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Height Effect on Response Modification Factor of Open Chevron Eccentrically Braced Frames

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Abstract: The necessity of making buildings taller and height of new buildings is now one of the major problems in building design and codes of interest is located. With height, the need for further ductility and the capability of energy dissipation can be a significant role in the proper and safe design of structures to play. It seems that changes the height of buildings can be treated directly on the response modification factor of structures and thus the seismic performance of structures under severe earthquake to be effective. This research is an attempt to evaluate the ductility, over-strength and response modification factors of the Open Chevron Eccentrically Braced Frames (EBFs) with 5, 10, 15 and 20 stories under Tabas, Naghan, Bam and El-Centro strong ground motions. These frames were analyzed by using static pushover analysis, linear dynamic and incremental nonlinear dynamic analysis and the values of these factors for different models have been determined separately and it is resulted that with increasing the height of buildings, the ductility reduction factor and the response modification factor of EBFs, decrease, but this variation hasn't any influence on the over strength reduction factor.

Key words: Response modification factor; Ductility reduction factor; Over strength reduction factor; Open chevron eccentrically braced frame; Static pushover analysis; Linear dynamic analysis; Incremental nonlinear dynamic analysis

INTRODUCTION

The structures should be designed in a way that they have resistant enough against severe earthquakes and they should also provide comfort and peace of mind of residents who live there against weaker earthquakes. In other words, a structure not only should dissipate a considerable amount of imported energy by ductile behavior, but also it should be able to control the deformations and transfer the force to foundation through enough lateral stiffness in ground motions.

Concentrically braced frames (CBFs) due to sufficient hardness are regarded designers, but on the other hand the architecture problems as well as their low capability of absorbing and dissipating energy and their buckling to pressure are the disadvantages of this bracing system. This matter drew the attention of designers to EBFs which have these characteristics.

Most of the codes have placed the basis of seismic design of EBFs on resistance and ductility and all designers agree this assumption that some part of input energy which is arrived by earthquake should be dissipated by plastic deformations, although considering this point that deformations should be limited and they have to be dependent upon permitted limits.

The final capacity of dissipated energy in every structure depends upon various factors such as: structure's seismic parameters, characteristic of earthquake records and the environmental conditions of constructing place of a structure. The response modification factor is the reflection of energy dissipation within the boundary of plastic with respect to the lack of overturning and big deformations in structure. Height of structure is a one of various parameters which is effective on the response modification factor, that in this research this matter is studied on EBFs.

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In the recent years, the need of a more appropriate and accurate design of tall buildings caused to study the height effects on response modification factor. For this reason, in this research by using static pushover analysis, linear dynamic and incremental nonlinear dynamic analysis, the effect of height on ductility reduction factor, over-strength reduction factor and response modification factor of EBFs studied.

Response Modification Factor: In most codes including UBC97 [1], NEHRP [2], NBCC [3] and Iranian Code of Practice for Seismic Resistant Design of Buildings [4], the effective load resulted from earthquake are assessed based on linear elastic analyses. Since these structures show nonlinear behavior, the forces are reduced using response modification factor and they are corrected in this way. Mazzolani and Piluso [5] evaluated several theoretical approaches such as low cycle fatigue, energy and maximum plastic deformation approaches to compute response modification factor. Based on ATC3-06 [6], ATC-19 [7] and ATC-34 [8], the response modification factor was calculated as the product of three factors: Ductility Reduction factor, Rì, Over-strength Reduction factor, Rs and Allowable stress factor, *Y*. Uang [9] presented these parameters by Fig. 1. In this figure, the response modification factor can be calculated as follows [Eq. (1)]:

$$R = \frac{V_e}{V_d} \tag{1}$$

In the above equation, V_e is the maximum base shear considering elastic behavior and V_d is the design base shear of structure.

Taking into account that the design methods are common between level A: the load factor and ultimate strength in steel (AISC-LRFD [10]) and level B: allowable stress design method (Iranian Steel Code [11] and AISC-ASD [12]), one of the two values of V_s (the base shear related to the first plastic hinge formation in structure) and V_w (the base shear related to the first allowable stress formation in any elements of structure) can be attributed to V_d . Thus the above relation can be written as follows:

$$R_u = \frac{V_e}{V_s} \tag{2}$$

$$R_w = \frac{V_e}{V_w} \tag{3}$$

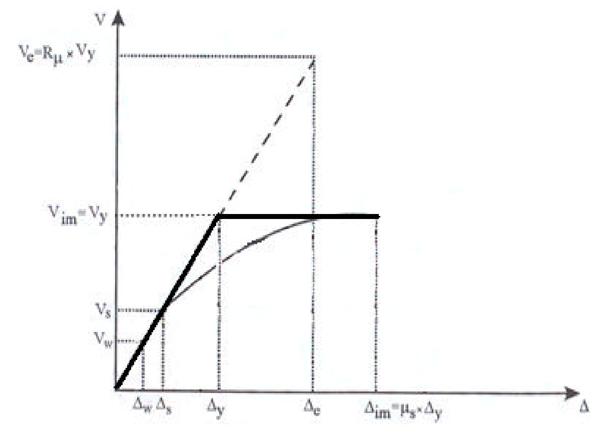

Fig. 1: Definition of nonlinear parameters

In the above relations, R_u is the response modification factor based on ultimate limit stress design method and R_w is the response modification factor based on allowable stress design method. The following relation can be defined between the two above-said design levels:

$$Y = \frac{R_w}{R_u} = \frac{V_s}{V_w} \tag{4}$$

In the above relation, *Y*, is the Redundancy factor determined based on the attitude of design codes toward design stress (yield stress and allowable stress) and the value of such factors is usually between 1.4 to 1.7. UBC97 code [1] considers the value of this factor 1.4, that is:

$$R_w = 1.4\ R_u \tag{5}$$

To specify the role of ductility and over strength in response modification factor, the said factor is defined as follows:

$$R_u = \frac{V_e}{V_s} = \frac{V_e}{V_y} \times \frac{V_y}{V_s} = R_m \times R_s \tag{6}$$

Where V_y is the maximum base shear in an elastic-perfectly plastic idealized response curve of the structure and Δ_y is the yield displacement [9]. Thus, having the value of $R_m = \frac{V_e}{V_y}$ (the reduction factor resulting from ductility) and $R_S = \frac{V_y}{V_S}$ (the reduction factor resulting from over strength), we can calculate the response modification factor of a structural system.

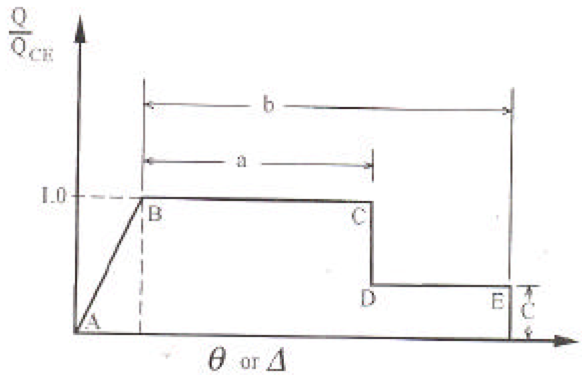

Fig. 2: An example of force curve - deformation of nonlinear hinges according to FEMA 273 code [13].

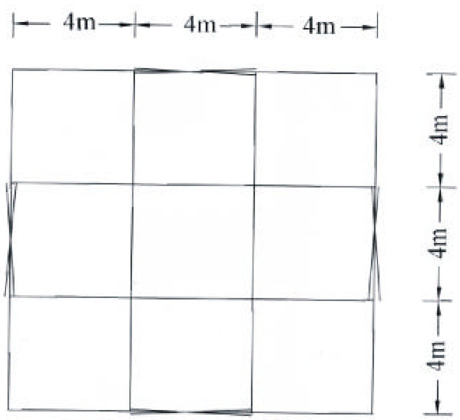

Fig. 3: Typical plans

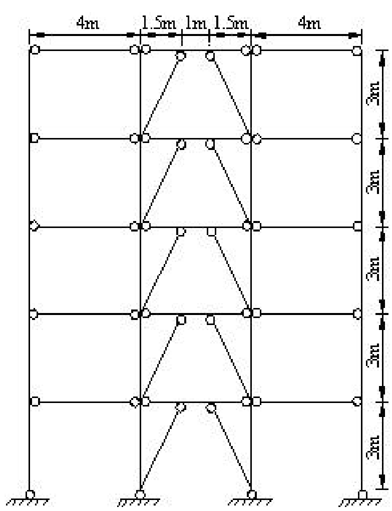

Fig. 4: layouts of 5 story models

Structural Damage Standards: In FEMA 273 code [13], a new method is presented in the assessment of safety and damage of structural systems. By this method which is based on a nonlinear static analysis method, deformation of sections is calculated based on yield limit deformation. This code classifies the section safety qualitatively based on the ratio of deformation sections to yield limit deformation. In this method, the damage of each section is specified using the above standards and as to the general behavior of the structure, the damage imposed on the whole structure will be specified by calculating the status of performance point (Fig. 2).

In Fig. 2, Q is the force at hinge that can be the moment or shear, Q_{CE} is the yield limit of these two values, Δ and θ are the displacement and rotation of the hinge, respectively. Having the performance point in hand, the status of the structure under the effect of maximum seismic load can be specified. In this way, we can precisely predict what are the deformations imposed on each structural element and on the installations and this way we can do as required for strengthening them.

In this article, to define nonlinear hinges for the sections of structural elements and assign them to the elements, FEMA 273 code [13] has been used.

Design of Samples: In this research, in order to evaluate the height effects on structure response modification factor, some buildings with 5, 10, 15 and 20 floors which each floor is 3 meters high with typical plans according to Fig. 3, are designed based on the Iranian National Building Code, part 10, steel structure design [11].

These frames have been designed for dead load about 550 kg/m^2 and live load about 200 kg/m^2 in an area with very high risk seismicity and in a third type soil according to Iranian Code of Practice for Seismic Resistant Design of Buildings (Standard No. 2800) [4]. In this type of soil, the average velocity of shear waves in 30-meter distance of the depth of the land is between 175 to 375 m/s [4].

The selected braces in these structures are open chevron eccentric braces which their linkage beam's length and their out of linkage beam's length are according to Fig. 4. For member design subjected to earthquakes, equivalent lateral static forces were used at all the story levels. These forces were calculated following the provisions stated in Iranian Code of Practice for Seismic Resistant Design of Buildings (Standard No. 2800) [4] and the preliminary response modification factors of $R_w = 7$ were considered in frame design. For the steel applied, the Modules of Elasticity, Yield Strength and Ultimate Strength were considered 2040000 kg/cm^2, 2400 kg/cm^2 and 3700 kg/cm^2, respectively.

Table 1: Characteristics of selected acceleration records

Earthquake Name	Station Name	Year	M_s	PGA (g)
Tabas	Bushrooye 33-57	1978	7.4	0.93
Bam	Bam 58-28	2004	6.5	0.80
Naghan	Naghan 235-325	1977	6.1	0.70
El-Centro	El-Centro S00E	1940	6.6	0.35

Table 2: Nonlinear and linear maximum base shear under scaled selected records

Number of Story	Tabas Earthquake		Bam Earthquake		Naghan Earthquake		El-Centro Earthquake	
	V_y (kg)	V_e (kg)	V_y (kg)	V_e (kg)	V_y (kg)	V_e (kg)	V_y (kg)	V_e (kg)
5	43644	238100	32216	239321	38476	230800	41520	238491
10	58822	288349	57190	286541	54310	281576	59100	289860
15	68257	280916	62860	280721	60900	280016	66810	280842
20	75020	246257	62230	247816	71850	241822	69140	249200

Table 3: Ductility, over-strength and response modification factors of the models

Number of stories	V_s(kg)	$V_{y(ave)}$(kg)	$V_{e(ave)}$(kg)	R_s	R_μ	R	R_w
5	25848	38964	236678	1.51	6.07	9.17	12.83
10	33016	57355	286581	1.74	5.00	8.70	12.18
15	41321	64707	280624	1.57	4.34	6.81	9.54
20	45720	69560	246274	1.52	3.54	5.38	7.53

All structures were modeled in software of SAP2000 [14]. The following assumptions were assumed for modelled by the members in a nonlinear range of deformation:

- All beams and braces were connected to columns by pin connections.
- For the dynamic analysis, story masses were placed in the story levels considering rigid diaphragms action.
- The idealized elastic-plastic behavior with strain hardening of 2% was considered for members with inelastic behavior.
- The P-Δ effect was considered for considering geometric nonlinearities.

Dynamics Analysis of Models and Results Evaluation: To evaluate the base shear related to the first plastic hinge formation in structure, V_s, it is supposed that the linear ultimate limitations of the structure in the nonlinear static analysis and dynamic analyses have been considered the same [15]. After modeling the frames, to gain V_s the base shear force were calculated based on equivalent static loading model specified in the Iranian Code of Practice for Seismic Resistant Design of Buildings (Standard No. 2800) [4] and distributed in stories and structures underwent non-linear static (pushover) analysis.

To gain V_y, at first, four severe earthquakes (Table 1) were selected and matched with the design spectrum. To do so, their PGA's with several tries and errors had altered in a way that the calculated time history resulted in the structure reaching to the following failure criteria:

- Based on the Iranian Code of Practice for Seismic Resistant Design of Buildings (Standard No. 2800) [4], the maximum inter-story drift was limited to the values 0.025 H and 0.020 H for the frames with a fundamental period less than 0.7 s and more than 0.7 s, respectively, where H is story height.

Then the incremental nonlinear dynamic analysis of the models under these scaled strong ground motion was carried out and the maximum nonlinear base shear of this time history, V_y, was calculated. Finally, by linear dynamic analysis of the structure under the same scaled records, the maximum linear base shear, V_e, was gained. Then the average of V_e and V_y for each model is computed and over-strength reduction factors, R_s and ductility reduction factors, R_μ, based on Equation 6 are computed.

As the primary frames were designed based on preliminary response modification factor and their tentative values were evaluated, a repeat on response modification factor calculation was performed considering latest values. To calculate final seismic response modification factors, the models were amended and designed based on new response modification factors. Then, according to the mentioned procedure, all models were analyzed and their final seismic response modification factors were calculated. Finally, these converged values are shown in Tables 2 and 3. Based on UBC97 code [1] recommendation, by using Y = 1.4,

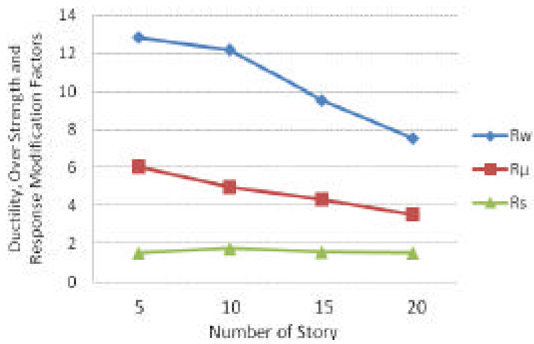

Fig. 5: Variations of ductility, over strength and response modification factors for frames with different number of stories

response modification factor for allowable stress design method, R_w, are gained for different models and these values are presented in Table 3. Based on these results, the mean response modification factors are 12.83, 12.18, 9.54 and 7.53 for buildings with 5, 10, 15 and 20 stories, respectively.

It should be noted that since earthquake records studied in this research have been scaled to design earthquake and according to the Iranian code of Practice for Seismic Resistant Design of Buildings (Standard No. 2800) [4] the live safety performance expect for typical buildings under design earthquake, so in this research, the response modification factor was evaluated for life safety level.

In Fig. 5, the ductility, over strength and response modification factors of frames with different floors' number are drawn. This figure shows that the number of floors can't affect the over strength reduction factor, R_s and its value is fixed, but with increasing the number of frame story, the ductility reduction factor, R_μ and the response modification factor for allowable stress design method, R_w, decreases.

Having been calculated Rw from the Equation 5 and its demonstrated in Fig. 5, it is clear that with increasing the height of floors from 5 to 20 floors, R_w decreases from 12.83 and it is limited to 7.53.

In the Iranian code of practice for seismic resistant design of buildings [4], number 7 for R_w and the maximum allowable height of building H_w = 50 meters are suggested for this kind of bracing that it seems to be conservative for all ranges of height of buildings. Since often buildings in Iran have maximum 10 stories, considering number 7 for R_w to design these buildings is a very conservative and it seems that it is better this parameter to be increased, or multiple R_w be presented for low, medium and high buildings.

With respect to the monotony of changes in R_w comparing to the number of floors for braced frames using eccentric open chevron bracings, we can suggest the following equation for low and medium buildings with maximum 20 stories:

$$R_w = 14.6 - 0.35\,N \qquad (7)$$

Where, N is the number of frame floors.

CONCLUSIONS

The following results are obtained in a braced frame with open chevron eccentric bracings which are influenced by severe earthquakes and are analyzed by using nonlinear static, linear dynamic and non-linear dynamic methods:

- By increasing the height of the frame, the response modification factor of consequence of over strength (over strength reduction factor), is relatively fixed.
- With increasing the frame height, the response modification factor of consequence of ductility (ductility reduction factor), decreases.
- The response modification factor of the frame based on allowable stress design method, R_w, will be decreased with increasing the height of buildings and the equation $R_w = 14.6 - 0.35N$ can be considered as the suggested equation for this kind of frames that in this equation N is the stories number.

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