

## ASSESSMENT OF BEHAVIOR FACTOR OF ECCENTRICALLY BRACED FRAME WITH VERTICAL LINK IN CYCLIC LOADING

H. Noorinekoo

M.S in structural engineering Responsibility: Design Manager Company: fara sazeh Omid

City: Teheran-Iran

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### ABSTRACT

Use of various Eccentrically Braced Frames for designing or retrofitting buildings and bridges is increasing daily. Thus, much research has been done or is being done in this area. In this study, a new type of these frame with vertical link is investigated. At first, Eccentrically Braced Frame with double vertical link (V-EBF) is introduced and then the parameters affecting links' length selection are proposed to obtain shear behavior and finally its design method will be explained. To investigate the seismic behavior of this system, nonlinear analyses (material and geometry) under a one-way cyclic load have been used. Then, after ensuring the proper behavior of these frames, the effect of number of floors is also studied. In order to understand the behavior of these structures using non-linear static and dynamic analysis of building's behavior factor, eccentric and exocentric systems were calculated and compared with each other. To investigate the seismic behavior of this system, non-linear analyses under one-way cyclic load are used. Then, after ensuring the proper behavior of these frames, the effect of length and number of stiffeners were studied. The results showed that all frames under shear behavior have adequate ductility and stiffness in addition to stability. But some cases including shorter links and web stiffeners have shown better performance.

**Keywords:** Eccentrically Braced Frames, vertical joint, seismic behavior, behavior factor

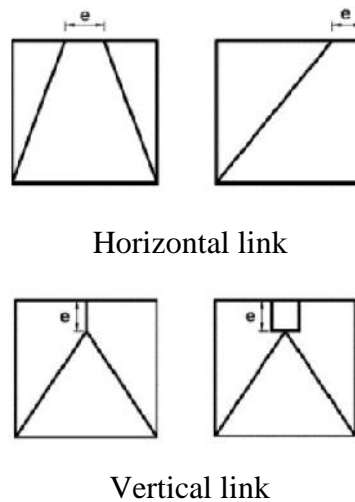
Author Correspondence, e-mail: [author@gmail.com](mailto:author@gmail.com)

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## 1. INTRODUCTION

Around 1970 in Japan, the term "eccentric systems" was first proposed by people such as "Fujimoto" et al. [1] in 1972 and Tanabashi et al. [2] in 1974. In America, Eccentrically Braced system was first tested by Professor "Popov" [3] in 1978 at the University of California, Berkeley. Reviews and studies on the eccentric braced frames (EBFs) were widely began in 70s. Popov et al. stated that EBFs in comparison with other lateral loader systems (such as MRF and CBF) satisfy two seismic design criteria, namely stiffness and ductility. These frames show high stiffness under mild and moderate earthquake and good ductility under severe earthquakes (Configuration of this system is shown in Figure 1). Eccentricity, that is known as "link" in these frames, is displayed by parameter  $e$ . Link is the weakest part of frame and the main cause of energy dissipation arisen from earthquake in this area.



**Fig.1.** Eccentrically Braced Frames [4]

In addition to "Popov" who studied this field since 70s, a study by "Itani" in 2002 was performed based on the nonlinear finite elements analysis results. Then, in 2004, further studies by Dosika et al. was conducted according to the AISC Seismic Standards. In addition, shear link beams with ASTM A992 steel and a yield point of 340 to 400 MPa were tested by Aras et al. in 2003 in order to examine the wing's thinness restrictions and to estimate increasing resistance coefficients, ultimately, the values of AISC 2002 Regulations were approved. The failure of some cases was due to rupture of the hard-link web beam that was started from web stiffener [4]. In 2006, Chao adjusted structural computational models to study the web failure for judging the failure of ductile steel properties based on the test results in order to better determine the location of ductile failure. Bremen and Bruno, in 2007, investigated link beam in eccentric braced frames numerically and experimentally and

obtained the deformation angle 0.15 Radian, an amount nearly double the required value in regulations related to wide flange profiles for the link beam. They also showed that these sections can well achieve high levels of ductility. Chan, in 2008, performed one-way and cyclic loading tests on shear link beam in the form of thin steel plates welded inside a piece. Due to its high energy dissipation, the application will be recommended for seismic rehabilitation of existing frameworks. In this thesis, the effect of using this reliable system on increasing the ductility and energy dissipation applied numerically is investigated using common profiles in the country; so that the utility of this system for designing and retrofitting steel buildings against earthquakes in Iran will be shown.

Since 2009, extensive research was carried out by the Center for Building and Housing Research and International Institute of Earthquake as well as a group of professors from the University of Science and Technology on eccentric bracing with vertical link in two forms of with and without stiffener. [6,5]

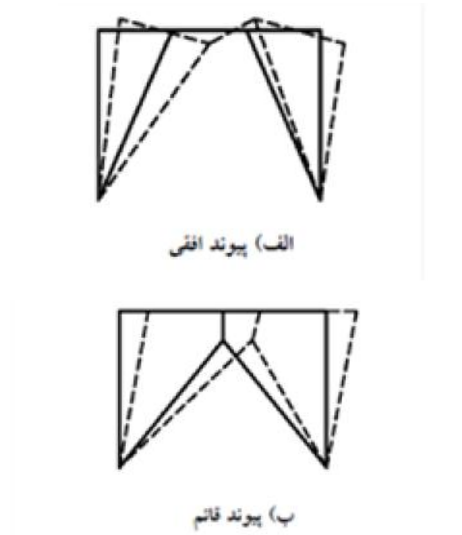
## 2. The advantages of vertical link to horizontal links

As shown in Figure (2), it is evident that placement of link is both horizontal and vertical. So, there are two general types of eccentric frame with horizontal link (H-EBF) and vertical link (V-EBF). In general, the advantages of vertical than horizontal link is:

- Transfer of nonlinear deformation to the outside floor beams and energy dissipation only in vertical link.
- Vertical link is designed only for lateral forces, because the configuration of vertical link frame is such that an insignificant force is transferred from gravity loads to vertical links.
- Simple replacement of vertical link after earthquake, because it is outside the base loading system.
- Application in retrofitting existing buildings, because it has little impact on the base loading system.

The remarkable thing in eccentrically braced frames is the link length; as the link length is shorter, the effect of shear force is more than bending anchor and thus, the link member behavior is shear. In the long link, the effect of bending anchor increases and the behavior of the link member is called bending. In the frames with vertical links, shear behavior is mainly emphasized. In applying vertical links in retrofitting structures, there are some restrictions such as proportionality between the floor beams and the link size, strengthening floor beams

due to concentrated moments at the end of links and etc. These restrictions are more significant in concrete structures due to the transfer of shear force to the concrete beams. To solve the above problems, using eccentrically braced frames with a doubled vertical link (DV-EBF) is recommended.

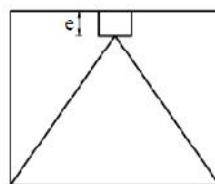


**Fig.2.** Deformation of the eccentrically braced frames [4]

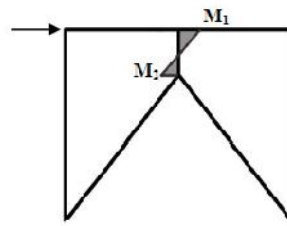
### 3. Advantages of doubled vertical link to single vertical link

According to Figure 3, as this system uses two vertical links at each opening, it has a number of advantages compared to the single vertical link:

1. Reduced number of braced openings (because double link transfers more lateral force from structure to the brace) that results in reduced number of bracing members and their joints.
2. Reduced architectural limitation to adapt with structure (due to reduced number of brace openings).
3. Faster run due to lower volume of operations.



(A)



(B)

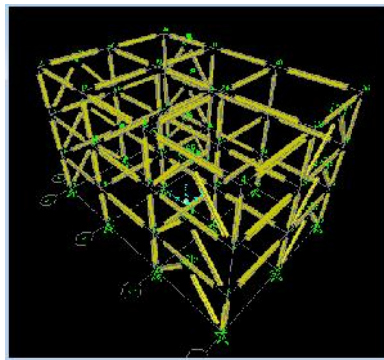
**Fig.3.** Eccentrically braced frame with doubled vertical link (a) anchor distribution in eccentrically braced frame (b) [4]

4. Reduced concentrated moment on the beam and controlling the joint adjacent to vertical link in the floor beam.
5. Reduced force of designing equipment for connecting vertical links to the floor beams and a bracing member.

#### 4. Specifications and location of models

The present project has been considered some building in the city of Mashhad. In Mashhad,  $A = 0.3g$  of soil is type II and number of floors in each plan has been considered three, six, twelve in the form of two centric braced frame (CBF) and eccentric braced frame (EBF).

Figure 4 shows three-dimensional view of one of the structures under study.



**Fig.4.** Three-dimensional model of a three-storey building with CBF

To control its members, non-linear static analysis and dynamic time history have been used.

The specifications of plans are as follows:

Height of all floors: 3 meters

Number of openings per frames: first plan:  $3 \times 2$ ; second plan:  $4 \times 3$ ; third plan:  $3 \times 3$ .

**Table 1.** Number and length of openings of the selected plans

Frame	X		Y	
	Number	Length	Number	Length
2×3	2	5	3	5
3×4	3	4	4	5
3×3	3	4	3	4

Elasticity Modulus in a plan that used Iranian sections:

$$E = 2 \times 10^6 \frac{Kg}{cm^2}$$

$$F_y = 2400 \frac{Kg}{cm^2}$$

$$F_{yc} = 2640 \frac{Kg}{cm^2}$$

And the plans that used American sections:

$$E = 2.03 \times 10^6 \frac{Kg}{cm^2}$$

$$F_y = 3500 \frac{Kg}{cm^2}$$

$$F_{yc} = 4500 \frac{Kg}{cm^2}$$

### 5. Parameters needed using the 10<sup>th</sup> section

A: member cross section

L: member length; if it is a member of brace, the brace length and its bottom sheets are included.

:  $F_a$  a coefficient smaller than 1 that is multiplied by  $F_a$  and prevents from possible buckling of the brace.

$$F_{as} = F_a \times \beta \quad (4-2)$$

$Q_{CE}$  is the expected resistance that is equal to the lowest limit between Global and Local Buckling for the brace under pressure.

$F_a$ : Allowable Stress

$P_{CE}$ : Effective design resistance

The following table shows an example of cross braces UNP10 and UNP12 with the length of 5.85 and the columns of three meters in 3-storey model.

**Table 2.** Parameters needed for plastic hinge of columns

Properties	A	Fa	L	P	$\Delta c$	Qce	$\Delta t$
2IPE16	38/8	1240	320	81658	0/32	100432	0/402
2IPE18	48/5	1343	320	106724	0/391	127780	0/402

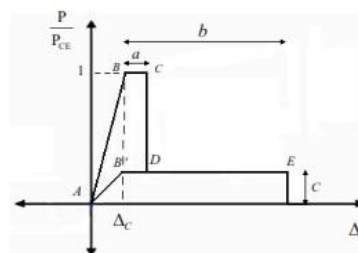
**Table 3.** Parameters needed for plastic hinge braces

Properties	L	P	$\Delta c$	Qce	$\Delta t$
2UNP1	/3	327	333	7117	723
0	58	28	0/	4	0/
2UNP12	/3	458	369	8997	723
	58	82	0/	1	0/

A	Fa	$\beta$	Fas
/96	965	739	714
26		0/	
34	104	757	791
	5	0/	

It should be noted that for centric braces under pressure, Bulletin 360 [7] and the parameters a, b and c and also inclusion criteria IO, LS, and CP are not clearly stated. For the introduction of plastic hinge of these braces to the software, the following action will be taken. Forces of Point B and C are equal. Displacement of points B, C and D are approximately equal to each other as shown in Figure (5).



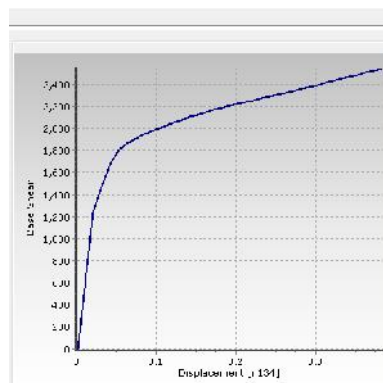
**Fig.5.** The introduction of plastic hinge of push braces [7]

Forces of D and E are also equal. According to what was said, after the introduction of all plastic hinges, we have to assign them to the hinges. The following points have to be considered to assign them:

- Location of hinge is introduced as a proportion of the member length that is 10 percent of the member length in the braces.
- We assign a hinge to the braces; because two ends of the braces are typically hinged and it will have 4 hinges that is instable.
- The hinge shouldn't be assigned to the member's middle part in cross brace; because two braces are connected at the middle and if the middle is selected, the two braces are selected and one hinge is assigned to two braces which is graphically wrong.
- In columns, the location of hinge between the two ends is zero to one.

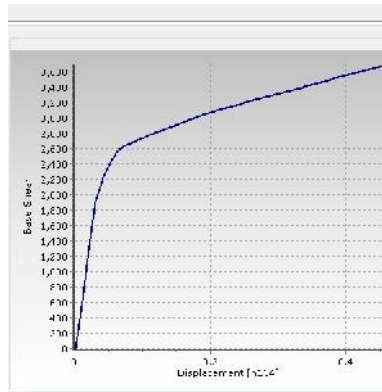
## 6. Loading protocol

In this research, structure capacity curve is usually estimated by a two-line curve for simplification. To do this, we perform according to the FEMA-356 instruction. Some conditions set forth in the Regulations ASCE41-06 to prevent the negative slope of non-linear zone of the capacity curve. Therefore, in this case, the curve with a negative slope is avoided. Figures (6) and (8) show the base shear curve - the capacity curve and determination of the structure performance point using modified displacement of three-storey and six-storey buildings with CBF.

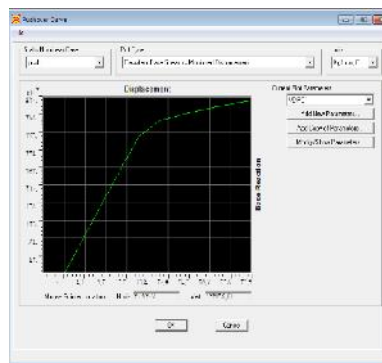


**Fig.6.** Base shear-displacement curve for three-storey with CBF





**Fig.7.** Base shear-displacement curve for six-storey with CBF



**Fig.8.** Base shear-displacement curve for twelve-storey with CBF

### 7. The general trend analysis:

Pushover analysis is an approximate analysis in which, after applying the gravity loads, the structure is exposed to lateral loads with a certain pattern that gradually increases (Incremental). As the lateral loads increase at every step, structure stiffness is also reformed. In other words, the static equilibrium equation  $F = K\Delta$  is controlled at every step and this operation will continue until the center of gravity of the roof reaches a defined amount (target displacement or yield point) or before the fall (unstable). The outcome of this step is the capacity curve which is displayed by displacement of the center of gravity of the roof and base shear. Internal forces as well as members' deformations at each step, such as target displacement or displacement before fall, are compared with their capacity.

### 8. Analysis of results

To determine the design strength, the regulation uses response correction factor (R) that reduces the level of elastic force to the design force level. The existing pushover strength in the structures designed and their ability to earthquake energy dissipation (ductility) are the two constituents of behavior factor. Many methods have been proposed for calculating the

response correction factor; one of these methods is the ductility theory presented by Casneza et al. This method uses a simple behavioral model for a system with one degree of freedom to estimate force reduction factor.

Another popular method for calculating the behavior factor is the proposed method of "Joung" [8]. The reduction factor resulting from ductility and pushover resistance coefficient ( ) are the constituent parameters of behavior factor. In this method, the behavior factor is obtained by multiplying the above two factors. Numerous equations have been suggested to estimate the reduction factor resulting from ductility including "krawinkler" [9] "Newmark" and "Hall" [10], and "Mirinda" and "Brotro" [11].

Given that the present research uses three earthquake records and less than seven accelerographs. The maximum values of accelerographs are used in accordance with the criteria of this force in combination with loads and controlling allowable relative displacement of floor. Thus, the first three modes of structure are those modes with mass participation over 90% or those with a period more than 0.4 second. In general, it is better to consider three modes for each floor (3-storey: first 9 modes, 6-storey: 18 modes, and 12-storey: 36 modes).

Calculation of performance coefficient (pushover resistance coefficient and behavior factor) using dynamic analysis in designing methods based on force is the behavior factor of resistance needed for structure to remain in the linear region. In other words, according to the definition given in regulation FEMAP695, the force proportion established under designed earthquakes with the assumption of perfectly elastic and linear behavior of structure, which is called the behavior factor design forces, are presented in FEMAP695 according to the above definition for calculating behavior factor.

In this study, we have tried to present base shear of designing braced buildings in association with this definition and obtain it by using nonlinear static and dynamic analysis and then calculate the behavior factor.

Since the elastic period begins to increase by the entry of structure into non-linear zone and formation of plastic hinge in structural members, time history of all modes after nonlinear dynamic analysis is investigated using the scaled records.

Nonlinear dynamic analysis results showed that main period of the structure increases in relation to its initial value after formation of the first plastic hinge. In other words, the structure behavior comes out of the linear zone and enters the nonlinear zone. Thus, with a long history of base shear (the base leads to the formation of the first plastic hinge), the main period increases.

2800 standard such as UBC regulations decreases the resistance level of  $V_s$  to the service load level  $V_w$  for further compliance with steel regulations, in which the design is based on limit method or allowable stress. Its value is calculated by dividing  $V_s$  on allowable stress coefficient. This coefficient, as Joung cited, is obtained 1.4 [12] and its value in accordance with the regulations AISC2002 comes from (4-16).

(4.16)

$$Y = \frac{ZF_y}{S(0.6F_y \times \frac{4}{3})} = \frac{1.25Z}{S}$$

In the above equation, S is elastic model and Z is plastic model of the cross section whose value is obtained 1.41 to 1.42 by this relationship. Thus, the behavior factor values in 2800 standard is obtained in accordance with equation (4-17) and (4-18):

$$R_v = \frac{V_c}{V_w} = \frac{V_c}{\frac{V_c}{Y}} = R \times Y \quad (17-4)$$

$$\begin{aligned} Y = 1.4 &\rightarrow V_w \\ &= V_s/Y \end{aligned} \quad (18-4)$$

As a result, after determining the maximum base shear created at each frame, the behavior factor is calculated in this thesis assuming linear elastic behavior in each earthquake using linear time history analysis.

Linear dynamic analysis results by the earthquake scaled with design and behavior factor of each mode based on 2800 standard, the average response of maximum three records has been used.

The ratio of design base shear values ( $V_w$ ) to the weight of the structure and the ratio of maximum elastic base shear ( $V_e$ ) to the weight of the structure are obtained.

According to the above, the base shear corresponding to the formation of the first plastic hinge in the structure that occurs in all models in vertical link, as well as the increasing factor of main period in relation to its initial value and entry to the nonlinear zone were determined as the design base shear ( $V_s$ ).

So the behavior factor, such as an upper limit (or LRFD method) or allowable stress (ASD), is obtained in accordance with Table (4-20) and (4-21).

**Table 4.** The results of centric brace

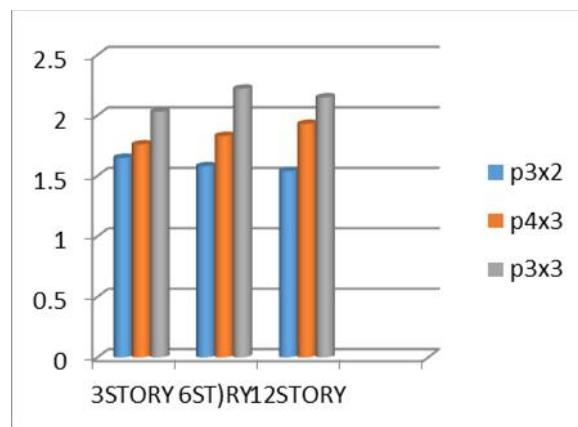
Number of floors	Formation of the first plastic hinge in CBF						Dynamic analysis		
	$V_s/W$	$V_e/W$	$V_{max}/W$	R		$RW = R * Y$	$V_m/W$ Chichi	$V_{max}/W$ Lander	$V_{max}/W$ Lomp
2×3 three-storey									
4×3 three-storey	0/3636	1/322	0/742	6/635	1/554	5/089	0/735	0/7939	0/75909
3×3 three-storey	0/315	1/4102	0/763	3/796	1/67	5/3144	0/701	0/7102	0/7333
3×3 three-storey	0/3412	1/295	0/712	3/795	1/701	5/313	0/67	0/7522	0/7012
six- 2×3 storey	0/307	1/369	0/756	4/459	2/040	6/24	0/735	0/66	0/76
six 4×3 -storey	0/3122	1/373	0/781	4/39	2/053	6/65	0/752	0/675	0/769
six 3×3 -storey	0/3225	1/379	0/786	4/275	2/086	5/985	0/74	0/61	0/71
12- 2×3 storey	0/156	0/7254	0/33734	4/65	2/01	6/51	0/195	0/446	0/235
12- 4×3 storey	0/18	0/8118	0/37592	4/51	2/006	6/314	0/203	0/477	0/237
12- 3×3 storey	0/221	0/95472	0/34965	4/32	2/224	6/048	0/231	0/466	0/288

**Table 5.** The results of eccentric brace

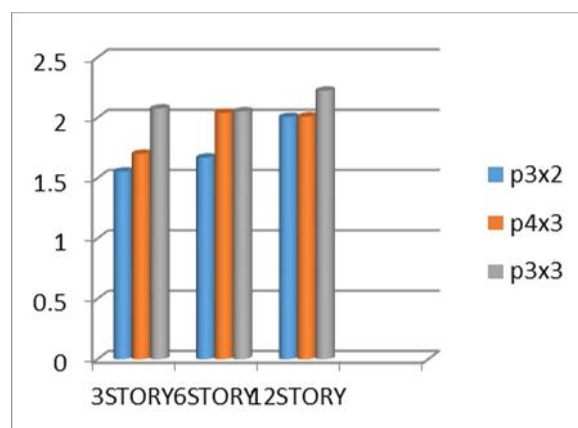
Number of floors	Formation of the first plastic hinge in CBF						Dynamic analysis		
	$V_s/W$	$V_e/W$	$V_{max}/W$	R		$RW = R * Y$	$V_m/W$ Chichi	$V_{max}/W$ Lander	$V_{max}/W$ Lomp
three- 2×3 storey	/3939 0								
three- 4×3 storey	/3712 0	1/602 1/59	0/990 0/821	4/067 4/28	1/54 1/58	5/7 6	0/6223 0/6012	0/5122 0/5961	0/6556 0/623
three- 3×3 storey	/3545 0	1/587	0/863	4/47	1/65	6/25	0/59	0/6125	0/6374
six- 2×3 storey	0/2	0/923	0/426	4/615	1/93	6/46	0/4122	0/435	0/426
six - 4×3 storey	0/212 /2221	0/945	0/422	4/45	1/76	6/23	0/447	0/451	0/460
six - 3×3 storey	0	0/979	0/455	4/43	1/83	6/17	0/452	0/461	0/409
12- 2×3 storey	0/165	0/8399	0/2541	5/09	2.13	7/126	0/239	0/265	0/231
12- 4×3 storey	0/155	0/8075	0/2558	5/21	2.22	7/294	0/241	0/266	0/236
12- 3×3 storey	0/146	0/7782	0/231	5/33	2.149	7/642	0/243	0/244	0/233

**Table 6.** comparison of pushover resistance

Number of floors	Pushover results		Dynamic analysis results	
	EBF	CBF	EBF	CBF
three-storey 2×3	1/65	1/554	1/546	1/56
three-storey 4×3	1/58	1/701	1/561	1/632
three-storey 3×3	1/54	2/086	1/654	1/54
six-storey 2×3	1/76	1/67	1/632	1/987
six -storey 4×3	1/83	2/040	1/657	1/897
six -storey 3×3	1/93	2/053	1/789	1/97
12-storey 2×3	2/13	2/006	1/967	1/93
12-storey 4×3	2/22	2/01	1/975	1/924
12-storey 3×3	2/149	2/224	1/657	2/06



**Fig.9.** bar graph of comparing pushover resistance CBF



**Fig.10.** bar graph of comparing pushover resistance EBF

**Table 7.** average results of behavior factor

Number of floors	Pushover results		Dynamic analysis results	
	EB F	CBF	EB F	CBF
three-storey	5/9833	5/24	6/67	5/45
six - storey	6/39	6/2	6/785	5/76
12-storey	7/294	6/29	7/098	6/87

Number of floors	three-storey		six - storey		12-storey	
	$\frac{V_w}{W}$ CB F	EB F	CB F	EB F	CB F	EBF
Dynamic analysis	137 0/	112 0/	/13 0	/11 0	111 0/	1105 0/
Pushover analysis	242 0/	125 0/	0.2 24	0.1 17	0.1 32	/102 0
Regulation 2800	125 0/	107 0/	125 0/	107 0/	105 0/	/107 0

## 9. CONCLUSION

In brace, the cross section of beam and column decreases the bending frame and it is more evident in the eccentric brace. Bending frame is favorable for architectures in which the opening is easily embedded. But in the bracing system, EBF frames is better than CBF to create opening. We can use two bracing systems without the use of resistant system for about eight floors.

- The ratio of the base shear to the structure weight is reduced with increased height.
- The number of openings has no effect on the values of behavior factor.
- By increasing the number of floors, pushover resistance increases that is due to the rising levels of structure uncertainty.

- Behavior factor of each frame is obtained by the maximum base shear of each frame to the base shear of the first plastic hinge multiplied by Joung coefficient.

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