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Seismic performance assessment of hybrid braced cold-formed steel walls

*** Nader Fanaie [1], Mostafa Rezvani Sharif [2], Ebrahim Afsar Dizaj [3]**

Abstract

Cold-formed steel (CFS) structures or light weight steel frame (LSF) structures have been frequently used in recent years due to their fast constructions as well as consumed light weight steels. CFS structures are constructed with light thin cold formed steel members. Lateral loading systems of these structures are often steel or wooden shear walls and merely tensional steel cross-straps in the form of shear panels. The performance of lateral loading system with hybrid bracing is evaluated in this study considering the simultaneous use of steel shear wall and strap cross-bracing in different floors of a CFS structure. For this purpose, nonlinear static analysis, linear dynamic analysis and incremental dynamic analysis (IDA) have been conducted on 1-3 storey frames with shear wall, steel strap cross-bracing and hybrid bracing using OpenSees software. Response modification factor (R) is one of the most important seismic parameters of structures against earthquake loads. In fact it shows the capability of structural system in absorbing and dissipating the energy caused by earthquake and creating nonlinear deformations without total collapse of structure. This factor has been calculated here for the mentioned lateral bracing systems. The obtained values are 4.1 and 3.1 for CFS structures with shear wall and cross-bracing, respectively, and 4.9 and 2.8 for hybrid bracing considering the shear wall and bracing in the lower storeys, respectively. Based on the results of this research, it seems that hybrid bracing of shear wall in the lower storeys is one of the best lateral bracing system for cold-formed steel structures.

Keywords:

Cold-Formed Steel Structures, Hybrid Bracing, Incremental Dynamic Analysis, Response Modification Factor, Over Strength Factor, Ductility Factor.

*[1]. Corresponding author, Ph.D., Department of Civil Engineering, K. N. Toosi University of Technology, Tehran, Iran, E-mail: fanaie@kntu.ac.ir

[2]. Ph.D., Department of Civil Engineering, K. N. Toosi University of Technology, Tehran, Iran, E-mail: barghi@kntu.ac.ir

[3]. Faculty of Engineering, University of Guilan, Rasht, Iran, Ph.D. Student, E-mail:ebrahim_afsar@live.com

Introduction

Today, the velocity of construction has been considered along with the progression in the building construction industry all over the world. By the way, CFS structures are presented in the construction arenas as the modern product of building industry as well as the consequence of serious champions for short and medium conventional steel and concrete structures. High ratio of strength to weight in the members of CFS structures will reduce the needed material which results in the improvement of accuracy and lowering of the costs.

The elements forming CFS structures are light thin steel sections. These sections are metallic cold formed steel elements which are formed by rolling method. The uniform thickness of the sections widths along with the mentioned construction method results in producing mass volume of uniform sections with high quality. The thin steel sections are light and easily portable. Different parts of the buildings can be assembled by these sections. All these result in the very fast constructing by this system (Yu and LaBoube, 2010).

Response modification factor is one of the important seismic parameters in the structural designing. It shows the capability of structures in absorbing and dissipating the energy caused by earthquakes and forming nonlinear deformations without total collapse of the structures. Besides, it is worthy to consider the effects of different earthquakes on the designed structures and assess the structural performance levels as well as the caused damages according to the codes. The results of structural designing are studied according to the last compiled codes to obtain the performance of structures against different earthquakes. By the way, the efficiency or deficiency of existed codes is evaluated for seismic designing of certain systems. There are various methods for calculating the response modification factor of structures and evaluating their seismic performances.

Recently, many researchers have been studied cold-formed steel section elements and their structural behavior both experimentally and numerically (Anbarasu et al., 2013; García-Palencia1a & Godoy, 2013; Heva & Mahendran, 2013; Kwon et al., 2014; Phan et al., 2013; Rosario-Galanes & Godoy, 2014; Valsa et al., 2013).

However, no comprehensive research is found in the literature, considering exact dynamic analysis on the seismic performance and response of hybrid braced systems in the cold formed structures against real earthquake accelerations. This research focuses on modifying the seismic design of cold formed steel structures with shear wall or cross bracings. This attempt can suggest the principles for designing codes and implementing such structures.

This study is conducted on the models which include shear wall system and/or cross bracing. Response modification factor and overstrength factor are of important parameters in the designing of structures which play crucial roles in the determination of the loads applied to the structures by earthquake as well as in controlling the seismic elements of structures. Different values have been suggested for these factors in the past studies conducted by the researchers. These values cannot be generalized in most cases due to the insufficient number of studied models or not performing dynamic analysis in nonlinear scope with proper number of records. In addition, the studies conducted on such systems under different earthquake records are insufficient. Therefore, it is recommended to evaluate the structural behavior under recorded earthquakes through incremental dynamic analysis. In this way, it is possible to modify some viewpoints and accurate the structural response obtained from the results and modeling under earthquake effects.

Concerning the objective of this research, 12 two-dimensional 1-3 storey structures are designed in SAP2000 software having cross bracing system and/or shear wall, separately and compositely. Then, they are modeled in OpenSees software for conducting time history dynamic analysis. Uang method has been used to calculate the values of response modification and overstrength factors.

Cold-Formed Steel Structures

Shear panel columns are composite sections constructed with two or more studs in the form of tubular sections, box or I. These elements are controlled by force and located in both sides of the shear panel. They are responsible for supporting the eccentric tensional-compressional loads of panel as well as maximum expected earthquake loads. These loads are created in the strap bracing due to the yielding and meeting the end limit of bracing strength because of earthquake forces. The mentioned columns should be designed and detailed properly for transmitting the lateral loads from top to bottom levels in order to remain elastic during earthquakes. The columns participate poorly in dissipating seismic energy. Due to the thin formed sections of shear panel columns, they cannot support the stresses, caused by bending, up to the yielding limit. Consequently, these sections cannot form plastic flexural hinge at the column base and are locally buckled prior to meeting the yielding (Us Army Corps of Engineers, 2006).

Cross straps are steel plates installed obliquely at one or both sides of the shear panel for supporting seismic loads. Cross straps performance is merely in tensional form for supporting lateral loads. These elements cannot act in the compression status because of being thin. They should be braced in the connection point with shear panel studs.

Cross straps are the members controlled by deformation and should have sufficient ductility to dissipate seismic energy by supporting large deformations, at least 10 times yielding axial deformation of the member, without brittle collapse. Pretensioning or other installation methods should be applied to the tensional cross straps to perform appropriately and prevent them from unclenching (Us Army Corps of Engineers, 2006).

The studied models

In this research, 3 groups of structures have been studied: 1) 1, 2 and 3 storey structures with steel shear walls; 2) 1-3 storey structures with cross steel strap bracing; 3) 1-3 storey structures with composed shear wall and cross steel strap bracing. English letters have been used for easier nomination of the structures. That is S is for shear wall and X for cross bracing. For instance, SSX is a 3 storey structure with shear wall in its first and second stories and cross bracing in the third. SAP2000 14.2.0 software has been used to model, analyze and design the samples with the load and resistance factor design method (LRFD) in two-dimensional form on the soil type II. The structures are designed using the standard AISI-LRFD 96, American Iron and Steel Institute (2007) and then evaluated and analyzed in the finite element program OpenSees (Mazzoni et al., 2007) to calculate the response modification factor of the system. It is assumed that the structures are in Tehran which is located on the relatively high risk region. Therefore, the base acceleration is considered as A=0.35 and considering the residential application of the building, its importance factor is selected as I=1. The initial response modification factor for designing the structures is assumed as R=4 according to American Iron and Steel Institute (2007). The height of floors has been considered as 2.44m; a ceiling with 30cm depth is between the stories (having 10cm concrete slab as well). The structures have 3 spans.

Figure 1. shows a 2- storey structure in the different statuses of bracing. The middle and side frames have 1.22m and 2.4m widths, respectively. The middle studs are located at the distance of 0.6m from each other. Each structure has 3 middle truck in each storey. The dead loads of 300kg/m2 are considered for gravitational loading and the live loads of 200kg/m2 and 150kg/m2 for stories and roof, respectively, regarding the residential application of the building.

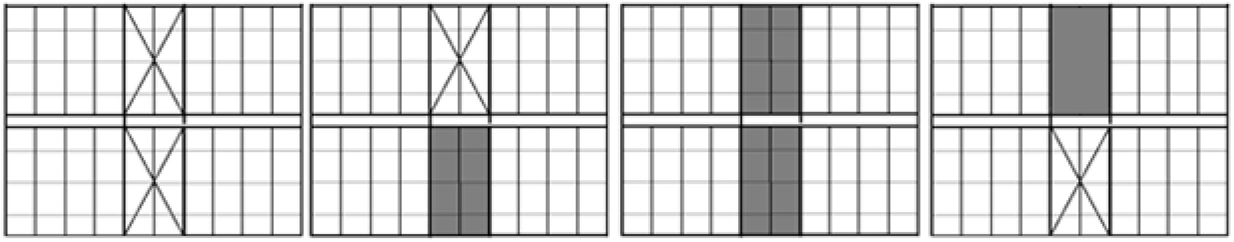

Figure 1. The configuration of a 2-storey structure with different bracings

Modeling the two dimensional structures in the OpenSees software

The considered structures have been modeled using OpenSees software (Mazzoni et al., 2007). In this study truss elements with pinching 04 material properties have been applied for modeling the shear wall which includes steel plate and bolt connections. In this regard the values presented in Shamim and Colin (2013) have been used concerning the equality of the sections with those of the mentioned research. The shear wall has been modeled with bracing net and truss element with hysteretic material specifications has been applied for cross bracing steel strap, Figure 2.

The main studs have been modeled with elastic beam-column element. The tracks are considered as rigid beam-column element. Elastic truss element has been applied for modeling the ceiling. In order to consider the effect of P-Δ, a virtual column with rigid beam-column element has been connected to the side of structure using a rigid truss element (Shamim and Colin, 2013). Four elastic rotational spring elements have been applied in the four corners of the frame in order to show the in-plane flexural stiffness of the frame (without steel plate and strap), Figure 3. This lateral stiffness is provided in the frame by the connections of stud to the track and blocking. Linear elastic spring element has been used for modeling the anchor rod (Shamim and Colin, 2013). The columns are designed in such a way to remain elastic after fracturing of strap or detaching the sheet. This is because the structure experiences severe damages in the earthquake. Therefore, the structure should be designed in such a way that its gravitational loading capacity does not confront critical status after damaging. Accordingly, the lateral loading elements such as shear wall and steel strap should be enter non-elastic zone in order to dissipate earthquake energy. However, other elements and connections of structure are designed to remain in the elastic range (FEMA 2000).

As the steel material, used in this research, is ASTM A653 steel with 230 and 340 grades, its yielding and fracture stresses are F_y=230 MPa and F_u=310 MPa for grade 230, respectively, and F_y=340 MPa and F_u=450 MPa for grade 340, respectively. The modulus of elasticity has been considered as E=2.03×105 MPa. The Rayleigh damping definition has been used to consider the energy dissipation in the nonlinear dynamic analyses. The corotational Coordinate Transformation has been used for all elements.

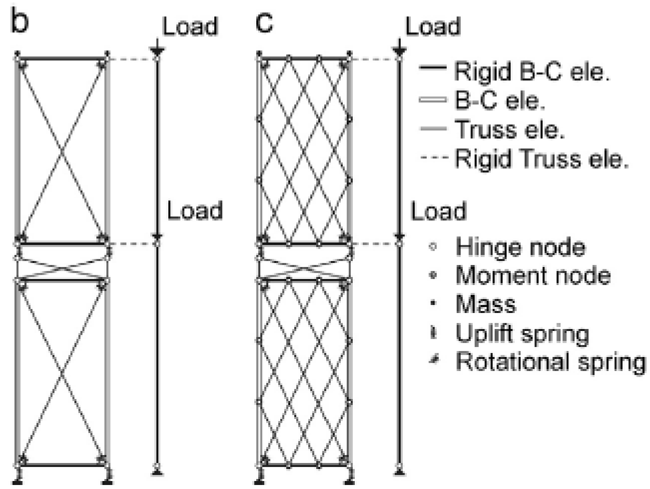

Figure 2. The structure modeled in OpenSees: a) Cross bracing; b) shear wall (bracing net)

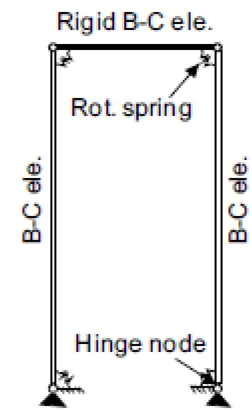

Figure 3. The structure without bracing or steel plate

Controlling the accuracy of modeling and OpenSees software application

The hysteretic curve has been plotted for a one-storey one-span structure with shear wall based on Shamim and colin (2013), and pushover curve has been also plotted for a one-storey one-span structure with steel strap cross bracing according to Comeau et al. (2007). These two curves are compared with those obtained from numerical models, Figure 4a & b.

Figure 4a & b definitely shows the concordance between the hysteretic curves obtained from experimental results and OpenSees model. The loading cycles are based on the CUREE protocol which is similar to that of ASTM E2126 (2005) (Balh and colin, 2010). In the OpenSees model, as shown in Figure 4a, the stiffness and strength degradation parameters have been considered which result in the higher concordance between the experimental and modeling results.

Figure 4b. shows the agreement between the results of experimental and OpenSees models in the pushover curves of the structure with cross bracing. These two curves show proportion limit concordance; however, as the bracing is modeled by hysteretic material for which only three points are assigned, the experimental results are slightly different from those of modeling in the regions between the mentioned points.

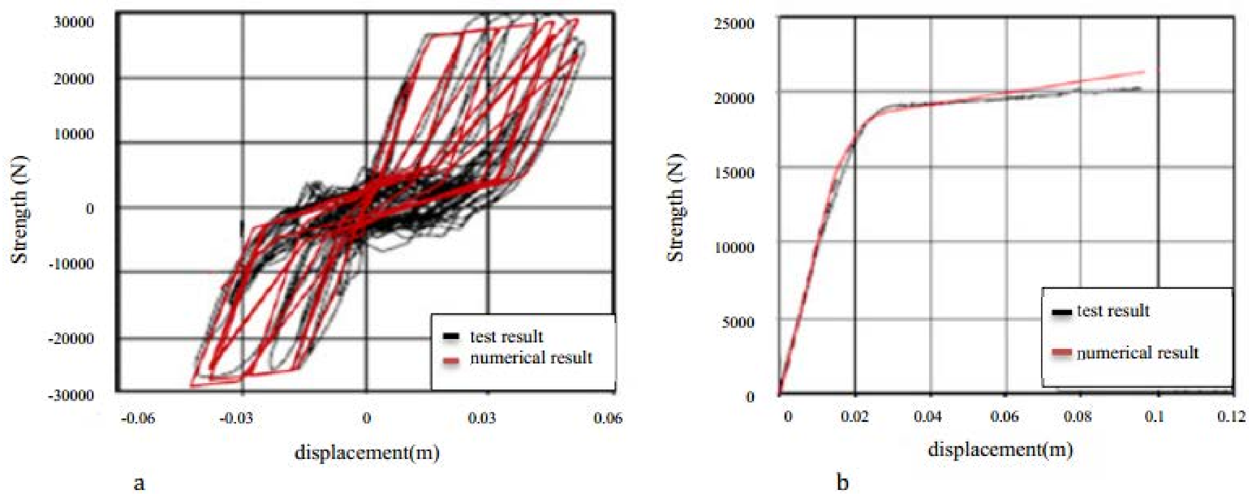

Figure 4. Comparing the experimental results and OpenSees model: a) Hysteresis curves for one-storey structure with shear wall; b) Pushover curves for one-storey structure with cross

Incremental dynamic analysis of models

According to the studies of Shome et al. (1998), 10- 20 earthquake records have appropriate accuracies in estimating the structural seismic responses of low and mid-rise structures. A sum of 12 records has been selected out of the earthquakes recommended in FEMA 695 (2009) for incremental dynamic analysis of 1-3 storey structures. These records are corresponded to the soil type II. The details of the records used for incremental dynamic analysis are prepared in Table 1.

Table 1. The specifications of the records used for incremental dynamic analysis (FEMA 695, 2009)

	Name	**Recording Station**	**Year**	**PGA**
1	Chi-Chi, Taiwan	CHY101	1999	**0.44**
2	Duzce, Turkey	Bolu	1999	**0.82**
3	Cape Mendocino	Rio Del Overpass	1992	**0.55**
4	Imperial Valley	Delta	1979	**0.35**
5	Kobe, Japan	Shin Osaka	1995	**0.24**
6	Kocaeli, Turkey	Duzce	1999	**0.36**
7	Landers	Yermo Fire Station	1992	**0.24**
8	Loma Prieta	Capitola	1989	**0.53**
9	Northridge	Beverli Hills – Mulol	1994	**0.52**
10	Northridge	Canyon Country - WLC	1994	**0.48**
11	Superstition Hills	El Centro lmp. Co.	1987	**0.36**
12	Superstition Hills	Poe Road(Temp)	1987	**0.45**

The behavior of a structure under an earthquake record is important; however, it cannot be generalized for all records. As the obtained result is not general and cannot be considered as the general behavior of structure under all conditions, different times and various earthquakes. In the other words, several single curves are needed to evaluate the seismic behavior of structure appropriately. Looking at the curves, all stages of the behavior of structure under earthquake can be well observed from elastic limit up to collapse and general instability. The IDA curves packages of SS structure are shown in Figure 5.

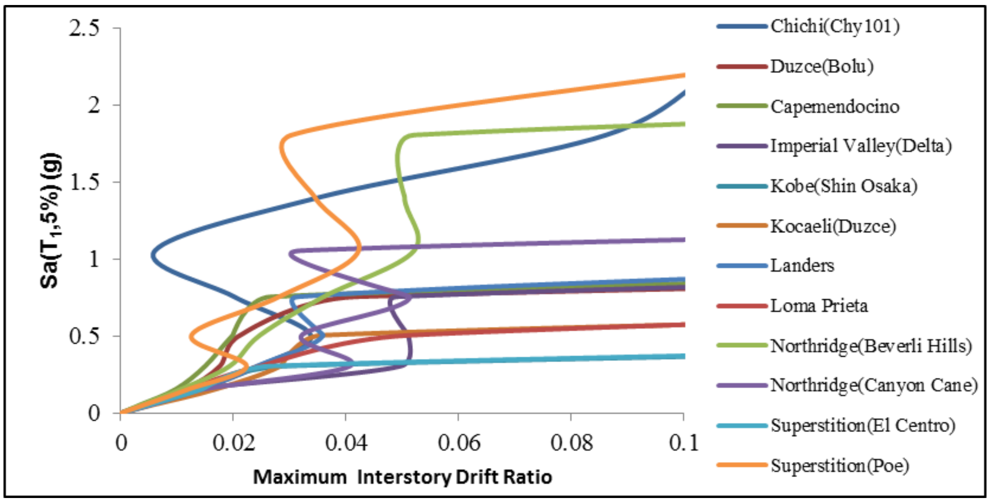

Figure 5. IDA curves packages for SS structure

Calculating response modification factor using the results of incremental dynamic analysis

The values of Overstrength can be calculated through nonlinear incremental dynamic analysis. In the method, presented by Elnashai and Mwafy (2002), nonlinear incremental dynamic analysis is used to obtain maximum base shear. The ratio of final base shear ($V_{b(Dyn,u)}$) to the base shear corresponding to the first yielding ($V_{b(Dyn,y)}$) is presented as overstrength factor. This method is modified according to the results obtained in Massumi et al. (2004) as follows:

$$R_s = \frac{V_{b(Dyn,u)}}{V_{b(St,y)}} \tag{1}$$

It means that oversrength factor is the ratio of dynamic base shear (which results in the formation of mechanism (instability) in the structure) to static base shear (which is corresponding to the formation of the first plastic hinge in the structure). In order to obtain $V_{b(Dyn,u)}$, spectral acceleration (seismic intensity criteria, used in this research) of the considered earthquake increases up to the formation of mechanism (instability) or meeting the concerned damage limit in the structure. The spectral acceleration which has made the formation of mechanism is accepted as the end limit and its corresponding base shear is obtained (Fanaie and Ezzatshoar, 2014; Fanaie and Afsar Dizaj, 2014). The considered damage limit is assigned for life safety regarding the base rehabilitation and earthquake risk level 1. Based on FEMA 356, the inter-storey drift ratio is 1.5% for the structures with braced steel frames.

In this method ductility factor (R_μ) is obtained directly using the results of incremental dynamic and linear dynamic analysis and expressed as follows:

$$R_\mu = \frac{V_{b(Dyn,el)}}{V_{b(Dyn,u)}} \tag{2}$$

In order to obtain $V_{b(Dyn,u)}$, the spectral acceleration (seismic intensity criteria, used in this research) of considered earthquake is increased up to the formation of mechanism (instability) or meeting the concerned damage limit in the structure. The spectral acceleration which has made the formation of mechanism is accepted as the end limit and its corresponded base shear is obtained. Maximum linear base shear ($V_{b(Dyn,el)}$) is also computed by dynamic analysis of structure assuming its elastic behavior under the same spectral acceleration (Elnashai and Mwafy, 2002).

Regarding the contents of previous sections, the values of response modification factor is calculated for all studied structures under earthquake records as follows (Elnashai and Mwafy, 2002):

$$R_{LRFD} = R_\mu \times R_s \tag{3}$$

$$R_{ASD} = R_\mu \times R_s \times Y \tag{4}$$

The relations (3) and (4) are based on the ultimate strength and allowable stress methods, respectively, of the designing code. Y is allowable stress factor equal to 1.44, according to Us Army Corps of Engineers (2006). How to obtain the response modification factor for 3-storey structures has been presented in Table 2. According to this table, in order to obtain the response modification factor with respecting to the Sa corresponded to the considered drift, final base shear ($V_{b(Dyn,u)}$) and maximum linear base shear ($V_{b(Dyn,el)}$) are obtained from incremental dynamic analysis. The values of overstrength and ductility factors are calculated, having the base shear corresponding to the first yielding ($V_{b(st,y)}$) obtained from nonlinear static analysis (Fanaie and Ezzatshoar, 2014; Fanaie and Afsar Dizaj, 2014).

Table 3. presents the values of response modification factors. As mentioned in Table 4, the maximum value of response modification factor belongs to the hybrid braced structure with shear wall in its lower stories. The value of overstrength factor is higher in the hybrid braced structure with cross bracing in the lower stories, comparing to other structures. The ductility factor of cross braced structure has higher values comparing to the structure with shear wall. The final value of response modification factor is obtained for different bracings by averaging the obtained values and presented in Table 5.

Table 2. The values of overstrength, ductility and response modification factors for SXX

Records	DM Max Drift	IM $Sa(T_1,5\%)$	$V_{b\,(Dyn,u)}$ (N)	$V_{b\,(st,y)}$ (N)	$V_{b\,(Dyn,e)}$ (N)	R_S	R_μ	R_{LRFD}	R_{ASD}
Chichi(CHY101)	0.015	0.850	28120	14673	96293	1.916	3.424	6.562	9.187
Duzce(Bolu)	0.015	1.790	27505		57367	1.874	2.086	3.910	5.473
Cape Mondecino	0.015	1.850	28296		86966	1.928	3.073	5.927	8.298
Imperial Valley(Delta)	0.015	1.570	27768		75730	1.892	2.727	5.161	7.225
Kobe(Shin Osaka)	0.015	2.200	28089		54570	1.914	1.943	3.719	5.207
Kocaeli(Duzce)	0.015	2.980	28700		103800	1.956	3.617	7.074	9.904
Landers(Yermo Fire Station)	0.015	2.080	27659		63842	1.885	2.308	4.351	6.091
Loma Prieta(Capotila)	0.015	2.750	28215		116220	1.923	4.119	7.920	11.089
Northridge(Beverli Hills)	0.015	0.940	27733		57728	1.890	2.082	3.934	5.508
Northridge(Canyon Country)	0.015	2.010	28298		51559	1.929	1.822	3.514	4.919
Superstition(El Centro)	0.015	1.310	29458		84377	2.008	2.864	5.750	8.050
Superstition(Poe)	0.015	0.800	27963		98912	1.906	3.537	6.741	9.437
Average						1.918	2.800	5.380	7.532

Table 3. The values of overstrength, ductility and response modification factors

Structures	R_S	R_μ	R_{LRFD}	R_{ASD}
S	2.0	2.1	4.2	**5.9**
X	1.4	2.3	3.2	**4.5**
SS	1.7	1.6	3.6	**5.1**
XX	1.6	2.1	3.4	**4.8**
SX	1.6	2.4	3.9	**5.5**
XS	1.8	1.4	2.5	**3.5**
SSS	1.9	2.4	4.6	**6.5**
XXX	1.2	2.3	2.9	**4.0**
SXX	1.9	2.8	5.4	**7.5**
SSX	1.9	2.8	5.4	**7.5**
XSS	2.1	1.5	3.0	**4.2**
XXS	1.6	1.9	3.0	**4.2**

Table 4. The structures with maximum values of overstrength, ductility and response modification factors

number of stories	Max R_S	Max R_μ	Max R_{LRFD}
1	S(2.0)	X(2.3)	**S(4.2)**
2	XS(1.8)	SX(2.4)	**SX(3.9)**
3	XSS(2.1)	SXX(2.8)	**SXX(5.4)**

Table 5. Average values of overstrength, ductility and response modification factors

bracing type	R_S	R_μ	R_{LRFD}	R_{ASD}
shear wall	1.87	2.03	4.13	**5.83**
cross bracing	1.40	2.23	3.17	**4.43**
hybrid (shear wall in the lower stories)	1.80	2.67	4.90	**6.83**
hybrid (shear wall in the upper stories)	1.83	1.60	2.83	**3.97**

Conclusion

The results obtained in this research are briefly summarized as follows:

1- The values of response modification factor are 4.1, 3.1, 4.9 and 2.8 for cold formed steel structures with shear wall, cross bracing, hybrid bracing of shear wall in the lower stories and hybrid bracing of cross bracing in the lower stories, respectively.

2- The values of ductility factor are 2.0, 2.2, 2.7 and 1.6 for cold formed steel structures with shear wall, cross bracing, hybrid bracing of shear wall in the lower stories and hybrid bracing of cross bracing in the lower stories, respectively.

3- Maximum response modification factor is obtained for the cold formed steel structure with hybrid bracing in which the shear walls are in the lower stories. Therefore, it is economically convenience to design the mentioned structures with such bracings.

4- Better results can be obtained from hybrid bracing if the shear walls are placed in the lower stories and cross bracing in the upper stories, comparing the structures with hybrid bracings.

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