ISSN XXXX XXXX © 2017 IJESC

Research Article Volume 7 Issue No.12

Response Modification Factor for Cold-Formed Steel Structures Using Pushover Analysis

Vafa Soltangharaei[1], Mehdi Zarean[2], Vahid Mahdavifar[3], Ramin Taghinezhad[4], Arash Taghinezhad[5]
Graduate Research Assistant[1, 2, 5], Post-Doctoral Research Associate[3], Research Assistant[4]
Department of Civil and Environmental Engineering[1, 4], Department of Civil Engineering[2],
Department of Environmental Conservation[3], Department of Construction Management[5]
University of South Carolina, USA[1]
Building and Housing Research Center, Tehran, Iran[2]
University of Massachusetts Amherst, USA[3]
Florida International University, USA[4]
Louisiana State University, USA[5]

Abstract:
Cold-formed light steel structure systems have recently gained attention by researchers for their unique features such as speed and ease of construction and lightness. The main object of this research is finding response modification factor for cold-formed diagonal walls and multi-story cold-formed steel structures to extend the applicability of these systems in seismic regions. For this purpose, a numerical model based on experimental results for a selected frame was prepared. To calibrate the modeling parameters, the results of the nonlinear static analysis are compared with the results of experimental research subjected to cyclic and uniform load. Then, four structures are modeled with a different number of stories but with similar plan. The effect of seismic load pattern is investigated on these structures using nonlinear static (pushover) analysis and response modification factor for the case study structures are obtained. It is shown that the capacity of cold-formed steel structures for load bearing in the plastic area decreases by increasing height.

Keywords: Cold-Formed Light Steel Structure, Nonlinear Static Analysis, Pushover Analysis, Response Modification Factor.

I. INTRODUCTION

Steel members for building applications are usually manufactured using two major methods: hot-rolled members in which the members manufacturing involve rolling the steel at high temperatures which are usually above the recrystallization temperature of steel. In this temperature, steel can be shaped and formed easily. Cold-formed steel members are the other way of manufacturing steel members in which the members are formed at essentially lower temperature close to room temperature. Cold-formed steel(CFS) structures have been experienced a significant application growth in recent years for their unique features, such as lightness, high strength, ease of implementation, identical quality, recycling capacity and cross-section variation. In cold-formed structures, diagonal members can be used as a lateral bracing system. A practical approach is the use of x-bracings within spans [1]. Cold-formed frame bracing mainly behaves as a fuse during an earthquake. In the past, some studies have been conducted on the seismic behavior of these structures. Adham et al. [2] conducted five cyclic loading tests on several frames with a dimension of 2.44 × 2.44 m with a double profile in boundary elements, diagonal x-bracing system, and gypsum board. Diagonal belts were recognized as the most important member in lateral resistance of the structures. The results showed that lateral bearing capacity increased as well as deformation decreased by increasing the cross-section of bracing belts. Moreover, it was stated that seismic lateral force resistance was reduced with a buckling in the upper corner of the diagonal profiles. In another experimental test, Kim et al. [3] tested cold-formed steel frames in full-scale on a shaking table. These specimens included 2-story and one span. The results showed that in full-scale experimental tests, the x-bracing system represented high ductile behavior. Dubina and Fulop [4] experimented three walls, braced on both sides, with an aspect ratio of 1.5 (3.6 m length × 2.44 height) under uniform and cyclic loadings. Loading-unloading curves indicated that maximum lateral load capacity in shear strength was observed when local failure happened in the lower corners. For steel and reinforced concrete buildings material ductility, cross-section ductility, member ductility, and structure ductility are widely used [5,6]. While in timber constructions since wood is an inherently brittle material and timber elements exhibit almost no potential for energy dissipation. Thus, in a timber structure the only elements that provide ductility, and consequently exhibit hysteretic dissipation of energy under cyclic loading, are the metal connection systems as documented by many researchers [7-9]. These philosophies are applied to other no building type structures facing extreme loading condition but are beyond the scope of this work [10-11]. The response modification factor can change from 1.5 for very brittle structures such as unreinforced concrete to 8 for very ductile elements such as moment frames. Several researchers worked on estimating response modification factor and other parameters such as over strength, ductility, redundancy, and compared with the suggested values with the code for different building and bridge structure systems [12-14]. An accurate estimation of the period of vibration is important to calculate the response modification factor [15]. For some non-ductile elements, the response modification factor should be taken as unity for example, for the pile to pile connection in the bridge structures, the response modification factor should be taken as unity, because this connection should behave elastically under earthquake ground motions. [15]. Gad et al. [17] conducted some experimental tests to evaluate the behavior of cold-formed steel frames under earthquake load. The results indicated that seismic behavior of the structures was governed by the belt-bracing system, and when plasterboard combined with a diagonal bracing, the overall stiffness of the system can be obtained from a simple superposition of plasterboard and brace stiffness, individually. Ronagh and Moghimi [18] studied ductility of cold-formed steel walls under reciprocal load including twenty experimental specimens in full-scale with five different types of bracing. These experiments were conducted to investigate the effect of gravity load on inelastic seismic response the effect of non-structural gypsum board on the lateral ductility of the braced wall, the effect of both-side bracing, and the effect of double boundary element. North American Specification for the Design of Cold-Formed Steel Structural Members (AISI S100) has been issued as guidance for design and construction of cold-formed steel structures [19]. In this study, single-span frames with the x-bracing system are modeled. The modeling procedure is based on the experimental research performed by AISI S213 [20]. The numerical model responses are compared with the experimental results. In second section of this paper, four buildings with light steel framed system are modeled and the response modification factor was derived.

II. MODELING OF SINGLE-SPAN FRAME AND COMPARISON OF ANALYTICAL WITH EXPERIMENTAL RESULTS

Typical frame specimens were selected for study [20]. Two specimens including 19A-M and 20A-C have been selected. The specimens with A-M suffix were subjected to uniform loading and the specimens with A-C suffix were subjected to cyclic loading. Typical frame specimens were selected for study [20]. Two specimens including 19A-M and 20A-C have been selected. The specimens with A-M suffix were subjected to uniform loading and the specimens with A-C suffix were subjected to cyclic loading.

A. Geometric Specifications of Frame and Materials Properties

The frame has 2440 mm length and 2440 mm height. Moreover, the frame has 5 middle and 2 main studs, which are located within 406 mm from each other. The bracings have belt sections with a dimension of 1.37×69.9 mm with a cold-formed steel at a resistance of 340MPa. The side studs (main) are C section with a dimension of 152×41×12.7 mm and with a thickness of 1.37 mm and a resistance of 340 MPa. Each main stud profile consists of double C sections that are bolted back to back to each other. Inner stud sections are 152×41×12.7 mm and with the thickness of 1.9 mm and a resistance of 230 MPa. For tracks, C sections with a dimension of 152×31.8, the thickness of 1.73 mm and a steel resistance of 340 MPa are used. Moreover, a C section with a dimension of 38×12.7 mm and a thickness of 1.09 mm is installed at half-height of the frame [8]. Figure 1 shows a typical specification of cold-formed steel frame modeled in CSI SAP 2000 [21]. For nonlinear static analysis, plastic hinges are assumed to be formed on the braces. Since the belt braces do not practically resist compression and only carry out tension, therefore, assigned plastic hinge curve was not symmetric. The belt braces do not resist compressive loads due to lack of out of plane stiffness and their slenderness. For this reason, the plastic hinges in the braces were defined asymmetrically based on FEMA 356 [22].

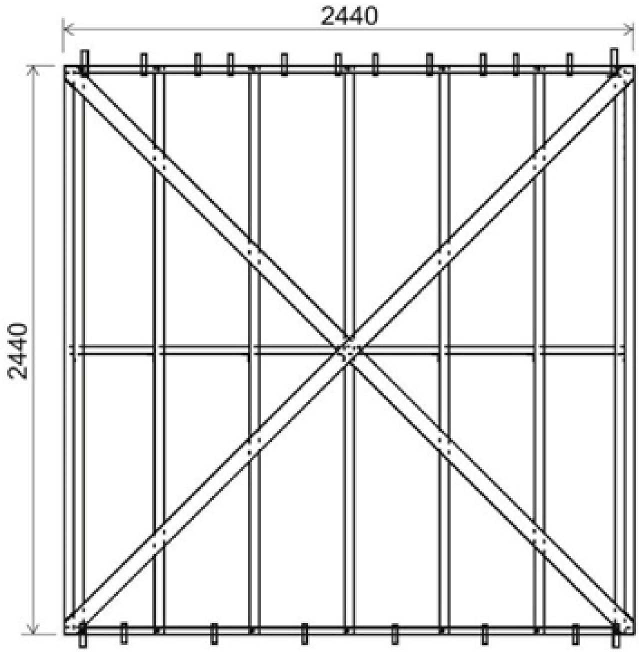

Figure.1. Schematic picture of the experimental frame (measurements in mm) [20].

B. Comparison between Results of Nonlinear Static (Pushover)Analysiss and Experimental Tests

The behavior of cold-formed frames, under uniform and cyclic loadings, is evaluated and compared. Figure 2 presents base shear force versus displacement of top of the frame based on the nonlinear static analysis and experimental test results under cyclic and uniform monotonic loading. The comparison between the curves shows that the analytical model is stiffer than experimental test specimens. The main reason for this difference can be attributed to the loose connections between frame elements during loading which were not considered in the model. Connections of experimental cold-formed steel specimens are pinned. These connections would fail under the lateral and cyclic forces during the experiments and this makes the structures to behave softer compared to the numerical model.

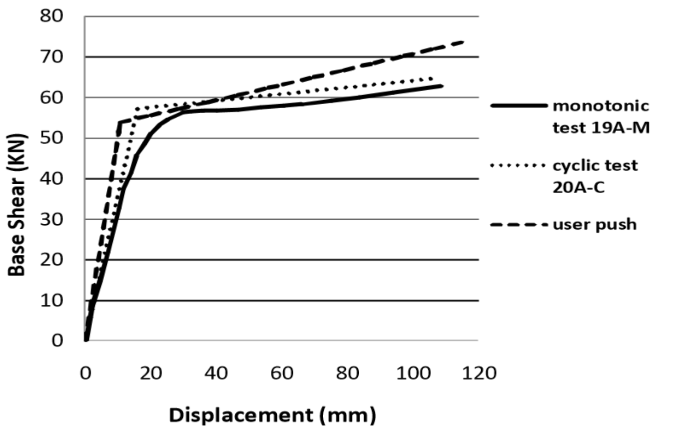

Figure .2. Lateral force–displacement for experimental and analytical results.

In Figure 2, the differences between the experimental results and the model using the CSI SAP 2000 are at most 10%, which is due to the inability of CSI SAP2000 to model the loosing in connections resulted by loading. This difference in the nonlinear part of the diagram is more evident when the structure must carry out large displacement. As a result, the CSI SAP2000 software can be introduced as a suitable tool for

Table .1. Designed elements of structures.

	story	Main studs (340 MPa*)		Middle studs (230 MPa*)		Track (230 MPa*)		Brace (230 MPa*)	
		Double section (mm)	t_p (mm)	Section (mm)	t_p (mm)	Section (mm)	t_p (mm)	Double section (mm)	t_p (mm)
1-story	1	152*41*12.7	1.37	152*41*12.7	1.09	152*31.8	1.37	30	1.09
2-story	1	152*41*12.7	1.37	152*41*12.7	1.09	152*31.8	1.37	55	1.09
	2	152*41*12.7	1.37	152*41*12.7	1.09	152*31.8	1.37	40	1.09
3-story	1	152*41*12.7	1.73	152*41*12.7	1.37	152*31.8	1.37	80	1.09
	2	152*41*12.7	1.37	152*41*12.7	1.09	152*31.8	1.37	70	1.09
	3	152*41*12.7	1.37	152*41*12.7	1.09	152*31.8	1.37	45	1.09
4-story	1	152*41*12.7	2.46	152*41*12.7	1.73	152*31.8	1.37	110	1.09
	2	152*41*12.7	1.73	152*41*12.7	1.37	152*31.8	1.37	95	1.09
	3	152*41*12.7	1.37	152*41*12.7	1.09	152*31.8	1.37	80	1.09
	4	152*41*12.7	1.37	152*41*12.7	1.09	152*31.8	1.37	45	1.09

* Steel yield strength

nonlinear static analysis of cold-formed structures, without considering the failure of connections.

III. MODELING AND DESIGNING MULTI-STORY STRUCTURES AND PERFORMING NONLINEAR STATIC ANALYSIS

In this section, four buildings were modeled and designed according to the plan presented in Figure 3. The buildings with different story number including 1-story, 2-story, 3-story, and 4-story, were considered. The span was assumed to be 2.8 m and the gravity loads were distributed in the directions shown in Figure 3.

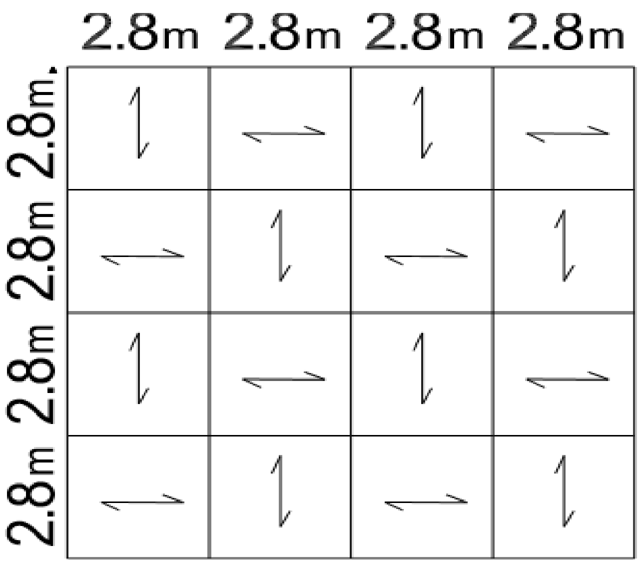

Figure. 3. The plan of investigated buildings.

In this research first, a linear model of the structure was prepared for an elastic analysis and member design. Then nonlinear model is prepared for conducting nonlinear static analyses.

A. Geometry of Numerical Models and Seismic Specifications

Each span has 2.8 m length and 2.8 m height. The studs are located at 40 cm from each other. For middle studs, C- shaped sections are used. For main studs, Double C-shaped sections with a larger thickness are used which are assumed to be bolted back to back of each other. For the tracks, C-shaped sections were utilized. The structures under study in this analysis are three-dimensional buildings that were evaluated for seismic behavior for K level, [10] i.e. life safety acceptance criteria and 10% probability of exceedance in 50 years under an earthquake of BSE-1 according to FEMA356. The structure under study is assumed to be in a high-seismic area. Design PGA of seismic risk level-1 for the assumed construction site is equal to 0.35 g based on the Standard No. 2800. Soil type II based on Standard No. 2800 is considered for the seismic design.

Parameters related to seismic response spectrum of the buildings are equal toT_0=0.1 s, T_S=0.5 s, and S=1.5. Two different steel types are used in this model with the yield strength of 230 MPa and 340 MPa. The dead load and live loads are 250 kg/m^2 and 200 kg/m^2 respectively. In the linear static analysis, earthquake force (basic shear force) is computed using V = CW Equation. According to Standard No. 2800, the effective gravity load of the building during the earthquake is equal to the total dead load with 20% extra contribution of the live load [23]. The response modification factor of cold-formed steel structures based on AISI [12] and FEM450 (NEHRP) [25] is assumed to be equal to three (R = 3). The selected members for design purpose were according to the reference [20], which have been checked according to the design codes for local buckling and other design criteria, such as effective section.

B. Analysis and Design of Structures

All the members were designed in accordance with the output forces from a linear static analysis [1, 24]. Table 1 shows the designed sections for the structure elements. The compressive strength of the bracings has been disregarded due to lack of out of plane stiffness

C. 3-3. Nonlinear Structure Model

Based on FEMA356 [10] at least two types of lateral load pattern should be used with dead and live gravity load combinations. 1.1(Q_D+Q_L) and 0.9Q_D load combinations are considered in which Q_D is the dead load and Q_L is the live load. Two lateral load patterns of modal and triangular are defined. To introduce modal lateral load pattern, the effective modal mass contribution of the structures in mode one in each direction should be determined. If the effective modal mass of

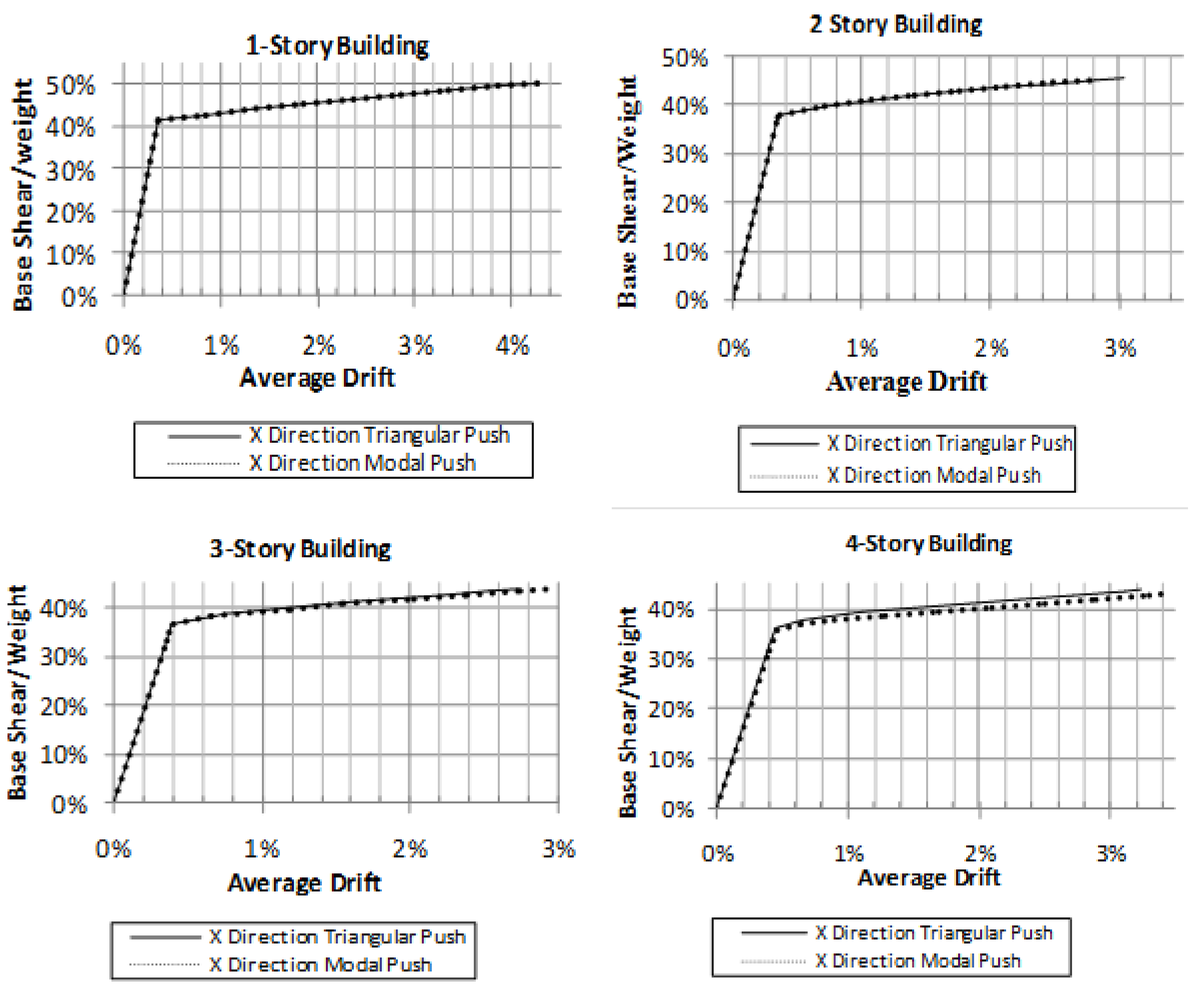

Figure .4. Pushover curves for modal and triangular load pattern for different structures (solid lines represent the triangular pushover, dotted lines represent the modal pushover).

The first mode is less than 75%, it is necessary to consider the effect of higher modes and combination of modes; otherwise, the effect of first mode in each direction is sufficient for the load pattern [10]. To control displacement, the displacements on the top story (roof) and the nearest node to the center of mass are considered. The effective modal mass contributions are presented in Table 2. It is shown that in all the structures, the effective modal mass percentages exceed 75%; therefore, mode one can be considered in lateral load pattern. As a result, in this study, the effect of higher modes is neglected and only the first mode load pattern is used for pushover analysis.

Table .2. Effective modal mass of structures.

Structure	Mode 1 x-direction	Mode 1 y-direction
1-story	1	1
2-story	0.9	0.89
3-story	0.84	0.81
4-story	0.78	0.76

In Figure 4, shear forces normalized to the building weights in term of average floor drifts for both triangular and modal load patterns and then were compared. The plots for two load patterns are very close to each other. The difference between the curves for 1, 2, and 3-story buildings is less than 1%. This difference increases up to 4% for 4-story buildings, due to the contribution of higher modes.

D. Evaluation of Story Drifts at Target Displacement

In this part, target displacements were calculated based on the proposed method by FEMA 356, and then interstory drifts were calculated by considering the results from modal load pattern and presented in Figure 5. In Figure 5, at the target displacements, in all the structures the drifts are less than 1.5%, which indicates performance of the structures is beyond the life safety. In Figure 6, ratio of the roof displacements over the structure heights (normalized roof displacement) are plotted in term of the periods of structures. The figure shows that by increasing story, the normalized roof displacement of cold-formed structures is declining.

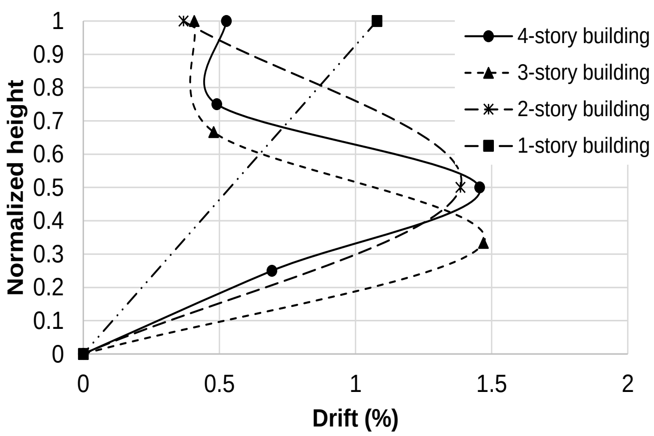

Figure. 5. Drift variation along building heights.

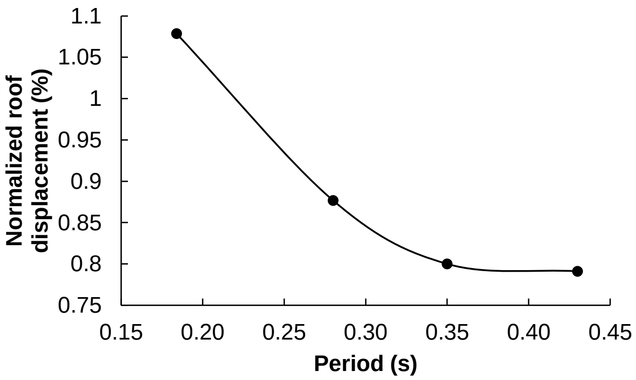

Figure.6. Normalized roof displacement versus periods of buildings.

E. LRFD Response Modification Factor of Structures

The lateral force on structures due to severe earthquakes are much higher than design values recommended by the earthquake design codes. The method of obtaining equivalent static lateral force in most building codes are based on the design method according to response modification factor. This coefficient reduces lateral forces resulting from earthquake. In this methodology, some members of structure are designed to behave as fuses against large earthquakes. The capacity curve for calculating of seismic parameters, can be derived either from incremental nonlinear dynamic analysis [26,27] or nonlinear static analysis [28]. Considering that the area under pushover curves represents amount of energy dissipated by a structure, therefore the more the area under the curve, the more capability of a structure to dissipate energy during an earthquake. The structures can dissipate and absorb portion of earthquake energy due to their inherent damping ratio. Building codes take advantage of this inherent property of structures, and instead of designing a structure for $V_{elastic}$ force and $\Delta_{elastic}$ displacement, they design it for $V_{elastoplastic}$ force and $\Delta_{elastoplastic}$ displacement [29]. In Figure 7, this concept is shown.

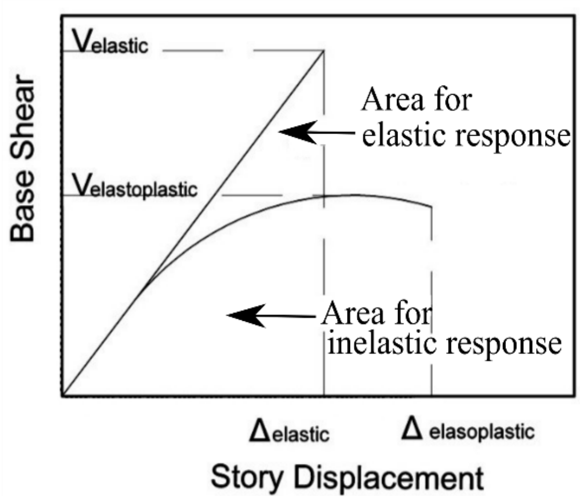

Figure.7. Force-displacement curve in elastic and elastoplastic behavior.

According to this method, in the earthquake codes, structures are designed for a less force, but instead, they should be able to resist larger displacements and be more flexible. According to Figure 8, the response modification factor based on LRFD method is obtained from the Equation 1.

$$R = \frac{C_{eu}}{C_s} \qquad (1)$$

That C_{eu} is the shear elastic coefficient of structure and C_s is a shear force that causes first plastic hinge [16].

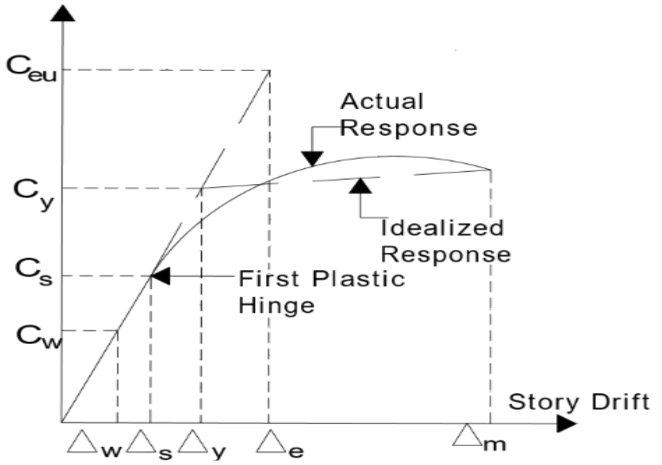

Figure 8. Seismic parameter for design of structures subjected to earthquake load.

Based on Equation 1, the response modification factor for all structures is calculated along x-direction. The values of the response modification factor for structures with 1, 2, 3, 4-story are estimated 5.02, 4.274, 4.021, and 3.86, respectively. The response modification factor which these structures were designed (R = 3) has been conservative and the value of R = 4 seems to be more realistic. Therefore, it can be concluded that a response modification factor proposed by other design codes such ad TI 809-07 code [30] is closer to the obtained results in this paper.

IV. CONCLUSION

In this paper, response modification factors of cold-formed steel structures were calculated, and seismic performance of these structures was investigated. For this purpose, first, the results of the nonlinear analysis of the two-dimensional frames were compared to the experimental results. Three-dimensional structures with 1, 2, 3, and 4-story were modeled and analyzed. Based on the obtained responses from the numerical model the following observations were made:

• By comparing the stiffness in the numerical model and experimental results, it is observed that laboratory specimens have a softer lateral behavior compared to the numerical model. The reason can be attributed to the effect of slight deformations of cold-formed thin-walled sections and loosing effect in the connections. This difference is estimated at a maximum of 10%.

• It is observed that in this kind of structures, failures initially appear in the bracing system. This observation is in accordance with the expectation, which braces behave like a fuse under seismic loading to prevent other structural elements from failure.

• The response modification factor of three (R = 3) was assumed for initial design purpose. After calculating response modification factor using nonlinear static analyses, it was determined that the value of R = 4 (which is recommended in some of the codes such as TI809-07) seems more logical.

• Descending trend of the response modification factors of light cold-formed steel structures by increasing the height of building indicates that the energy absorption capacity in the plastic hinges decreases by increasing the structure height. It can be concluded that light cold-formed steel structures with larger number of story shows a relative brittle behavior under lateral loading.

V. REFERENCES

[1]. Yu, W.W.; LaBoube, R.A., "Cold Formed, Steel Design", John Wiley & Sons .0102, Hoboken, New Jersey

[2]. Adham, S.A.; Avenessian, V.; Hart, G.C.; Anderson, R.W.; Elmilinger, J.; Gegory, J.; "Shear Wall Resistance of Light-Gauge Steel Stud Wall System", Earthquake Spectra, 1990, 6(1), 1-.41

[3]. Kim, T.W.; wikcoski, J.; Foutch, D.A.; Lee, M.S., "Shake table tests of a cold-formed steel shear panel", Engineering Structure, 2006, 28, 1462-70.

[4]. Fulop, L.A.; Dubina, D., "Performance of Wall-Stud Formed Shear Panels under Monotonic and Cyclic Loading. Part I: Experimental Research", Thin-Walled Structures, 2004, 42, 321-.823

[5]. Oskouei, V.A. and Mahdavifar, V., 2013. Modeling of Two-Cell Concrete Cores for Investigation of Reliabality of Equivalent Column Method,

[6]. Gioncu, V. (2000). Framed structures. Ductility and seismic response: General Report. Journal of Constructional Steel Research, 55 (1): 125-154.

[7]. Mahdavifar, V., Barbosa, A., Sinha, A., Muszynski, L. and Gupta, R., 2017. Hysteretic behaviour of metal connectors for hybrid (high-and low-grade mixed species) crosses laminated timber, World Conference on Timber Engineering, Vienna, Austria.

[8]. Mahdavifar, V., Barbosa, A.R. and Sinha, A., 2016. Nonlinear Layered Modelling Approach for Cross Laminated Timber Panels Subjected to Out-of-Plane Loading., 41st IAHS World Congress on Housing, Albufeira, Portugal.

[9]. Mahdavifar, V., 2017. Cyclic performance of connections used in hybrid cross-laminated timber (Doctoral dissertation).

[10]. Hamedi, A., Mahdavifar, V., Sajjadi, S. and Fesharaki, M., 2017. Sensitivity Analysis of Earthquake Acceleration and Drainage Efficiency on the Stability of Weighted Concrete Dams.

[11]. Hamedi, A., Mansoori, A., Shamsai, A., & Amirahmadian, S. 2014. The Effect of End Sill and Stepped Slope on Stepped Spillway Energy Dissipation. Journal of Water Sciences Research, 6 :1-15.

[12].Ramin Taghinezhad; Arash Taghinezhad; Vahid Mahdavifar; Vafa Soltangharaei, Numerical Investigation of Deflection Amplification Factor in Moment Resisting Frames Using Nonlinear Pushover Analysis, International Journal of Innovations in Engineering and Science.

[13]. R Taghinezhadbilondy,2016, Extending Use of Simple for Dead Load and Continuous for Live Load (SDCL) Steel Bridge System to Seismic Areas, Ph.D. dissertation.

[14]. Huy Pham, Ramin Taghinezhad, Atorod Azizinamini, 2017, Experimental Investigation of Redundancy of Twin Steel Box-Girder Bridges under Concentrated Load, Transportation Research Board 96th Annual Meeting Transportation Research Board

[15]. Ramin Taghinezhad, JH Gull, H Pham, LD Olson, A Azizinamini, 2017, Vibration Monitoring During the Deconstruction of a Post-tensioned Segmental Bridge: Case Study, Transportation Research Board 96th Annual Meeting Transportation Research Board.

[16]. Atorod Azizinamini, Aaron Yakel, Ardalan Sherafati, Ramin Taghinezhad, Jawad H Gull,2016, Flexible Pile Head in Jointless Bridges: Design Provisions for H-Piles in Cohesive Soils, Journal of Bridge Engineering

[17]. Gad, E.F.; Duffield, C.F.; Hutchinson, G.L.; Mansell, D.S.; Stark, G., "Lateral Performance of cold-formed steel framed domestic structures", Elsevier Engineering Structures, 1999, 21, 83-95.

[18]. Moghimi, H.; Ronagh, H.R., "Better connection details for strap-braced CFS stud wall in seismic regions", Thin-Walled Structures, 2009, 47, 122-135.

[19]. AISI S100-12; "North American Specification for the Design of Cold-Formed Steel Structural Members", American Iron and Steel Institute, Washington, DC, 2013

[20]. Velchev, K.; Comeau, G.; Balh, N.; Rogers, C.A., "Evaluation of the AISI S213 seismic design procedures through testing of strap braced cold-formed steel walls", Thin-Walled Structures, 2010, 48, 846-856.

[21]. CSI SAP 2000 (version 14). Computers and Structures, Inc., 2014.

[22]. ASCE, FEMA 356, "Pre-standard and commentary for the seismic rehabilitation of buildings", Reston, Virginia, USA, 2000.

[23]. Standard No, 2800-84. Iranian code of practice for seismic resistant design of buildings, 3 , 2005.

[24]. AISI. Standard for Cold-Formed Steel Framing, General Provision, American Iron and Steel Institute, Washington DC. 2004

[25]. BSSC, "NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures", (FEMA 450), Washington DC, 2003.

[26]. Soltangharaei, V., M. Razi, and M. Gerami. "Behaviour factor of buckling restrained braced structures for near and far fault ground motions." International Journal of Structural Engineering 6, no. 2 (2015): 158-171.

[27]. Soltangharaei, Vafa, Morteza Razi, and Mohsen Gerami. "Comparative Evaluation of Behavior Factor of SMRF Structures for Near and Far Fault Ground Motions." Periodica Polytechnica. Civil Engineering 60, no. 1 (2016): 75.

[28]. Mahdi, T., and V. Soltan Gharaie. "Plan irregular RC frames: comparison of pushover with nonlinear dynamic analysis." Asian J Civil Eng Build Housing 12, no. 6 (2011): 679-690.

[29]. Miranda, E; Bertero, Evaluation of Strength Reduction Factors for Earthquake Resistance Design, J Eng Struct. 1994: 2(10), 357-79.

[30]. TI 809-07, "US Army Corps of Engineers-technical Instructions, Engineering and construction division", Directorate of military programs, Washington DC, 20314-1000-30, 1998.