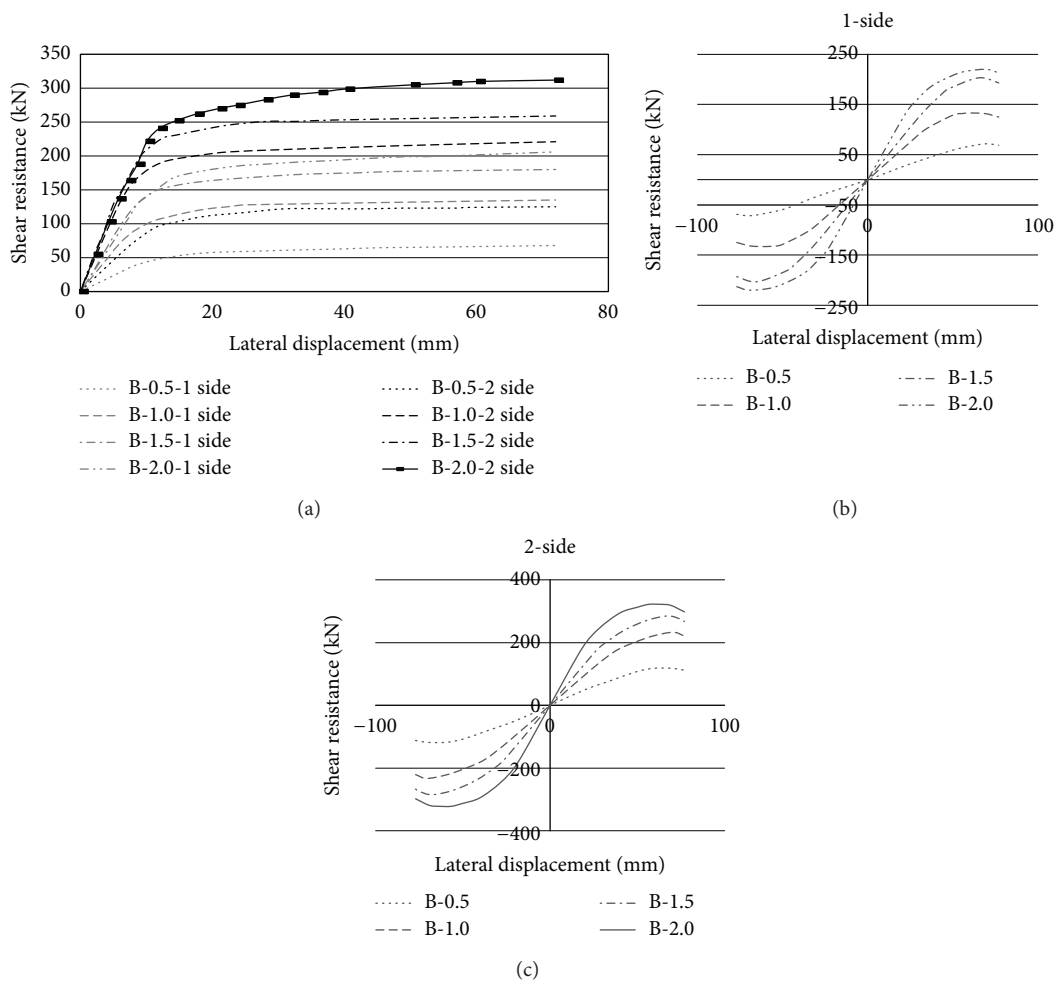

Figure 4: Curves of specimens B. (a) Monotonic curves. (b) Hysteretic envelope curves 1-side. (c) Hysteretic envelope curves 2-side.

obtained as 73.24% and 135.91%, respectively. The value of the yielding capacity of bilateral brace is obtained around 60% more than that of the lateral brace. Sample B with a lateral brace could not get the total of predicted yielding capacity and, at the moment of wall failure, braces did not reach the total of its yielding capacity; however, the status of bilateral brace specimen has somewhat improved compared to that of lateral brace specimen and it could get the total of predicted capacity under cyclic loading.

For the ratio of yielding capacity to the nominal yielding capacity, the mean value was 101.56% and 161.64%, respectively, under monotonic loading of these specimens

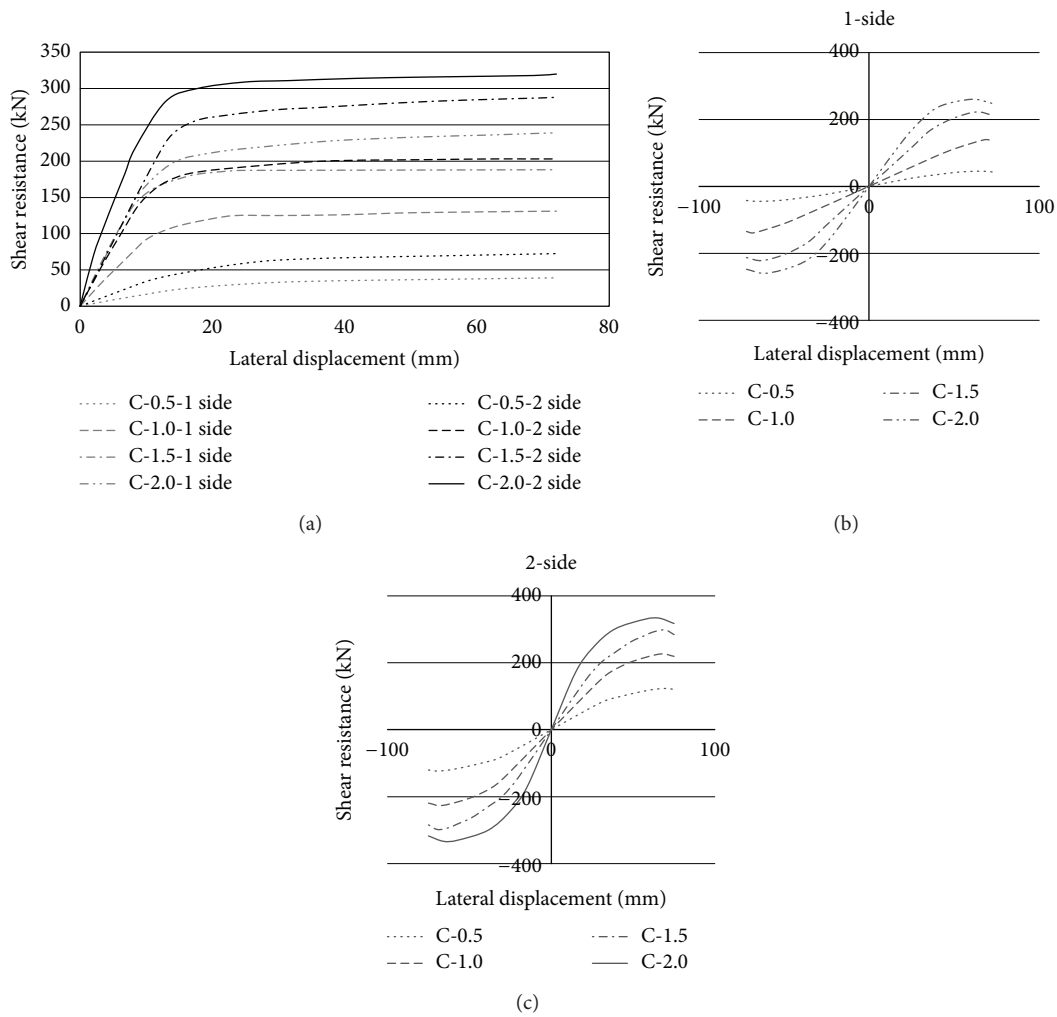

Figure 5: Curves of specimens C. (a) Monotonic curves. (b) Hysteretic envelope curves 1-side. (c) Hysteretic envelope curves 2-side.

with lateral and bilateral braces and 109.33% and 174.84%, respectively, under cyclic loading. These values show that specimens B with lateral and bilateral braces could get the expected nominal design shear resistance well (Figure 4).

The mean value of ($\Delta_{0.8}$) was evaluated as 45.51 and 43.24 mm, respectively, under monotonic loading of specimens B with lateral and bilateral braces, and 44.41 and 42.93 mm, respectively, under cyclic loading. The presence of bilateral brace on a wall causes about 4% reduction on its maximum displacement compared to lateral brace specimens. The mean value of ductility was evaluated as 2.02 and 2.27, respectively, under monotonic loading of samples B with lateral and bilateral braces, and 2.14 and 2.23, respectively, under cyclic loading. To calculate (R_d) from (3) for wall with lateral braces, a mean value of ductility equal to 2.08 has been used and the value of R_d was evaluated as 1.78. The mean value of additional resistance (R_0) was obtained equal to 1.05 for walls with lateral braces and, finally, the value of the response modification factor was obtained as 1.87. To calculate the value of (R_d) for walls with bilateral braces, a mean value of ductility equal to 2.25 has been used and the value of R_d was evaluated equal to 1.87. Based on values of (S_y/S_{yp}), the mean value of additional resistance (R_0) was obtained equal to 1.68 for walls with bilateral braces and, finally, the value of the response modification factor was obtained as 3.14 for specimen B with bilateral braces (Table 9). If the bilateral brace is used for specimen B, the value of the response modification factor will be about 66% more than that of the lateral brace.

The value of predicted stiffness was obtained as 19.62 and 37.56 kN/mm, respectively, for samples B with lateral and bilateral braces under monotonic loading, and 42.95 and 45.69 kN/mm, respectively, under cyclic loading. Obviously, the value of (K_e) is considerably obtained less than (K_p). The shear resistance is increased by increasing the ratio of the height to length of the wall. On the other hand, the shear wall resistance of bilateral braces was significantly obtained more than lateral braces. This has clearly been shown in Figures 4(a)–4(c).

4.3. Numerical Analysis of Frames of Specimen C. The mean value of yielding capacity under monotonic loading of specimens C with lateral and bilateral braces was obtained as

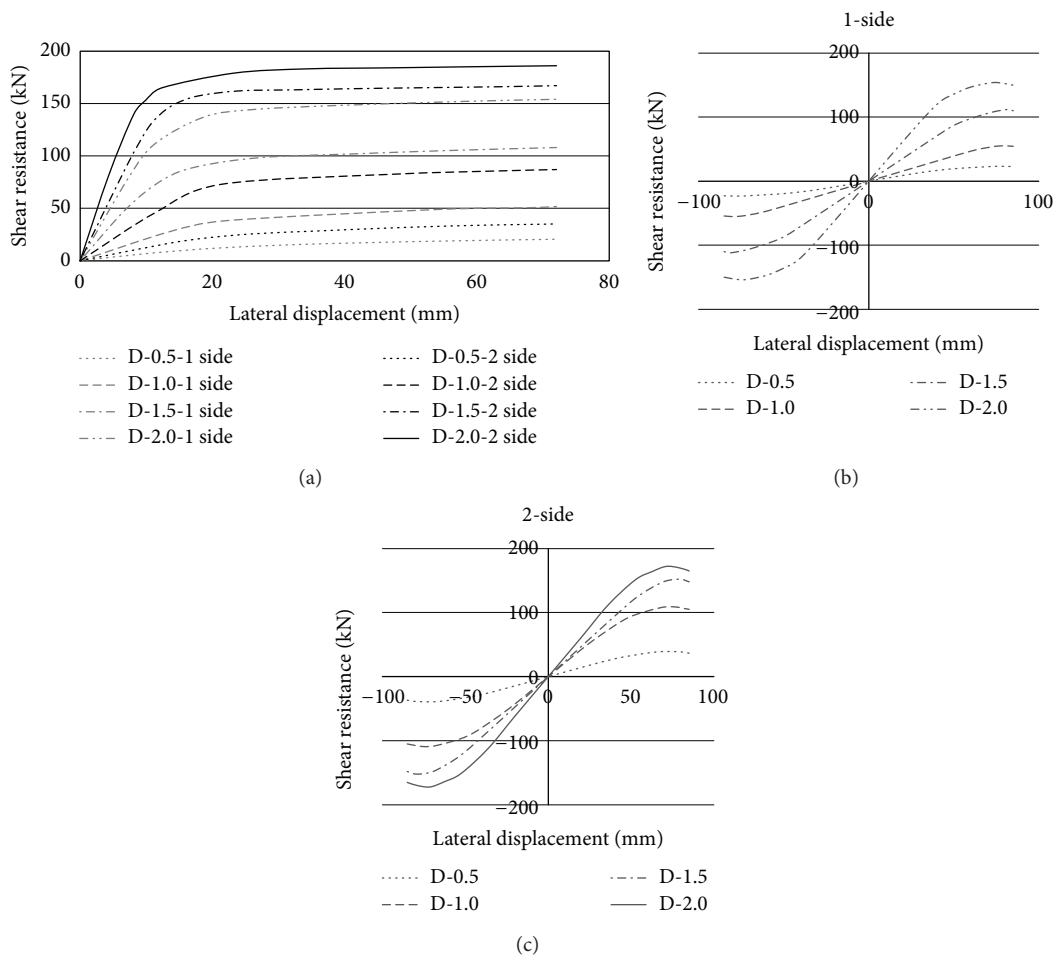

Figure 6: Curves of specimens D. (a) Monotonic curves. (b) Hysteretic envelope curves 1-side. (c) Hysteretic envelope curves 2-side.

118.41 and 180.14 kN, respectively, that has been predicted equivalent to 74.85% and 75.76% of capacity values. The mean value of yielding capacity under cyclic loading of specimens C with lateral and bilateral braces was obtained as 125.35 and 191.59 kN, respectively, and also the ratio of (S_y/S_{yp}) was obtained as 74.22% and 137.16%, respectively. The value of the yielding capacity of bilateral brace is obtained around 52% more than that of the lateral brace. Sample C with a lateral brace could not get the total of predicted yielding capacity and, at the moment of wall failure, braces did not reach the total of their yielding capacity; however, the status of bilateral brace specimen has somewhat improved compared to that of lateral brace specimen and it could get the total of predicted capacity under cyclic loading. For the ratio of yielding capacity to the nominal yielding capacity, the mean value was 112.77% and 171.56%, respectively, under monotonic loading of these specimens with lateral and bilateral braces and 119.38% and 182.47%, respectively, under cyclic loading. These values show that specimens C with lateral and bilateral braces could get the expected nominal design shear resistance well.

The mean value of $(\Delta_{0.8})$ was evaluated as 37.14 and 36.89 mm, respectively, under monotonic loading of specimens C with lateral and bilateral braces, and 35.78 and 34.59 mm, respectively, under cyclic loading. The presence of bilateral brace on wall causes about 2% reduction on its maximum displacement compared to lateral brace specimens. The mean value of ductility was evaluated as 1.72 and 2.03, respectively, under monotonic loading of samples C with lateral and bilateral braces, and 1.82 and 1.89, respectively, under cyclic loading. To calculate (R_d) from (3) for wall with lateral brace a mean value of ductility equal to 1.77 has been used and the value of R_d was evaluated as 1.59. The mean value of additional resistance (R_0) was obtained equal to 1.16 for wall with lateral braces and, finally, the value of the response modification factor was obtained as 1.85. To calculate the value of (R_d) for wall with bilateral braces, a mean value of ductility equal to 1.96 has been used and the value of R_d was evaluated equal to 1.71. Based on values of (S_y/S_{yp}), the mean value of additional resistance (R_0) was obtained equal to 1.77 for walls with bilateral braces and finally, the value of the response modification factor was obtained as 3.02 for specimen C with bilateral brace (Table 9). If the bilateral brace is used for specimen C, the value of the response modification factor will be about 63% more than that of the lateral brace.

The value of predicted stiffness was obtained as 24.71 and 45.36 kN/mm, respectively, for samples C with lateral and bilateral braces under monotonic loading, and 42.46 and 45.18 kN/mm, respectively, under cyclic loading. Obviously,

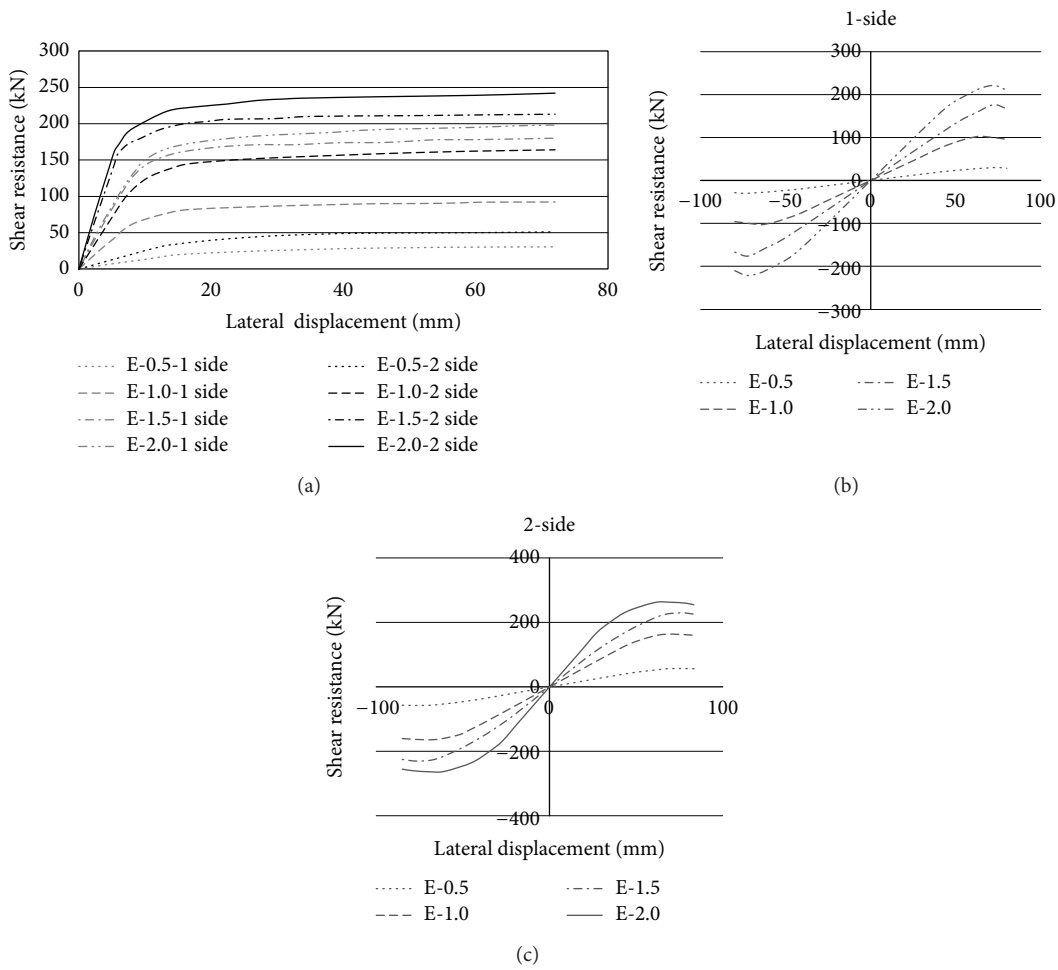

Figure 7: Curves of specimens E. (a) Monotonic curves. (b) Hysteretic envelope curves 1-side. (c) Hysteretic envelope curves 2-side.

the value of (K_e) is considerably obtained less than (K_p). The shear resistance is increased by increasing the ratio of the height to length of the wall. On the other hand, the shear wall resistance of bilateral braces was significantly obtained more than lateral braces. This has clearly been shown in Figures 5(a)–5(c).

4.4. Numerical Analysis of Frames of Specimen D. The mean value of yielding capacity under monotonic loading of specimens D with lateral and bilateral braces was obtained as 61.93 and 87.11 kN, respectively, that has been predicted equivalent to 69.62% and 79.01% of capacity values. The mean value of yielding capacity under cyclic loading of specimens D with lateral and bilateral braces was obtained as 69.13 and 96.02 kN, respectively, and also the ratio of (S_y/S_{yp}) was obtained as 74.27% and 120.70%, respectively. The value of the yielding capacity of bilateral braces is obtained around 40% more than that of the lateral braces. Sample D with a lateral brace could not get the total of predicted yielding capacity and, at the moment of wall failure, braces did not reach the total of their yielding capacity; however, the status of bilateral brace specimen has somewhat improved compared to that of lateral brace specimen and it could get the total of predicted capacity under cyclic loading. For the ratio of yielding capacity to the nominal yielding capacity, the mean value was 58.98% and 82.96%, respectively, under monotonic loading of these specimens with lateral and bilateral braces, and 65.84% and 91.44%, respectively, under cyclic loading. These values show that specimens D with lateral and bilateral braces could not get the expected nominal design shear resistance (Figure 6).

The mean value of ($\Delta_{0.8}$) was evaluated as 73.21 and 55.31 mm, respectively, under monotonic loading of specimens D with lateral and bilateral braces, and 71.49 and 69.11 mm, respectively, under cyclic loading. The presence of bilateral brace on wall causes about 13.8% reduction on its maximum displacement compared to lateral brace specimens. The mean value of ductility was evaluated as 2.88 and 2.97, respectively (Tables 5 and 6), under monotonic loading of samples D with lateral and bilateral braces, and 3.06 and 3.19, respectively (Tables 7 and 8), under cyclic loading. To calculate (R_d) from (3) for walls with lateral braces, a mean value of ductility equal to 2.97 has been used and the value of R_d was evaluated as 2.22. Since the value of (S_y/S_{yp}) is less than 1, there is no additional resistance; thus, the value of (R_0) is considered equal to 1. Finally, the value

Table 6: Monotonic loading specimens of 1-side sheathed CFS results with thickness of 12.5 mm.

Specimen	H/L	S_y kN	S_{yp} kN	K_e kN/mm	$\Delta_{0.8}$ mm	μ	Energy kN·mm	K_p kN/mm	S_y/S_{yp} (%)	K_e/K_p (%)	K_e/K_n (%)
DFP	0.5	209.82	249.42	9.41	78.00	2.49	15377	37.91	84.12	24.82	53.77
	1.0	214.06	255.76	9.78	76.80	2.41	16758	38.26	83.70	25.56	55.89
	1.5	221.71	263.19	10.22	75.46	2.33	17591	39.83	84.24	25.66	58.40
	2.0	228.24	271.35	10.58	75.34	2.27	18299	41.25	84.11	25.65	60.46
Avg		218.46	259.93	10.00	76.40	2.38	17006	39.31			
SD		8.17	9.47	0.510	1.25	0.096	1255	1.54			
Cov		0.037	0.036	0.051	0.016	0.040	0.074	0.039			
OSB	0.5	197.19	238.13	8.84	87.35	2.75	14413	35.37	82.81	24.99	50.51
	1.0	203.94	244.66	9.11	85.74	2.67	15912	36.54	83.36	24.93	52.06
	1.5	209.81	253.91	9.58	83.48	2.51	16752	38.70	82.63	24.75	54.74
	2.0	214.01	259.48	9.83	81.82	2.45	17431	39.64	82.48	24.80	56.17
Avg		206.24	249.05	9.34	84.60	2.60	16127	37.56			
SD		7.31	9.50	0.447	2.43	0.139	1300	1.95			
Cov		0.035	0.038	0.048	0.029	0.054	0.081	0.052			
CSP	0.5	181.87	227.34	8.49	76.14	3.19	13628	32.91	80.00	25.80	48.51
	1.0	198.26	234.21	8.76	102.21	3.08	15084	34.36	84.65	25.49	50.06
	1.5	203.47	245.07	8.92	101.40	2.95	15844	36.02	83.03	24.76	50.97
	2.0	209.25	256.53	9.15	101.82	2.86	16707	37.95	81.57	24.11	52.29
Avg		198.21	240.79	8.83	95.39	3.02	15316	35.31			
SD		11.78	12.78	0.277	12.84	0.145	1306	2.17			
Cov		0.059	0.053	0.031	0.135	0.048	0.085	0.061			
GWB	0.5	163.16	198.33	8.02	123.49	3.70	10649	29.96	82.27	26.77	45.83
	1.0	181.69	224.62	8.19	123.45	3.55	12436	31.30	80.89	26.17	46.80
	1.5	187.53	235.24	8.47	118.16	3.47	13123	32.74	79.72	25.87	48.40
	2.0	194.58	241.03	8.7	118.26	3.39	14297	34.53	80.73	25.20	49.71
Avg		181.74	224.81	8.35	120.84	3.53	12626	32.13			
SD		11.66	18.91	0.125	3.04	0.132	1525	1.96			
Cov		0.064	0.084	0.015	0.025	0.038	0.121	0.061			

of the response modification factor was obtained as 2.97. To calculate the value of (R_d) for walls with bilateral braces, a mean value of ductility equal to 3.08 has been used and the value of R_d was evaluated equal to 2.27. Since the value of (S_y/S_{yp}) is less than 1, there is no additional resistance; thus, the value of (R_0) is considered equal to 1. Finally, the value of the response modification factor was obtained as 2.27 for specimen D with bilateral braces (Table 9). If the bilateral brace is used for specimen D, the value of the response modification factor will be about 2.2% more than that of the lateral brace. This small difference can result from the lack of additional resistance in specimens.

The value of predicted stiffness was obtained as 9.99 and 19.37 kN/mm, respectively, for samples D with lateral and bilateral braces under monotonic loading, and 14.95 and 15.91 kN/mm, respectively, under cyclic loading. Obviously, the value of (K_e) is considerably obtained less than (K_p). The shear resistance is increased by increasing the ratio of the height to length of the wall. On the other hand, the shear wall resistance of bilateral braces was significantly obtained more than lateral braces. This has clearly been shown in Figures 6(a)–6(c).

4.5. Numerical Analysis of Frames of Specimen E. The mean value of yielding capacity under monotonic loading of specimens E with lateral and bilateral braces was obtained as 92.92 and 127.39 kN, respectively, that has been predicted equivalent to 77.71% and 75.91% of capacity values. The mean value of yielding capacity under cyclic loading of specimens E with lateral and bilateral braces was obtained as 97.20 and 135.22 kN, respectively, and also the ratio of (S_y/S_{yp}) was obtained as 72.05% and 109.52%, respectively. The value of the yielding capacity of bilateral braces is obtained around 38% more than that of the lateral braces. Sample E with a lateral brace could not get the total of predicted yielding capacity and, at the moment of wall failure, braces did not reach the total of their yielding capacity; however, the status of bilateral brace specimen has somewhat improved compared to that of lateral brace specimen and it could get the total

Table 7: Monotonic loading specimens of 2-side sheathed CFS results with thickness of 12.5 mm.

Specimen	H/L	S_y kN	S_{yp} kN	K_e kN/mm	$\Delta_{0.8}$ mm	μ	Energy kN·mm	K_p kN/mm	S_y/S_{yp} (%)	K_e/K_p (%)	K_e/K_n (%)
DFP	0.5	237.19	295.66	15.07	36.27	1.78	17588	43.71	80.22	34.48	86.11
	1.0	247.54	307.69	15.32	35.81	1.62	20445	45.26	80.45	33.85	87.54
	1.5	251.65	312.45	15.51	33.66	1.54	22587	48.17	80.54	32.20	88.63
	2.0	258.19	316.87	15.68	31.98	1.47	23914	50.22	81.48	31.22	89.60
Avg		248.64	308.17	15.40	34.68	1.60	21134	46.84			
SD		8.80	9.14	0.262	1.61	0.133	2762	2.92			
Cov		0.035	0.030	0.017	0.046	0.083	0.131	0.062			
OSB	0.5	239.91	271.15	13.19	45.34	1.92	15924	38.81	88.48	33.98	75.37
	1.0	245.51	298.94	13.41	43.27	1.83	18126	41.31	82.13	32.46	76.63
	1.5	250.09	305.72	13.65	41.82	1.76	20735	42.58	81.80	32.06	78.00
	2.0	254.32	309.35	13.32	40.73	1.59	21653	44.02	82.21	30.26	76.11
Avg		247.46	296.29	13.39	42.79	1.78	19110	41.68			
SD		6.19	17.31	0.194	1.99	0.140	2597	2.21			
Cov		0.025	0.058	0.014	0.047	0.079	0.136	0.053			
CSP	0.5	238.44	292.17	12.21	50.48	2.17	14825	35.29	81.61	34.60	69.77
	1.0	244.49	298.43	12.35	49.83	2.04	16982	38.11	81.93	32.41	70.57
	1.5	248.12	303.81	12.69	48.25	1.95	18899	40.92	81.67	31.01	72.51
	2.0	251.87	307.98	12.84	46.29	1.88	20017	43.73	81.78	29.36	73.37
Avg		245.73	300.60	12.52	48.71	2.01	17681	39.51			
SD		5.72	6.84	0.292	1.86	0.125	2279	3.63			
Cov		0.023	0.023	0.023	0.038	0.062	0.129	0.092			
GWB	0.5	201.59	242.42	10.37	67.62	2.39	13039	33.81	83.16	30.67	59.26
	1.0	211.26	254.26	10.51	63.95	2.28	15370	35.43	83.09	29.66	60.06
	1.5	217.98	258.09	10.74	60.67	2.16	17098	37.22	84.46	28.86	61.37
	2.0	225.43	263.87	10.94	57.12	2.04	18873	38.87	85.43	28.16	62.54
Avg		214.07	254.66	10.64	62.34	2.22	16095	36.33			
SD		8.65	9.07	0.274	4.49	0.151	2489	2.19			
Cov		0.040	0.036	0.026	0.072	0.068	0.155	0.060			

of predicted capacity under cyclic loading. For the ratio of yielding capacity to the nominal yielding capacity, the mean value was 88.50% and 121.33%, respectively, under monotonic loading of these specimens with lateral and bilateral braces, and 92.57% and 128.78%, respectively, under cyclic loading. These values show that specimens E with bilateral braces could get the expected nominal design shear resistance well (Figure 7).

The mean value of ($\Delta_{0.8}$) was evaluated as 52.18 and 52.88 mm, respectively, under monotonic loading of specimens E with lateral and bilateral braces, and 51 and 49.30 mm, respectively, under cyclic loading. The mean value of ductility was evaluated as 2.31 and 2.48, respectively, under monotonic loading of samples E with lateral and bilateral braces, and 2.41 and 2.51, respectively, under cyclic loading. To calculate (R_d) from (3) for walls with lateral braces, a mean value of ductility equal to 2.36 has been used and the value of R_d was evaluated as 1.93. Since the value of (S_y/S_{yp}) is less than 1, there is no additional resistance; thus, the value of (R_0) is considered equal to 1. Finally, the value of the response modification factor was obtained as 1.93. To calculate the value of (R_d) for walls with bilateral braces, a mean value of ductility equal to 2.50 has been used and the value of R_d was evaluated equal to 2. Based on values of (S_y/S_{yp}), the mean value of additional resistance (R_0) was obtained equal to 1.25 for wall with bilateral brace and, finally, the value of the response modification factor was obtained as 2.50 for specimen E with bilateral brace (Table 9). If the bilateral brace is used for specimen E, the value of the response modification factor will be about 29.50% more than that of lateral brace.

The value of predicted stiffness was obtained as 15.51 and 21.89 kN/mm, respectively, for samples E with lateral and bilateral braces under monotonic loading, and 30.58 and 32.53 kN/mm, respectively, under cyclic loading. Obviously, the value of (K_e) is considerably obtained less than (K_p). The shear resistance is increased by increasing the ratio of the height to length of the wall. On the other hand, the shear wall resistance of bilateral braces was significantly obtained more than that of lateral braces. This has clearly been shown in Figures 7(a)–7(c).

Table 8: The evaluated seismic parameters of sheeting walls ($H/L = 1$).

Specimen	Thickness (mm)	1-side				2-side			
		μ	R_d	R_o	R	μ	R_d	R_o	R
DFP	12.5	2.41	1.95	2.04	3.99	1.62	1.50	2.36	3.53
	15	2.06	1.77	2.24	3.96	1.54	1.44	2.65	3.82
	17.5	1.83	1.63	2.42	3.95	1.42	1.36	2.82	3.83
	20	1.49	1.41	2.60	3.66	1.31	1.27	3.04	3.87
OSB	10	3.06	2.26	1.80	4.07	1.97	1.71	1.97	3.38
	12.5	2.67	2.08	1.94	4.04	1.83	1.63	2.24	3.65
	15	2.38	1.94	2.18	4.23	1.72	1.56	2.56	4.00
	17.5	2.08	1.78	2.34	4.16	1.61	1.49	2.74	4.08
	20	1.70	1.55	2.53	3.92	1.49	1.41	2.92	4.11
CSP	10	3.52	2.46	1.68	4.13	2.18	1.83	1.92	3.52
	12.5	3.08	2.27	1.89	4.29	2.04	1.75	2.33	4.09
	15	2.64	2.07	2.09	4.32	1.86	1.65	2.44	4.02
	17.5	2.31	1.90	2.24	4.26	1.79	1.61	2.66	4.27
	20	1.88	1.66	2.45	4.07	1.65	1.52	2.78	4.22
GWB	10	4.06	2.67	1.54	4.11	2.57	2.03	1.68	3.42
	12.5	3.55	2.47	1.73	4.27	2.28	1.89	2.01	3.79
	15	2.86	2.17	2.02	4.39	2.09	1.78	2.88	5.14
	17.5	2.48	1.99	2.16	4.30	1.92	1.69	2.51	4.23
	20	2.02	1.74	2.30	4.01	1.77	1.59	2.65	4.22

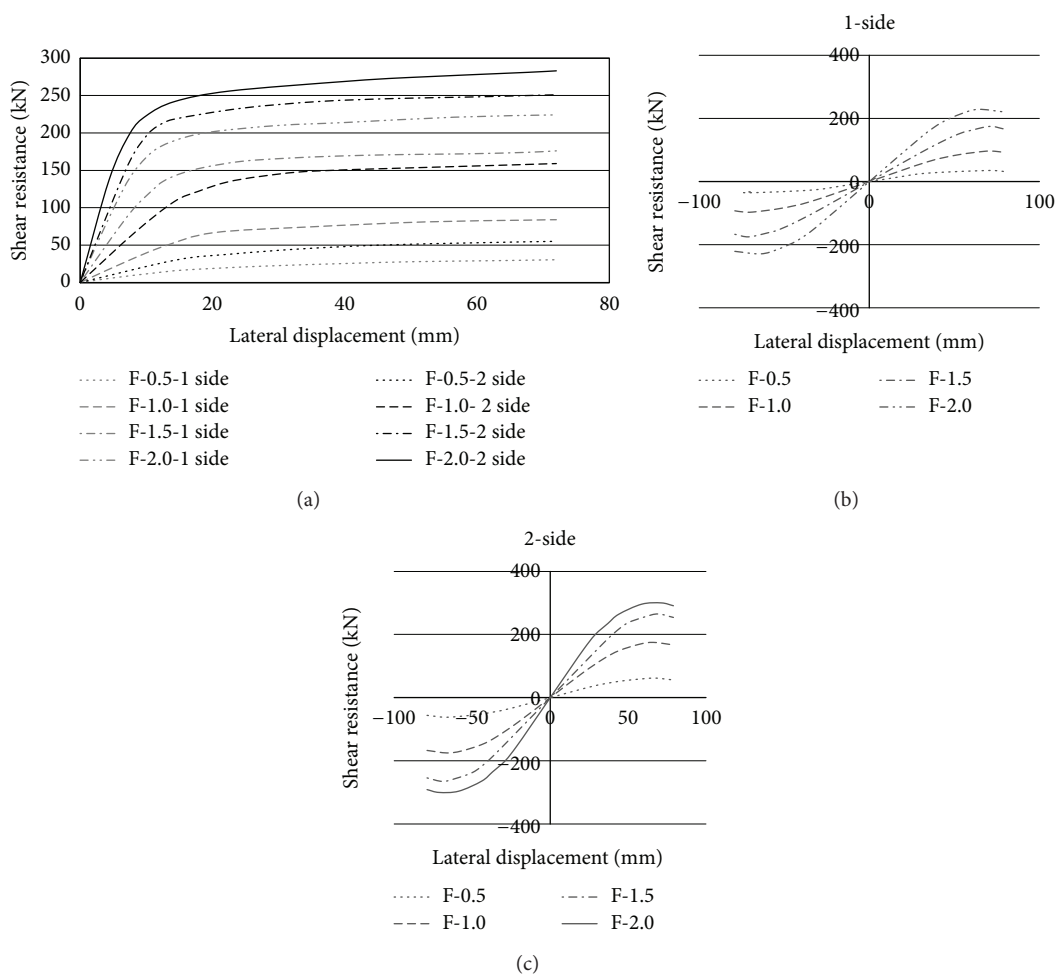

Figure 8: Curves of specimens F. (a) Monotonic curves. (b) Hysteretic envelope curves 1-side. (c) Hysteretic envelope curves 2-side.

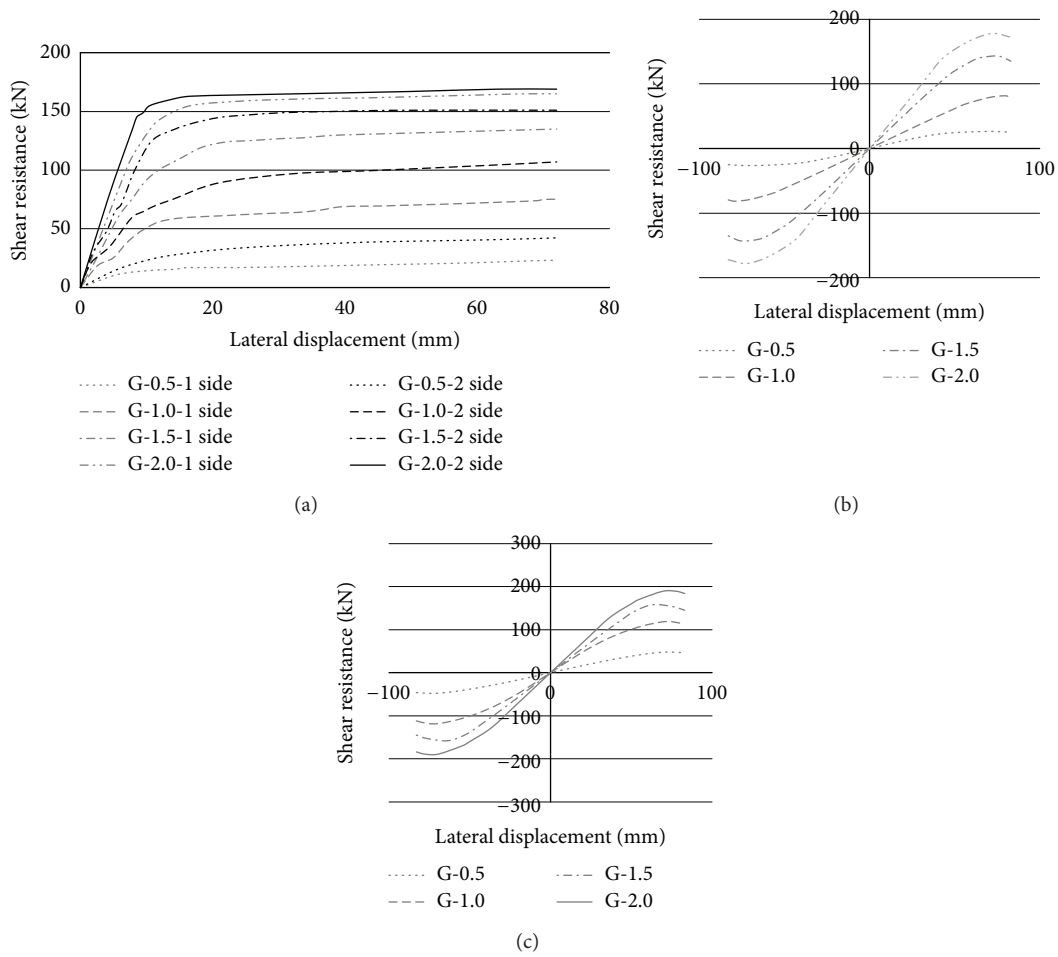

Figure 9: Curves of specimens G. (a) Monotonic curves. (b) Hysteretic envelope curves 1-side. (c) Hysteretic envelope curves 2-side.

Table 9: The evaluated seismic parameters of braced walls.

Specimen	1-side				2-side			
	μ	R_d	R_o	R	μ	R_d	R_o	R
B	2.08	1.78	1.05	1.87	2.25	1.87	1.68	3.14
C	1.77	1.59	1.16	1.85	1.96	1.71	1.77	3.02
D	2.97	2.22	1.00	2.22	3.08	2.27	1.00	2.27
E	2.36	1.93	1.00	1.93	2.50	2.00	1.25	2.50
F	2.30	1.89	1.00	1.89	2.42	1.96	1.45	2.84
G	2.74	2.13	1.00	2.13	2.85	2.17	1.00	2.17
H	3.26	2.35	1.00	2.35	3.36	2.39	1.00	2.39
I	2.52	2.01	1.00	2.01	2.62	2.06	1.07	2.20

4.6. Numerical Analysis of Frames of Specimen F. The mean value of yielding capacity under monotonic loading of specimens F with lateral and bilateral braces was obtained as 95.90 and 146.77 kN, respectively, that has been predicted equivalent to 76.58% and 77.10% of capacity values. The mean value of yielding capacity under cyclic loading of specimens F with lateral and bilateral braces was obtained as 107.17 and 157.76 kN, respectively, and also the ratio of (S_y/S_{yp}) was obtained as 76.52% and 122.78%, respectively. The value of the yielding capacity of bilateral braces is obtained around 50% more than that of the lateral braces. Sample F with a lateral brace could not get the total of predicted yielding capacity and, at the moment of wall failure, braces did not reach the total of their yielding capacity; however, the status of bilateral brace specimen has somewhat improved compared to that of lateral brace specimen and it could get the total of predicted capacity under cyclic loading. For the ratio of yielding capacity to the nominal yielding capacity, the mean value was 91.33% and 139.78%, respectively, under monotonic loading of these specimens with lateral and bilateral braces, and 102.07% and 150.25%, respectively, under cyclic loading. These values show that specimens F with bilateral braces could get the expected nominal design shear resistance well.

The mean value of $(\Delta_{0.8})$ was evaluated as 49.27 and 45.60 mm, respectively, under monotonic loading of specimens F with lateral and bilateral braces, and 48.08 and 46.48 mm, respectively, under cyclic loading. The presence of bilateral brace on wall causes about 5.4% reduction on its maximum displacement compared to lateral brace specimens. The mean value of ductility was evaluated as 2.23 and 2.37, respectively, under monotonic loading of samples F with lateral and bilateral braces, and 2.36 and 2.46, respectively, under cyclic loading. To calculate (R_d) from (3) for walls with

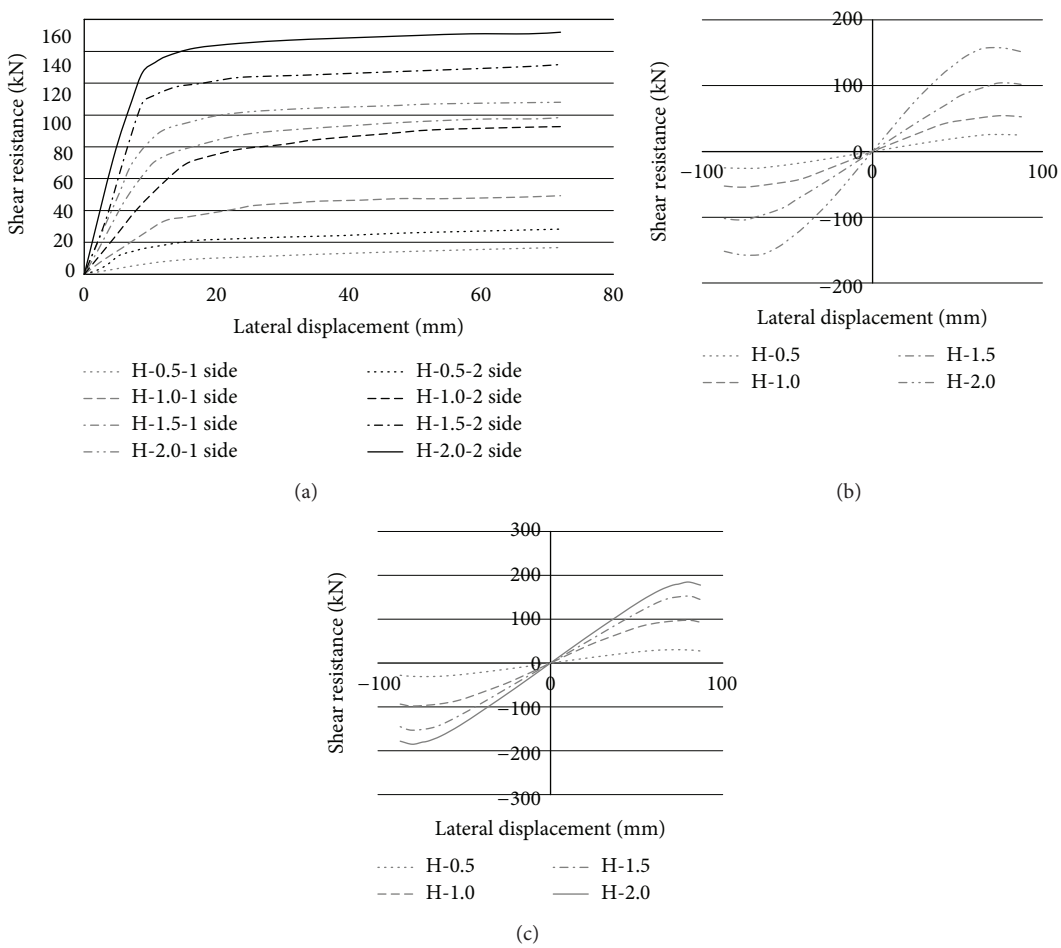

Figure 10: Curves for specimens H. (a) Monotonic curves. (b) Hysteretic envelope curves 1-side. (c) Hysteretic envelope curves 2-side.

lateral braces, a mean value of ductility equal to 2.30 has been used and the value of R_d was evaluated as 1.89. Since the value of (S_y/S_{yp}) is less than 1, there is no additional resistance; thus, the value of (R_0) is considered equal to 1. Finally, the value of the response modification factor was obtained as 1.89. To calculate the value of (R_d) for wall with bilateral braces, a mean value of ductility equal to 2.42 has been used and the value of R_d was evaluated equal to 1.96. Based on values of (S_y/S_{yp}), the mean value of additional resistance (R_0) was obtained equal to 1.45 for walls with bilateral braces and, finally, the value of the response modification factor was obtained as 2.84 for specimen F with bilateral braces (Table 9). If the bilateral brace is used for specimen F, the value of the response modification factor will be about 50% more than that of the lateral brace.

The value of predicted stiffness was obtained as 17.86 and 31.95 kN/mm, respectively, for samples F with lateral and bilateral braces under monotonic loading, and 34.53 and 36.73 kN/mm, respectively, under cyclic loading. Obviously, the value of (K_e) is considerably obtained less than (K_p). The shear resistance is increased by increasing the ratio of the height to length of the wall. On the other hand, the shear wall resistance of bilateral braces was significantly obtained more than that of the lateral braces. This has clearly been shown in Figures 8(a)–8(c).

4.7. Numerical Analysis of Frames of Specimen G. The mean value of yielding capacity under monotonic loading of specimens G with lateral and bilateral braces was obtained as 83.23 and 95.82 kN, respectively, that has been predicted equivalent to 77% and 77.10% of capacity values. The mean value of yielding capacity under cyclic loading of specimens G with lateral and bilateral braces was obtained as 87.17 and 104.73 kN, respectively, and also the ratio of (S_y/S_{yp}) was obtained as 73.86% and 100.75%, respectively. The value of the yielding capacity of bilateral brace is obtained around 18% more than that of the lateral brace. Sample G with a lateral brace could not get the total of predicted yielding capacity and, at the moment of wall failure, braces did not reach the total of their yielding capacity; however, the status of bilateral brace specimen has somewhat improved compared to that of lateral brace specimen and it could get the total of predicted capacity under cyclic loading. For the ratio of yielding capacity to the nominal yielding capacity, the mean value was 79.27% and 91.26%, respectively, under monotonic

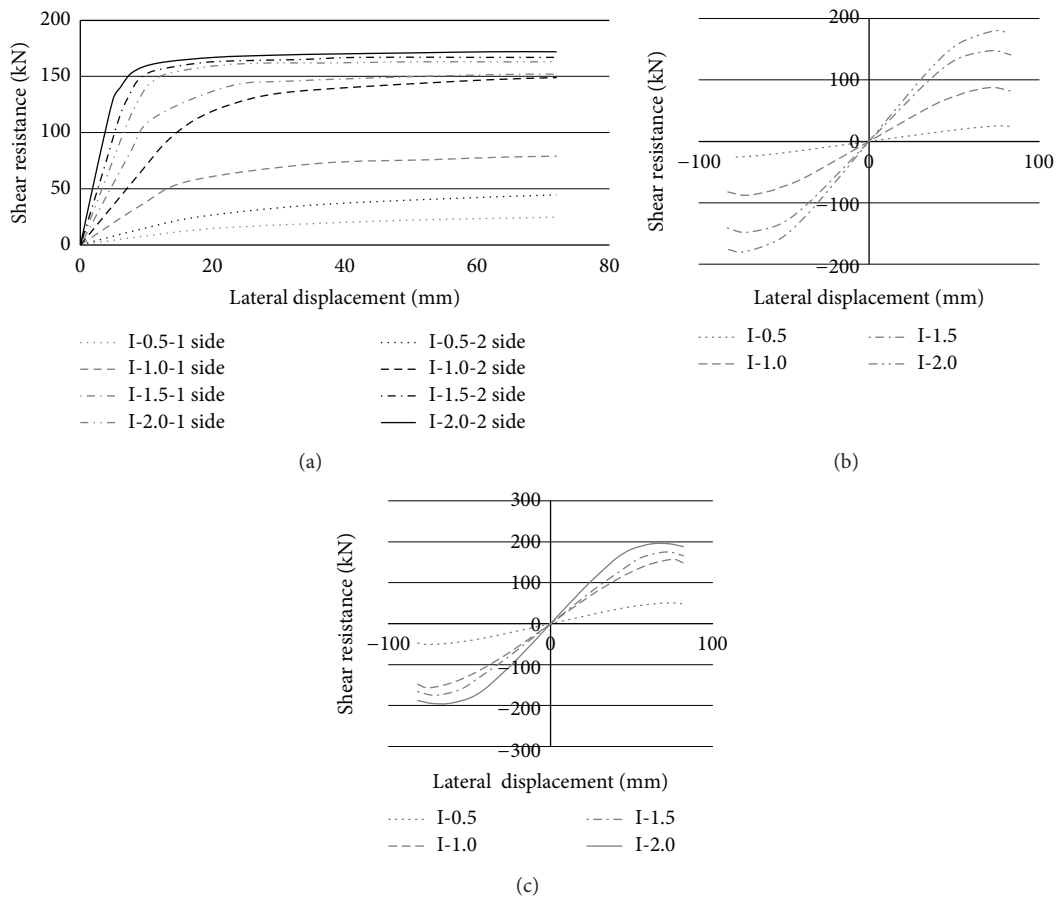

Figure 11: Curves for specimens I. (a) Monotonic curves. (b) Hysteretic envelope curves 1-side. (c) Hysteretic envelope curves 2-side.

loading of these specimens with lateral and bilateral braces, and 83.02% and 99.74%, respectively, under cyclic loading. These values show that specimens G with lateral and bilateral braces could not get the expected nominal design shear resistance.

The mean value of ($\Delta_{0.8}$) was evaluated as 66.34 and 56.54 mm, respectively, under monotonic loading of specimens G with lateral and bilateral braces, and 63.66 and 61.55 mm, respectively, under cyclic loading. The presence of bilateral brace on wall causes about 9% reduction on its maximum displacement compared to lateral brace specimens. The mean value of ductility was evaluated as 2.70 and 2.75, respectively, under monotonic loading of samples G with lateral and bilateral braces, and 2.82 and 2.94, respectively, under cyclic loading. To calculate (R_d) from (3) for walls with lateral braces, a mean value of ductility equal to 2.76 has been used and the value of R_d was evaluated as 2.13. Since the value of (S_y/S_{yp}) is less than 1, there is no additional resistance; thus, the value of (R_0) is considered equal to 1. Finally, the value of the response modification factor was obtained as 2.13. To calculate the value of (R_d) for walls with bilateral braces, a mean value of ductility equal to 2.85 has been used and the value of R_d was evaluated equal to 2.17. Since the value of (S_y/S_{yp}) is less than 1, there is no additional resistance; thus, the value of (R_0) is considered equal to 1. Finally, the value of the response modification factor was obtained as 2.17 for specimen G with bilateral braces (Table 9). If the bilateral brace is used for specimen G, the value of the response modification factor will be about 1.87% more than that of the lateral brace. This small difference can result from the lack of additional resistance in specimens.

The value of predicted stiffness was obtained as 13.18 and 19.06 kN/mm, respectively, for samples G with lateral and bilateral braces under monotonic loading, and 18.30 and 19.47 kN/mm, respectively, under cyclic loading. Obviously, the value of (K_e) is considerably obtained less than (K_p). The shear resistance is increased by increasing the ratio of the height to length of the wall. On the other hand, the shear wall resistance of bilateral braces was significantly obtained more than lateral braces. This has clearly been shown in Figures 9(a)–9(c).

4.8. Numerical Analysis of Frames of Specimen H. The mean value of yielding capacity under monotonic loading of specimens H with lateral and bilateral braces was obtained as 60.49 and 83.24 kN, respectively, that has been predicted equivalent to 74.93% and 79.30% of capacity values. The mean value of yielding capacity under cyclic loading of specimens H with lateral and bilateral braces was obtained as 64.40 and 89.38 kN, respectively, and also the ratio of (S_y/S_{yp}) was

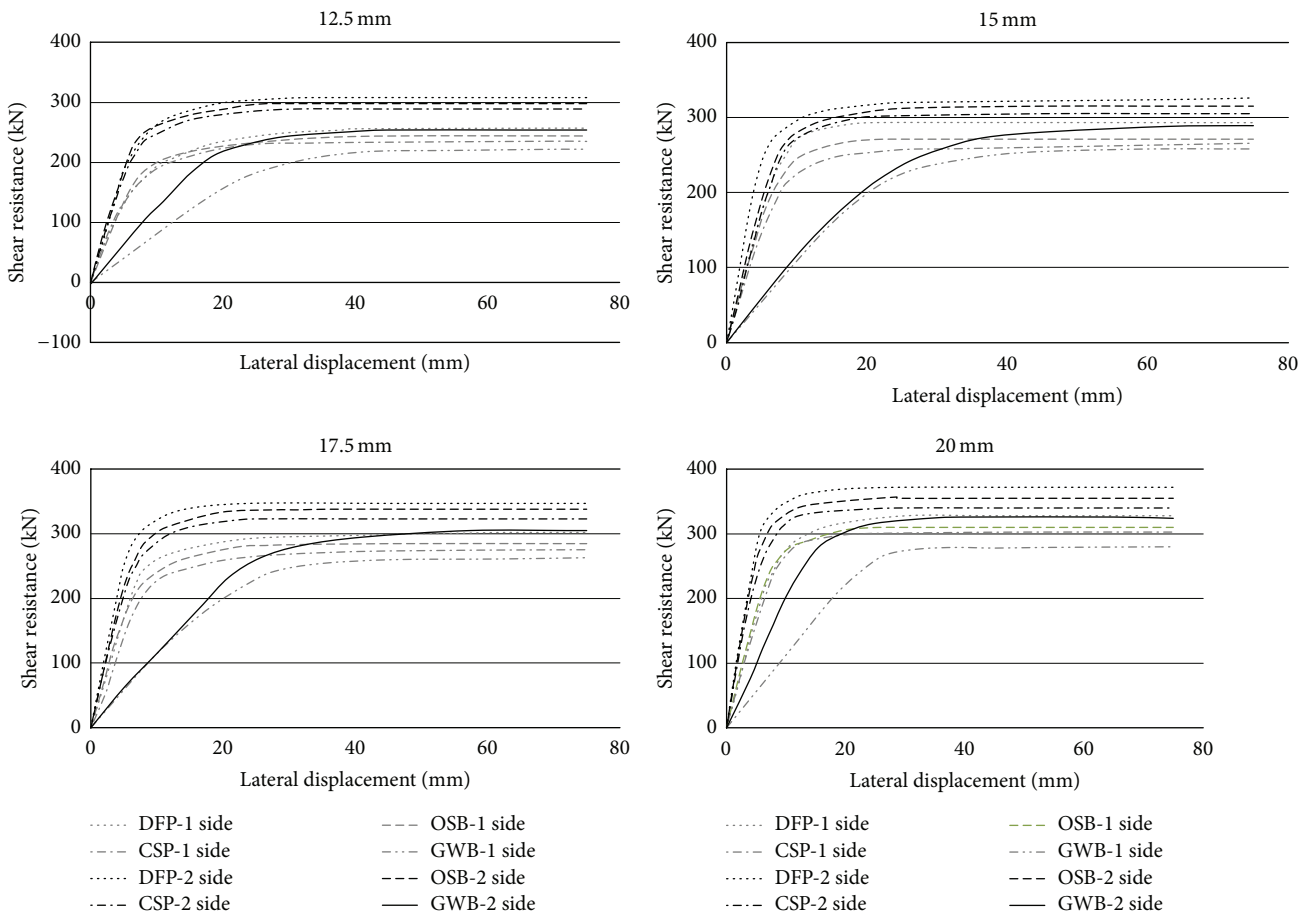

Figure 12: Monotonic curves of sheeting walls with variable of the sheets and thicknesses.

obtained as 71.90% and 114.73%, respectively. The value of the yielding capacity of bilateral braces is obtained around 38% more than that of the lateral braces. Sample H with a lateral brace could not get the total of predicted yielding capacity and, at the moment of wall failure, braces did not reach the total of their yielding capacity; however, the status of bilateral brace specimen has somewhat improved compared to that of lateral brace specimen and it could get the total of predicted capacity under cyclic loading. For the ratio of yielding capacity to the nominal yielding capacity, the mean value was 57.60% and 79.28%, respectively, under monotonic loading of these specimens with lateral and bilateral braces, and 61.33% and 85.12%, respectively, under cyclic loading. These values show that specimens H with lateral and bilateral braces could not get the expected nominal design shear resistance (Figure 10).

The mean value of ($\Delta_{0.8}$) was evaluated as 80.90 and 73.89 mm, respectively, under monotonic loading of specimens H with lateral and bilateral braces, and 78.90 and 76.27 mm, respectively, under cyclic loading. The presence of bilateral brace on wall causes about 6% reduction on its maximum displacement compared to lateral brace specimens. The mean value of ductility was evaluated as 3.16 and 3.22, respectively, under monotonic loading of samples H with lateral and bilateral braces, and 3.35 and 3.49, respectively, under cyclic loading. To calculate (R_d) from (3) for walls with lateral braces, a mean value of ductility equal to 3.26 has been used and the value of R_d was evaluated as 2.35. Since the value of (S_y/S_{yp}) is less than 1, there is no additional resistance; thus, the value of (R_0) is considered equal to 1. Finally, the value of the response modification factor was obtained as 2.35. To calculate the value of (R_d) for walls with bilateral braces, a mean value of ductility equal to 3.36 has been used and the value of R_d was evaluated equal to 2.39. Since the value of (S_y/S_{yp}) is less than 1, there is no additional resistance; thus, the value of (R_0) is considered equal to 1. Finally, the value of the response modification factor was obtained as 2.39 for specimen H with bilateral braces (Table 9). If the bilateral brace is used for specimen H, the value of the response modification factor will be about 1.70% more than that of the lateral brace. This small difference can result from the lack of additional resistance in specimens.

The value of predicted stiffness was obtained as 8.30 and 16.25 kN/mm, respectively, for samples H with lateral and bilateral braces under monotonic loading, and 11.03 and 11.74 kN/mm, respectively, under cyclic loading. Obviously, the value of (K_e) is considerably obtained less than (K_p). The shear resistance is increased by increasing the ratio of the height to length of the wall. On the other hand, the shear wall resistance of bilateral braces was significantly obtained

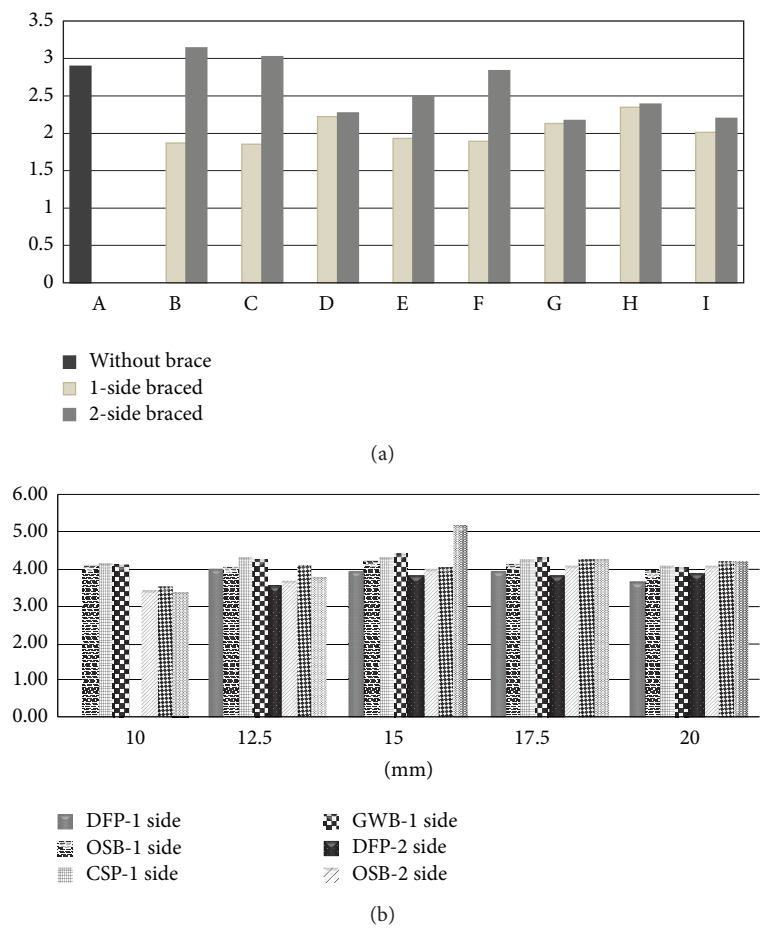

Figure 13: Comparing values of response modification factors. (a) Braced walls. (b) Sheeting walls.

more than that of lateral braces. This has clearly been shown in Figures 10(a)–10(c).

4.9. Numerical Analysis of Frames of Specimen I. The mean value of yielding capacity under monotonic loading of specimens I with lateral and bilateral braces was obtained as 86.21 and 108.61 kN, respectively, that has been predicted equivalent to 74.73% and 78.78% of capacity values. The mean value of yielding capacity under cyclic loading of specimens I with lateral and bilateral braces was obtained as 91.87 and 116.92 kN, respectively, and also the ratio of (S_y/S_{yp}) was obtained as 74.67% and 112.10%, respectively. The value of the yielding capacity of bilateral braces is obtained around 26% more than that of the lateral braces. Sample I with a lateral brace could not get the total of predicted yielding capacity and, at the moment of wall failure, braces did not reach the total of their yielding capacity; however, the status of bilateral brace specimen has somewhat improved compared to that of lateral brace specimen and it could get the total of predicted capacity under cyclic loading. For the ratio of yielding capacity to the nominal yielding capacity, the mean value was 82.11% and 103.44%, respectively, under monotonic loading of these specimens with lateral and bilateral braces, and 87.49% and 11.35%, respectively, under cyclic loading. These values show that specimens I with bilateral braces could get the expected nominal design shear resistance well (Figure 11).

The mean value of $(\Delta_{0.8})$ was evaluated as 57.75 and 53.53 mm, respectively, under monotonic loading of samples I with lateral and bilateral braces, and 56.42 and 54.55 mm, respectively, under cyclic loading. The presence of bilateral brace on wall causes about 5.3% reduction on its maximum displacement compared to lateral braces' samples. The mean value of ductility was evaluated as 2.23 and 2.37, respectively, under monotonic loading of samples I with lateral and bilateral braces, and 2.36 and 2.46, respectively, under cyclic loading. To calculate (R_d) from (3) for walls with lateral braces, a mean value of ductility equal to 2.52 has been used and the value of R_d was evaluated as 2.01. Since the value of (S_y/S_{yp}) is less than 1, there is no additional resistance; thus, the value of (R_0) is considered equal to 1. Finally, the value of the response modification factor was obtained as 2.01. To calculate the value of (R_d) for walls with bilateral braces, a mean value of ductility equal to 2.62 has been used and the value of R_d was evaluated equal to 2.06. Based on values of (S_y/S_{yp}), the mean value of additional resistance (R_0) was obtained equal to 1.07 for walls with bilateral braces and, finally, the value of the response modification factor was obtained as 2.20 for sample I with bilateral braces (Table 9).

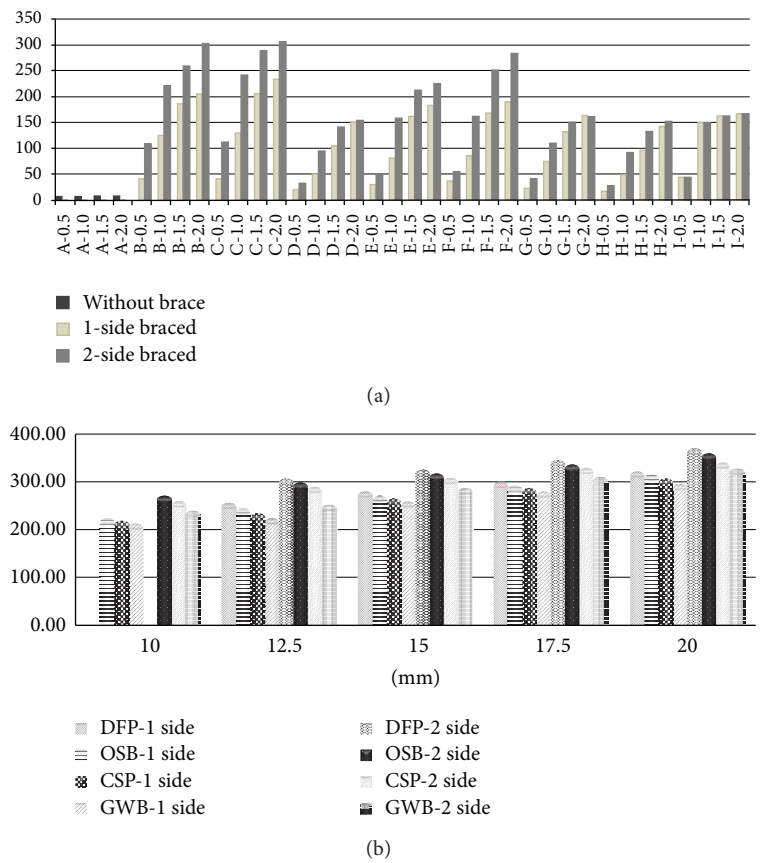

Figure 14: Maximum strength of the specimens (kN). (a) Braced walls. (b) Sheeting walls.

If the bilateral brace is used for sample I, the value of the response modification factor will be about 9.45% more than that of the lateral brace. This small difference can result from the lack of additional resistance in samples.

The value of predicted stiffness was obtained as 14.58 and 19.57 kN/mm, respectively, for samples I with lateral and bilateral braces under monotonic loading, and 20.79 and 22.11 kN/mm, respectively, under cyclic loading. Obviously, the value of (K_e) is considerably obtained less than (K_p). The shear resistance is increased by increasing the ratio of the height to length of the wall. On the other hand, the shear wall resistance of bilateral brace was significantly obtained more than that of lateral brace. This has clearly been shown in Figures 11(a)–11(c).

4.10. Numerical Analysis of the Shear Wall Panels with the Sheathings (Cover Plates). The shear wall panels could not be anticipated to lodge the quota but could be expected to lodge the resistance of the nominal design shear well. The K_e value was obtained considerably less than K_p.

4.11. The Effect of the Frame Aspect Proportions on the Behavior of the Shear Wall Panels. As indicated in Tables 6 and 7, the frame aspect proportions have no effect on the behavior of the shear wall panels with the wooden covers. Then, with a good approximation, the aspect proportion 1 can be considered as a basis to continue the calculation of these walls.

4.12. The Effect of Thickness and the Sheathing Types on the Behavior of the Shear Wall Panel. DFP sheathings have the most strength and rigidity in comparison to other studied sheathings. OSB, CSP, and GWB are other sheathings that followed DFP. Using two-sided sheathings increases the rigidity and strength in the shear walls compared to one-sided sheathings. On the other hand, by increasing the thickness, the amount of the hardness decreases in two-sided braces compared to one-sided braces; so, there was observed an 18% increase for the thickness 12.5 mm and a 12% increase for the thickness 20 mm in walls with sheathings DFP, OSB, and CSP and the amount of the increase in all thicknesses of walls with the sheathing GWB was approximately 10% (see Figure 12).

4.13. Comparing the Amounts of the Response Modification Factors of Specimens. The results of the calculation related to the response modification factor of all specimens are shown in Tables 8 and 9. Also, quality diagram is shown in Figure 13. The maximum value of the factor modifications in the shear wall panels with the sheathings is related to GWB with the thickness 15 mm and the amount 5.14 between braced specimens and in two-sided braced specimens were related to the specimen B with the amount 3.14. The results

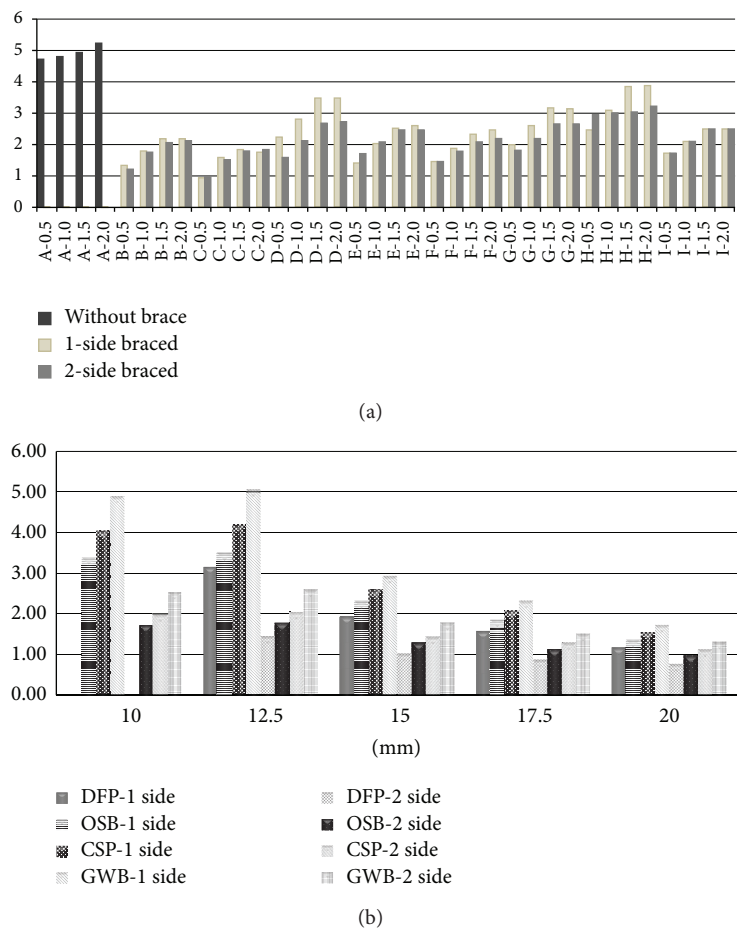

Figure 15: Maximum lateral drift ratio (%). (a) Braced walls. (b) Sheeting walls with variable of the thickness.

showed the response modification factor of the shear wall models was higher than the braced models. In the shear wall specimens, by increasing the thickness of the wall, the amount of its plasticity has decreased but, on the other hand, the amounts of the hardness and resistance have increased. As the response modification factor is related to these components, by increasing the thickness, one is increased and the other one is decreased; therefore, we cannot consider a regular routine toward the increase and the decrease of the response modification factor by increasing the thickness of the wall. In DFP sheathing, the highest amount of the response factor modification was 3.99 in one-sided positions with the thickness 12.5 mm. In OSB sheathing, the highest amount of the response factor modification was 4.23 in one-sided position with the thickness 15 mm. In CSP sheathing, the highest amount of the response factor modification was 4.29 in one-sided positions with the thickness 12.5 mm.

4.14. Comparing the Maximum Drift and Resistance of Samples. There is shown a quality comparison in Figures 14 and 15 to compare the maximum resistance and the drift in specimens. The maximum amount of resistance among the specimens with bilateral (2-side) bracing systems belongs to the specimen C (2-side double X-bracing) with the dimension ratio of 2 (4.8 m × 2.4 m) and resistance of 305.60 kN and also among the shear wall panels with sheathing plates, it belongs to DFP (douglas fir plywood) with a thickness of 20 mm and resistance of 371.34 kN. The least amount of the drift between braced specimens is related to specimen C with the aspect proportion 0.5 and between the shear wall panels with the sheathings is related to DSP specimen with the thickness 20 mm and they are the least drifts 0.92 and 0.72, respectively.

As it can be observed, compared to braced frames, frames without braces have very low resistance and more drifts. In unilateral braced samples, the average maximum resistance in samples B, C, D, E, F, G, H, and I, 94%, 95%, 91%, 93%, 94%, 92%, 90%, and 94%, respectively, was observed more than frames without braces. The amount of this difference in bilateral braces for the above-mentioned samples was 97%, 97%, 93%, 95%, 96%, 93%, 92%, and 94%, respectively, more than that in frames without braces. However, compared to the unilateral braced samples, drift in specimens without braces increased as 62%, 69%, 39%, 56%, 59%, 45%, 33%, and 55%.

Compared to the above-mentioned with bilateral cross bracing systems, the amount of drift in samples without bilateral cross bracing system increased as 64%, 69%, 54%, 56%, 62%, 53%, 38%, and 55%, respectively.

5. Conclusion

Between braced specimens, specimen C, and between the shear wall panels with the sheathings, the sheathing DFP had the maximum mean value of yield strength S_y, the maximum mean value rigidity, and maximum energy absorption capability. Obviously, the lowest amount in these parameters is related to specimen A, which is the wall without any bracing. The highest mean ratio S_y/S_{yp} between braced specimens is related to the specimen D, and the highest mean ratio S_y/S_{yp} between the shear wall panels with the sheathings is related to the specimens DFP with the sheathing thickness of 20 mm. The highest mean ratio S_y/S_{yp} between braced specimens is observed in the specimen A and between the shear wall panels with the sheathing DFP with the sheathing thickness of 20 mm. Also, there is observed maximum drift among the walls without any bracing in the specimen A and minimum drift in specimen C. In general, the shear wall panels with the sheathings and one-sided braced plates could not achieve their anticipated quota. However, the statuses of two-sided braced specimens compared to one-sided braced specimens were improved and they can achieve their anticipated quota in cyclic experiments.

The shear wall panels, the sheathings, and braced specimens B and C in one-sided and two-sided bracings could reach the expected shear resistance of nominal design. In one-sided bracing, they cannot, entirely, reach the expectations. Samples E and I could reach the expected shear resistance of nominal design in two-sided bracing.

In all cases, the amount K_e was considerably lower than K_p.

The maximum response modification factor in two-sided bracing for the specimen B was 3.14 and in one-sided bracing for the specimen H was 2.35. It is also observed that there is little difference between one-sided bracing and two-sided bracing on the response modification factor of specimens D, G, H, and I. In the wall without any bracings, there was observed higher plasticity than other specimens but in this specimen, there was no excessive strength. The response modification factors in the shear wall models were better than those in braced walls. The highest amount was 5.14 related to the shear wall GWB with two-sided sheathing and the thickness 15 mm.

In all braced specimens with the increase of the wall height-to-length ratio, the shear resistance of two-sided braced wall was 60% more than one-sided bracing. In the increase of the resistance of the shear wall panels with the sheathings in the sheet with lower thickness, the effect of two-sided sheathing was more than two-sided sheathing. As for the walls with the sheathing DFP, OSB, and CSP, there was seen an 18% increase for the thickness 12.5 mm and a 12% increase for the thickness 20 mm in the one-sided sheathing resistance (compared to two-sided sheathing resistance). In the sheathing GWB, the amount of the increase in all thicknesses was about 10%.

The aspect proportion of the frame had no effect considerably on the shear wall panels with the sheathing plats. The only influential component on the behavior of these walls was the thickness of the panels.

Conflict of Interests

The authors declare that there is no conflict of interests regarding the publication of this paper.

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