International Journal of Engineering

Journal Homepage: www.ije.ir

Seismic Behavior of Asymmetric Two-Story X-braced Frames

S. S. Seyedjafari Olia, H. Saffari*, A. Fakhraddini

Department of Civil Engineering, Shahid Bahonar University of Kerman, Kerman, Iran

PAPER INFO

Paper history:
Received 29 December 2018
Received in revised form 28 January 2019
Accepted 09 March 2019

Keywords:
Asymmetric Two-Story X-braced Frames
Concentrically Braced Frames
Seismic Performance
Hysteresis Cycle
Ductility

ABSTRACT

Concentrically braced frames (CBFs) are one of the efficient lateral load resisting systems in high seismicity regions. One of the common problems with the use of concentrically braced frames is limitation in the architectural application and position of the openings. Two-story X braced frames have more advantages than other configurations of concentrically braced frames, since in many cases the position of the openings due to the need for architectural spaces and executive imperfections causes the use of asymmetric X-braced frames, present study tries to evaluate the seismic behavior of asymmetric two-story X braces. In this study, the behavior of these braces has been studied. For this purpose, firstly, several symmetric two-story X braced frames are modeled by OpenSees software. Then, by changing the position of braces to beam connection, the new asymmetrical braces are obtained which initially designed. Finally, parameters such as stiffness, strength and stable hysteresis cycle of asymmetric systems are compared with symmetrical braces by nonlinear static and dynamic analysis. The results show that if asymmetric braces are distributed symmetrically in the structure, they do not lose their ability in comparison with the symmetrical models.

***doi**: 10.5829/ije.2019.32.04a.06*

1. INTRODUCTION

Concentrically braced frames (CBFs) are commonly used in steel structures to resist lateral forces due to wind or earthquakes. They provide vertical concentric truss system, which the axes of its members aligning concentrically at the joints. They can also provide high strength and stiffness and acceptable seismic response [1].

In general, The CBFs are economical system to use for low-rise structures in areas of high seismicity. They are usually preferred over moment frames due to their material efficiency and the smaller required size of beam and column.

Researchers and engineers have strived to improve the performance of CBFs. Khatib et al. [2] conducted a comprehensive study of the buckling and post-buckling behavior of braces. Typically, the compressive strength of the braces is less than their tensile strength; accordingly, a large unbalanced vertical force is generated in the middle of chevron (V or inverted V) configuration. Khatib suggested that zipper columns are added in chevron braced steel frames in order to achieve a more uniform distribution of the inelastic deformations and moderating unbalanced force in these structures. Another solution is the use of bracing members which enhanced hysteretic behaviour such as buckling restrained braces (BRBs). BRBs first introduced by Yoshino and Karino [3], Amiri et al. [4], which significantly improved the hysteresis behavior, increased compressive strength, and reduced the unbalanced forces of the beams in these braces. Another idea was the use of Self-Centering Concentrically Braced Frame, which was introduced by Sause et al. [5]. They evaluated several concentrically steel braced frames. Uriz and Mahin [6], Lai and Mahin [7] conducted studies to complement and improve the behavior of braces with zipper columns and combine this type of system with BRBs to prevent the formation of a weak floor. They also carried out investigations by changing the position of braces to beam connection on a variety of braces. Guo [8] presented a new form of buckling restrained bracing system which uses two distinct steel cores [9].

Two-story X braced frames are a combination of V

*Corresponding Author Email: *hsaffari@uk.ac.ir* (H. Saffari)

Please cite this article as: S. S. Seyedjafari Olia, H. Saffari, A. Fakhraddini, Seismic Behavior of Asymmetric Two-Story X-braced Frames, International Journal of Engineering (IJE), IJE TRANSACTIONS A: Basics Vol. 32, No. 4, (April 2019) 495-502

and inverted V braces configurations. This type of brace compared to the V and inverted V has less unbalanced force, which reduces the section size of the beam [10]. Because of architectural requirements, the use of asymmetric two-story X braced frame seems essential. Wang et al. [11] studied the behavior of asymmetric chevron concentrically braced frames.

Since in many cases the position of the openings due to the need for architectural spaces and executive imperfections causes the use of asymmetric X-braced frames, present study tries to evaluate the seismic behavior of asymmetric two-story X braces.The aim of the present study is to evaluate the effect of asymmetry on the seismic behavior of two-story X braced frames, to do this, three frames are studied under pushover and nonlinear dynamic analyses. The results show that two-story X braced frames with asymmetric bracing members not only did not have a significant drop rather than symmetrical structures, but also in some cases they have more strength and stiffness and thus give less drift responses.

2. FRAMEWORK OF THE STUDY

The effects of cyclic loading on the braces include some phenomena such as inelastic buckling, yielding, local buckling, post-buckling residual strength, Bauschinger effects, and strain hardening in tension [12]. These phenomena create a significant demand for other members and connections of braced frames. Special concentrically braced frames should tolerate plastic deformation and absorb hysteresis energy with stable behavior during successive cycles of tensile yielding and inelastic buckling [13, 14]. The design philosophy of special concentrically braced frames is such that plastic deformation takes place only in the braces and other structural parts such as columns and beams remain essentially elastic [15].

As previously mentioned, Two-story X Braced Frames (TXBF) are a combination of braces with V and inverted V configurations as shown in Figure 1. This type of brace in comparison with the V and inverted V has less unbalanced force which reduces the section size of the beam [16]. Asymmetric Two-story X Braced Frame (ATXBF) is created by changing the position of brace to beam connection (Figure 2).

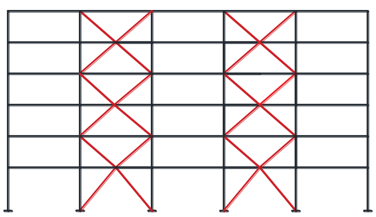

Figure 1. Symmetrical two-story X brace

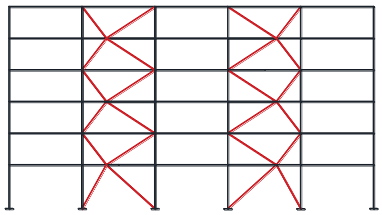

Figure 2. Asymmetrical two-story X brace

2. 1. Structural Models

The TXBFs selected in this study consist of regular 2, 4, and 6-story CBFs. where 6-story CBF were previously studied by Lai and Mahin [7], the others have been designed and Member stress checks were performed in SAP2000 using the load combinations listed in ASCE-07 [17]. In the design, tried to approximate the stress ratio in the corresponding members of each structure. The 2-story frame has 7 bays of 20 feet and the height of the floors are 13 feet (Figure 3). The 4 and 6-story frames have 5 bays of 30 feet (Figure 4) so that the height of the first floor is 18 feet and other floor heights are 13 feet. In all frames, the brace-to-beam and the beam-to-column connections are pined. Rigid end zones were applied at member ends based on the actual member sizes in the models.

The position of brace to beam connection is shown by "e" in Figure 5, five values, 0.5, 0.58, 0.67, 0.75 and 0.83 for parameter e/L, were specified in the design phase. A total of 15 double-span frames is considered.

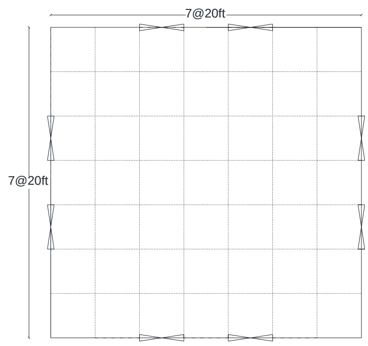

Figure 3. Two story structure plan

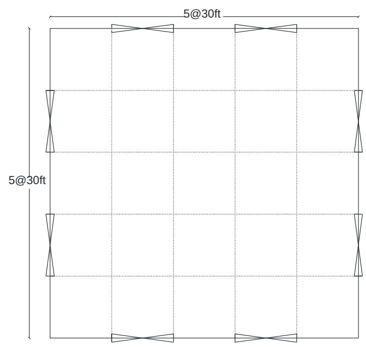

Figure 4. Four and six story structures plan

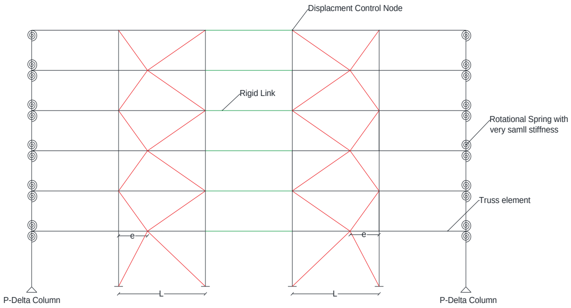

Figure 5. Two-dimensional Frame Modeled By OpenSees Software

The dead load and live load of all floors are 100 and 50 psi, respecively. The earthquake loading has been determined according to the ASCE-07. Design of these braced frame systems basically follows the AISC [18] Seismic Provisions. Seismic design parameters are listed in Table 1. The section sizes of these TXBFs are shown in Table 2.

TABLE 1. Seismic design parameters for CBFs

Parameters	**Values**
Spectral response acceleration at short period, S_S	2.014 g
Spectral response acceleration at period of 1.0 s, S_1	0.787 g
Acceleration site coefficient, F_a	1.0
Velocity site coefficient, F_v	1.5
Design spectral response acceleration in the short period range, S_{DS}	1.343 g
Design spectral response acceleration at a period of 1.0 s, S_{D1}	0.787 g
Site class	D
Seismic design category	D
Response modification factor	R = 6
Deflection amplification factor	C_d =5
Overstrength factor	Ω_0=2
Importance factor	I = 1.0

TABLE 2. Section sizes of the CBFs

n_s	**Model (e/L)**	**Floor Level**	**Column***	**Beam***	**Brace****	
					Left	**Right**
2-Story	0.5	2	12x50	14x38	5x5x1/2	5x5x1/2
		1	12x50	14x38	6x6x5/8	6x6x5/8
	0.58	2	12x50	14x38	6x6x5/16	5x5x3/8
		1	12x50	14x38	7x7x1/2	6x6x5/8
	0.67	2	12x50	14x38	6x6x3/8	4x4x5/8
		1	12x50	14x38	7x7x1/2	6x6x5/8
	0.75	2	12x50	14x38	6x6x5/8	4x4x1/4
		1	12x50	14x38	7x7x5/8	6x6x5/8
	0.83	2	12x50	14x38	7x7x3/8	4x4x3/16
		1	12x50	14x38	8x8x1/2	6x6x1/2
4-Story	0.5	4	12x45	16x89	6x6x1/2	6x6x1/2
		3	12x45	16x89	7x7x1/2	7x7x1/2
		2	12x170	16x89	9x9x1/2	9x9x1/2
		1	12x170	16x89	10x10x5/8	10x10x5/8
	0.58	4	12x45	16x89	7x7x1/2	5x5x5/8
		3	12x45	16x89	8x8x1/2	7x7x1/2
		2	12x170	16x89	10x10x1/2	8x8x1/2
		1	12x170	16x89	12x12x1/2	10x10x5/8
	0.67	4	12x45	16x89	7x7x5/8	5x5x1/2
		3	12x45	16x89	8x8x5/8	6x6x5/8
		2	12x170	16x89	10x10x5/8	7x7x1/2
		1	12x170	16x89	12x12x5/8	10x10x5/8
	0.75	4	12x45	16x89	8x8x1/2	4x4x5/8
		3	12x45	16x89	9x9x1/2	6x6x1/2
		2	12x170	16x89	12x12x1/2	6x6x1/2
		1	12x170	16x89	12x12x5/8	10x10x5/8
	0.83	4	12x45	16x89	8x8x5/8	4x4x1/2
		3	12x45	16x89	10x10x1/2	6x6x1/2
		2	12x170	16x89	12x12x5/8	6x6x3/16
		1	12x170	16x89	14x14x1/2	10x10x5/8
6-Story	0.5	6	14x132	18x86	6x6x1/2	6x6x1/2
		5	14x132	18x86	7x7x1/2	7x7x1/2
		4	14x132	18x86	8x8x1/2	8x8x1/2
		3	14x342	18x86	8x8x1/2	8x8x1/2
		2	14x342	18x86	9x9x5/8	9x9x5/8
		1	14x342	18x86	10x10x5/8	10x10x5/8
	0.58	6	14x132	18x86	6x6x5/8	5x5x5/8
		5	14x132	18x86	7x7x5/8	6x6x5/8
		4	14x132	18x86	9x9x1/2	8x8x1/2
		3	14x342	18x86	9x9x1/2	8x8x1/2
		2	14x342	18x86	10x10x1/2	9x9x5/8
		1	14x342	18x86	12x12x5/8	10x10x5/8
	0.67	6	14x132	18x86	7x7x1/2	5x5x3/8
		5	14x132	18x86	8x8x1/2	6x6x1/2
		4	14x132	18x86	9x9x5/8	8x8x3/8
		3	14x342	18x86	9x9x5/8	8x8x1/2
		2	14x342	18x86	10x10x5/8	9x9x5/8
		1	14x342	18x86	12x12x5/8	9x9x5/8
	0.75	6	14x132	18x86	8x8x1/2	4x4x1/2
		5	14x132	18x86	9x9x1/2	6x6x1/2
		4	14x132	18x86	10x10x5/8	6x6x61/2
		3	14x342	18x86	10x10x5/8	7x7x5/8
		2	14x342	18x86	12x12x5/8	7x7x5/8
		1	14x342	18x86	12x12x5/8	10x10x5/8
	0.83	6	14x132	18x86	8x8x1/2	4x4x1/2
		5	14x132	18x86	9x9x5/8	5x5x1/2
		4	14x132	18x86	12x12x1/2	5x5x1/2
		3	14x342	18x86	12x12x1/2	8x8x1/2
		2	14x342	18x86	14x14x1/2	8x8x3/8
		1	14x342	18x86	14x14x5/8	10x10x5/8

* These elements, are W-type. ** These elements, are HSS-type.

2. 2. Finite Element Modeling and Nonlinear Behavior Due to symmetry in plan and simplicity of analysis, the studied bracing structures have been modeled by OpenSees as two-dimensional frames (Figure 5). One of the main requirements of nonlinear analysis is the definition of non-linear responses of members. Members of the CBFs are designed such that non-linear behavior concentrates on brace member, while other members remain linear elastic [6, 19]. A Rayleigh damping parameter of 2% was used for both first and second vibration mode.

In this research, for beams and columns, I-standard sections ASTM A992 with a yield stress of Fy = 50 ksi and for braces using a standard section of ASTM A500 Grade B with a yield stress of Fy = 46 ksi and an elastic modulus E = 29,000 ksi were used. This material is introduced by Steel02 [20] with a strain hardening ratio of b = 0.003 and R = 25 in OpenSees [7]. In order to consider the nonlinear behavior of the structural members, a "forceBeamColumn" element with fiber section has been used. The number of fibers of each section is shown in Figure 6.

An initial imperfection in the middle of the braces is assumed with a factor of 1/500 of the member length [21]. The shape of the initial imperfection in accordance with the first buckling mode of the brace is sinusoid as following relation. Each brace is divided into four parts, as shown in Figure 7.

The vertical floor mass tributary to the braces intersecting a beam or column was included in the models. In any stability analysis, it is essential to capture the destabilizing influences of columns that rely on the lateral frame for stability but are not a portion of the lateral frame. These columns with pinned ends are normally referred to as "leaning columns". P-Δ effects were represented using a leaning column. The leaning column was constrained to have the same lateral displacement as the most adjacent column at a level in the braced bay. The axial and flexural stiffness of the columns are assumed to be large, but a pin was introduced at the bottom of the column in each story [22]. These columns are shown in Figure 5.

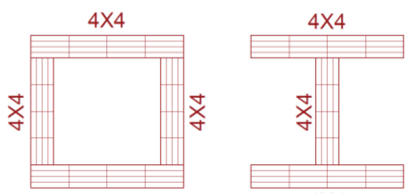

Figure 6. Number of fibers in a section

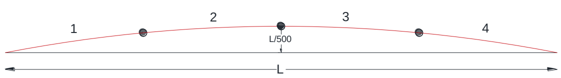

Figure 7. The value of initial imperfection and number elements

2. 3. Ground Motion Ensemble A total of ten ground motions (two components from five records) were considered for the dynamic analysis in OpenSees. These ground motions were previously used by Lai and Mahin [7]. Each record contained two pairs of ground motions, representing the fault-normal and fault-parallel components. This results in ten excitations being considered for the 2D model analyzed. Vertical components of ground motions were not included in this study. The earthquake ground motions which are scaled in accordance with the ASCE-07 have been selected, and listed in Table 3. Response acceleration spectra and the average response spectrum are depicted in Figure 8. The important objective in the selection of these causative records is their magnitudes which ranges from 5-7.5; also, the type of fault is Strike Slip and the scale factors of the ground motions are limited to be less than three. The basic parameters of the records are summarized in Table 3.

3. RESULTS OF NONLINEAR ANALYSES AND DISCUSSION

All models were subjected to pushover and nonlinear dynamic analyses by means of OpenSees. In the following, the results of nonlinear analyses are discussed.

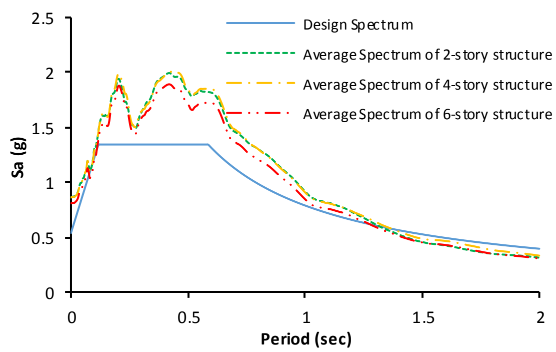

Figure 8. Average Response spectra Of Different Structures

TABLE 3. Selected ground motion pairs for nonlinear dynamic response history analysis

RSN No.	Event	Year	Magnitude	*V*S30 (m/sec)	*R*rup (km)	Scale Factor		
						2-Story	4-Story	6-Story
160	Imperial Valley-06	1979	6.53	223.03	2.66	0.881	0.912	0.984
558	Chalfant Valley-02	1986	6.19	316.19	7.58	1.438	1.440	1.494
1119	Kobe-Japan	1995	6.9	312.0	0.27	0.999	1.449	1.215
1602	Duzce-Turkey	1999	7.14	293.57	12.04	1.821	1.448	1.415
1853	Yountville	2000	5.0	328.57	11.5	2.011	1.882	1.496

3. 1. Results of Pushover Analysis All models were subjected to a non-linear static analysis in a positive direction up to 5% of the roof height. The lateral load pattern is according to ASCE 41 [23] as follows.

$$F_x = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} V \tag{1}$$

Figures 9, 10 and 11 show the pushover curves of frames which the horizontal axis is the roof displacement and the vertical axis is the base shear force. Tables 4 to 6 list the results of the pushover analysis which have compared with the symmetric model. According to Figures 9, 12 and Table 4, in all asymmetric 2-story frames, the strength and stiffness decreased in comparison with symmetric 2-story frame. The maximum strength drop in asymmetric model rather than symmetric model is about 22%. But the highest residual strength in 2% and 4% story drifts is belong to the asymmetric model with e/L=0.75. In fact, which has an increase of 12.1% in weight.

In the 4-story frames, according to Table 5, the highest stiffness and residual strength is occurred in the asymmetric model with e/L=0.67, which has increase of 2 0.9% in residual strength in 2% drift and 25.7% in 4% drift rather than the symmetric model. Models with e/L=0.58 and e/L=0.75 have 0.58% strength drop compared with the symmetric model, but in 2% and 4% drifts, increased residual strength of up to 22.15% rather than model e/L =0.5 (Figures 11 and 13).

As shown in Figures 11, 14 and Table 6, the most stiffness and post-buckling strength is occurred in the asymmetric model with e/L=0.75, which has 15.1% increase in post-buckling strength than the symmetric model. As can be seen, in the 6-story structure with increasing e/L, in addition to increasing the weight of the structure, the stiffness and strength increased up to e/L=0.75 but they decreased in e/L=0.83. This shows that, with excessive increase in the length of the brace, the slenderness factor (KL/r) increases and decreases compressive strength.

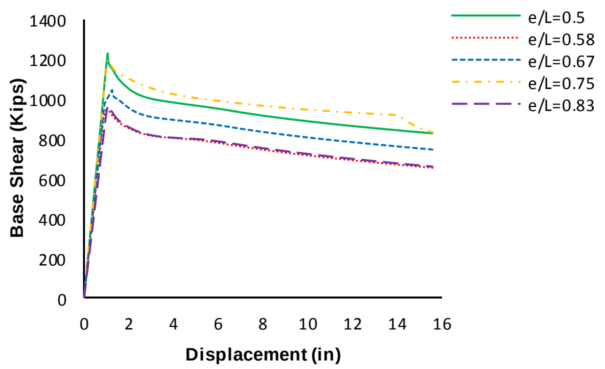

Figure 9. Pushover curves of 2-story structure

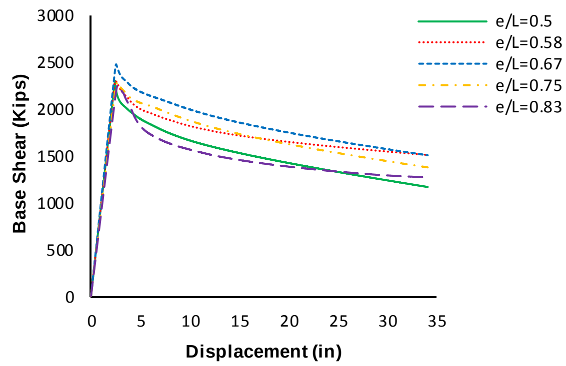

Figure 10. Pushover curves of 4-story structure

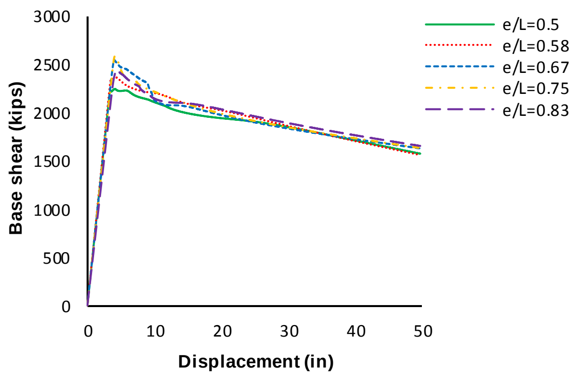

Figure 11. Pushover curves of 6-story structure

TABLE 4. Results of pushover analysis of 2-story structure compared with symmetrical structure (%)

Model (e/L)	Period	Weight	Strength	PBR*	PBR**
0.58	5.78	-8.01	-22.04	-18.16	-20.03
0.67	4.54	0.27	-15.02	-8.73	-9.44
0.75	0.73	12.08	-2.82	4.24	8.09
0.83	11.97	-0.32	-21.67	-17.32	-19.24

*Post-Buckling Resistance in 2% Drift ** Post-Buckling Resistance in 4% Drift

TABLE 5. Results pushover analysis of 4-story structure compared with symmetrical structure (%)

Model (e/L)	Period	Weight	Strength	PBR*	PBR**
0.58	0.03	4.36	-0.13	11.30	22.15
0.67	-2.01	17.34	7.60	20.90	25.67
0.75	3.01	17.00	-0.58	13.09	15.83
0.83	6.01	26.96	-2.01	-5.30	1.95

*Post-Buckling Resistance in 2% Drift ** Post-Buckling Resistance in 4% Drift

TABLE 6. Results pushover analysis of 6-story structure compared with symmetrical structure (%)

Model (e/L)	Period	Weight	Strength	PBR*	PBR**
0.58	-1.14	7.75	5.36	4.19	-0.41
0.67	-0.02	10.50	13.64	1.73	0.72
0.75	0.91	22.35	15.11	2.51	1.23
0.83	5.12	30.03	8.64	4.76	3.07

*Post-Buckling Resistance in 2% Drift ** Post-Buckling Resistance in 4% Drift

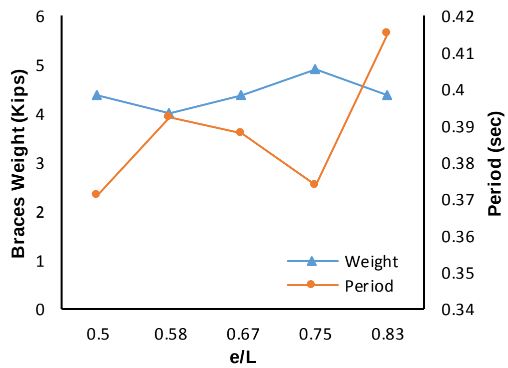

Figure 12. Braces weight and period of 2-story structure

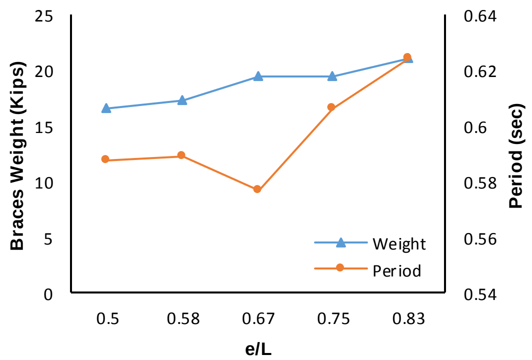

Figure 13. Braces weight and period of 4-story structure

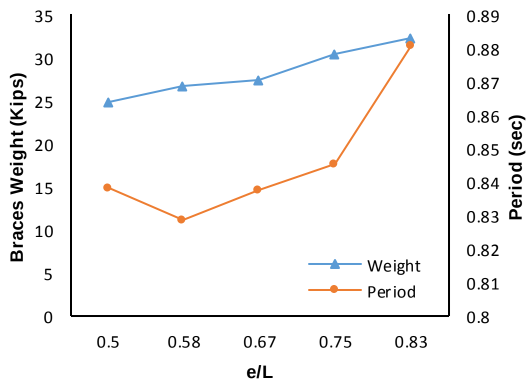

Figure 14. Braces weight and period of 6-story structure

The results shows that by increasing the number of story, the stiffness and strength of asymmetric frames increase in compared with symmetric frames; furthermore, pushover curves of models are closed together.

3. 2. Results of Nonlinear Dynamic Analysis

A total of 15 models were subjected to nonlinear dynamic analysis and the maximum drifts were obtained under existing earthquakes. In Figures 15 to 17, the maximum inter-story drifts resulting from nonlinear dynamic analysis are shown. Table 7 shows the maximum story drift values of floors in a decreasing or incremental proportion to the symmetric model.

As shown in Figure 15 and Table 7, in the 2-story frame, the minimum drift is belong to the model with e/L=0.75 symmetric model. Other asymmetric models have a maximum increase of 33.43% compared with the symmetric model.

In the 4-story frame, according to Figure 16 the minimum drift is belong to the model e/L=0.58 which has a 7% decrease in drift compared with the symmetric model. The rest of the asymmetric models have up to 9.82% increase in drifts compared to the symmetric model.

In the 6-story frame, in all asymmetric models, except for the model e/L=0.58, the maximum reduction in drifts of the symmetric model is observed. The minimum drift is belong to the model e/L=0.75 which has a reduction of 32.87% drift rather than the symmetric model (Figure 17).

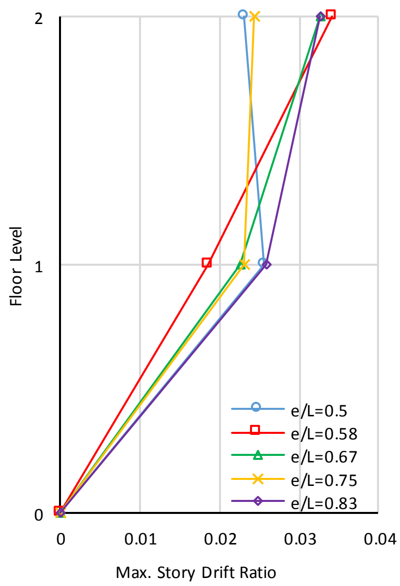

Figure 15. Maximum story drift ratio of 2-story structure

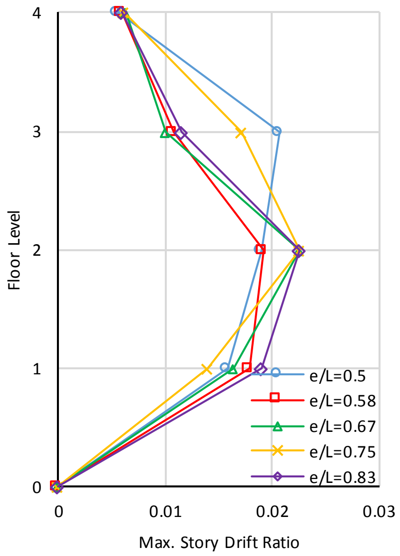

Figure 16. Maximum story drift ratio of 4-story structure

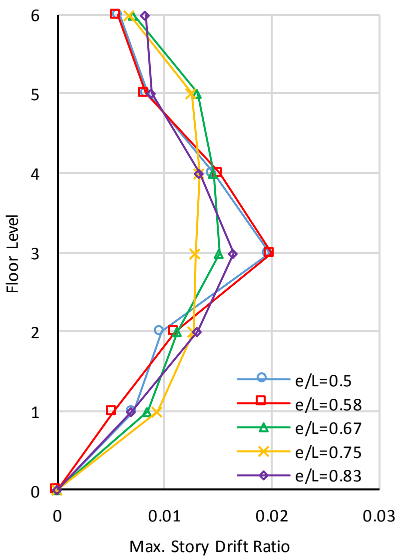

Figure 17. Maximum story drift ratio of 6-story structure

TABLE 7. Maximum drift ratio of different structures compared with symmetrical structure (%)

Structure	e/L=0.58	e/L=0.67	e/L=0.75	e/L=0.83
2-Story	33.43	27.87	-4.36	27.31
4-Story	-7.03	9.82	9.08	9.22
6-Story	0.65	-23.83	-32.87	-17.40

4. CONCLUSION

In this study, structures with 2, 4 and 6 stories with two braced bays were studied by an asymmetric two-story X braced system. Parameters such as strength, buckling and drift were studied by nonlinear static and dynamic analyzes and compered with their symmetric models.

Results show that not only the strength of the asymmetric models did not show a significant drop in comparison with symmetric model but also in the 4-story and 6-story structures, strength increased. Nonlinear dynamical analysis showed that the drifts in asymmetric structures not only does not increase, but also in some asymmetric models, have the lowest drifts than the symmetric model. Results indicate that if asymmetric braces are symmetrically distributed in the structure, they not only do not have a significant drop in stiffness and strength than symmetrical braces, but also in some cases strength increase about 15% and the drifts of structural structures decrease about 30%. Based on the obtaind results, when the e/L value exceeds 0.75, the structure stiffness has dropped and drift increased.

In the implementation of structures often occurs imperfections that are due to lack of coordination between the architectural and structural parts. The use of asymmetric two-X braces can cover these defects without causing problems in structural design and performance.

5. REFERENCES

1. Yamanouchi, H., Midorikawa, M., Nishiyama, I., and Watabe, M., "Seismic Behavior of Full- Scale Concentrically Braced Steel Building Structure", ***Journal of Structural Engineering***, Vol. 115, No. 8, (1989), 1917–1929.
2. Khatib, I., Mahin, S., and Pister, K., "Seismic behavior of concentrically braced steel frames", UCB/EERC-88/01, Earthquake Engineering Research Center, University of California, Berkeley, CA, (1988).
3. Yoshino, T., and Karino, Y., "Experimental study on shear wall with braces: Part 2", In Summaries of technical papers of annual meeting, Vol. 11, Architectural Institute of Japan, (1971), 403–404.
4. Amiri, J. V., Goltabar, A. R. M., and Seifabadi, H. S., "Effect of the Height Increasing on Steel Buildings Retrofitted by Buckling Restrained Bracing Systems and TTD Damper", ***International Journal of Engineering - Transactions A: Basics***, Vol. 26, No. 10, (2013), 1145–1154.
5. Sause, R., Ricles, J.M., Roke, D., Seo, C.Y., and Lee K.S., "Design of self-centering steel concentrically-braced frames", In Proceedings from the 4th International Conference on Earthquake Engineering, Taiwan, (2006).
6. Uriz, P., and Mahin, S. A., "Towards earthquake resistant design of concentrically braced steel structures", PEER Report 2008/08, Pacific Earthquake Engineering Research Center, University of California, Berkeley, CA, (2008).
7. Lai, J.W., and Mahin, S.A., "Experimental and analytical studies on the seismic behavior of conventional and hybrid braced frames", PEER Report 2013/20, Pacific Earthquake Engineering Research Center, University of California, Berkeley, CA, (2013).
8. Guo, Y.L., Zhang, B.H., Zhu, B.L., Zhou, P., Zhang, Y.H., and Tong, J.Z., "Theoretical and experimental studies of battened buckling-restrained braces", ***Engineering Structures***, Vol. 136, (2017), 312–328.
9. Mohebkhah, A., and Farahani, S., "Seismic Behavior of Direct

Displacement-based Designed Eccentrically Braced Frames", ***International Journal of Engineering - Transactions C: Aspects***, Vol. 29, No. 6, (2016), 752–761.

10. Shen, J., Wen, R., and Akbas, B., "Mechanisms in two-story X-braced frames", ***Journal of Constructional Steel Research***, Vol. 106, (2015), 258–277.
11. Wang, J., Dai, K., Yin, Y., and Tesfamariam, S., "Seismic performance-based design and risk analysis of thermal power plant building with consideration of vertical and mass irregularities", ***Engineering Structures***, Vol. 164, (2018), 141–154.
12. Saffari, H., Damroodi, M., and Fakhraddini, A., "Assesment of Seismic Performance of Eccentrically Braced Frame with Vertical Members", ***Asian Journal of Civil Engineering (Building and Housing)***, Vol. 18, No. 2, (2017), 255–269.
13. Lee, K., and Bruneau, M., "Energy dissipation of compression members in concentrically braced frames: Review of experimental data", ***Journal of structural engineering***, Vol. 131, No. 4, (2005), 552–559.
14. Chen, C. H., and Mahin, S., "Seismic collapse performance of concentrically steel braced frames", In Proceedings of the Structures Congress and 19th Analysis and Computation Specialty Conference, Orlando, Florida., (2010).
15. Shaback, B., and Brown, T., "Behaviour of square hollow structural steel braces with end connections under reversed cyclic axial loading", ***Canadian Journal of Civil Engineering***, Vol. 30, No. 4, (2003), 745–753.
16. Shen, J., Seker, O., Sutchiewcharn, N., and Akbas, B., "Cyclic behavior of buckling-controlled braces", ***Journal of Constructional Steel Research***, Vol. 121, (2016), 110–125.
17. Minimum Design Loads for Buildings and Other Structures. ASCE 7-10. American Society of Civil Engineers, Reston, VA, (2010).
18. Specifications for Structural Steel Buildings. AISC 360-10. American Institute of Steel Construction, Inc. Chicago, IL, (2010).
19. Hsiao, P.C., Lehman, D. E., and Roeder, C.W., "Improved analytical model for special concentrically braced frames", ***Journal of Constructional Steel Research***, Vol. 73, (2012), 80–94.
20. Mazzoni, S., McKenna, F., Scott, M.H., and Fenves, G.L., "OpenSees command language manual", Pacific Earthquake Engineering Research Center, University of California, Berkeley, CA, (2006).
21. Ghanaat, Y., "Study of X-braced steel frame structures under earthquake simulation", Report No. UCB/EERC-80/08, Earthquake Engineering Research Center, University of California, Berkeley, CA, (1980).
22. Fakhraddini, A., Fadaee, M. J., and Saffari, H., "A lateral load pattern based on energy evaluation for eccentrically braced frames", ***Steel and Composite Structures***, Vol. 27, No. 5, (2018), 623–632.
23. Seismic rehabilitation of existing buildings. ASCE/SEI 41-13. American Society of Civil Engineers, Reston, VA, (2013).

Seismic Behavior of Asymmetric Two-Story X-braced Frames

S. S. Seyedjafari Olia, H. Saffari, A. Fakhraddini

Department of Civil Engineering, Shahid Bahonar University of Kerman, Kerman, Iran

P A P E R I N F O

Paper history:
Received 29 December 2018
Received in revised form 28 January 2019
Accepted 09 March 2019

Keywords:
Asymmetric Two-Story X-braced Frames
Concentrically Braced Frames
Seismic Performance
Hysteresis Cycle
Ductility

چکیده

قاب‌های مهاربندی شده همگرا یکی از سیستم‌های موثر در برابر بارهای جانبی در مناطق با لرزه خیزی بالا محسوب می‌شوند. یکی از مهمترین مشکلات استفاده از بادبندهای همگرا، محدودیت در فضای معماری و بازشوها می‌باشد. از آنجا که در بسیاری از موارد موقعیت بازشوها به علت نیاز معماری از یک سو و نقص‌های اجرایی از سوی دیگر باعث لزوم استفاده از بادبندهای X شکل نامتقارن می‌گردد. در مقاله حاضر رفتار این بادبندها مورد بررسی قرار گرفته است. بدین منظور، ابتدا نمونه‌هایی از مهاربندهای X دو طبقه متقارن توسط نرم‌افزار OpenSees مدل‌سازی شده است. سپس با تغییر موقعیت گره میانی از مرکز تیر این بادبندها به شکل نامتقارن درآمده‌اند. بادبند نامتقارن جدید طراحی شده و تحت آنالیز استاتیکی و دینامیکی غیرخطی قرار گرفته‌اند و در رابطه با سختی، مقاومت و چرخه هیسترزیس پایدار در مقایسه با بادبندهای متقارن بحث شده است. نتایج بدست آمده نشان می‌دهد که بادبندهای نامتقارن افت چشمگیری در پارامترهای مذکور نسبت به بادبندهای متقارن ندارند.

***doi**: 10.5829/ije.2019.32.04a.06*