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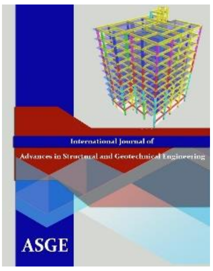

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ASSESSMENT OF SEISMIC BEHAVIOR OF ECCENTRIC BRACED STEEL FRAMES

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ABSTRACT

Eccentrically braced frames (EBFs) are commonly used as lateral load resisting systems for steel structures. When subjected to strong seismic excitations, it is expected that the lateral load resisting system will experience inelastic deformations. Most design codes list strength reduction factor values corresponding to each structural system as a parameter to consider the ability and the efficiency of such system to dissipate earthquake energy through inelastic deformation. Hence, earthquake loads are reduced to match the actual behavior of structures under seismic loads. This is performed by dividing the calculated base shear by strength reduction factor. Strength reduction factor consists of three main components representing ductility, over-strength and redundancy of the considered structure. These factors depend mainly on properties of the structural system and construction material. In this research, a parametric study is conducted to calculate the strength reduction factor values for eccentrically braced steel frames. The considered parameters include storey height, number of bays, and shear link length in addition to location of braced bay. Nonlinear static (pushover) and time history analyses were performed in order to calculate reduction factor for different 2-D models. Effects of different considered parameters on strength reduction factor were evaluated and compared to the values proposed by different design codes.

Keywords: Eccentrically braced frame, Parametric Study, Force Reduction factor.

INTRODUCTION

Braced frames are called eccentric when the centerline of braces does not intersect at the same point. Link length (e) is the clear distance between the ends of the connections of two diagonal braces. There are many configurations for these frames (K, D, X and Y-shapes). Studies have shown that (EBFs) are stiff and can easily satisfy storey drift limitations, and are economical compared to concentric braced frames. Eccentrically braced steel frames (EBFs) combine both the stiffness of ordinary bracing and the ductility of moment resisting ones. They can be designed to provide inelastic behavior and energy dissipation for structures. Therefore, such structures have sufficient strength and stiffness to remain elastic and serviceable under moderate but frequently occurring earthquakes. Moreover, it has sufficient ductility to prevent collapse under extreme earthquakes.

During the last few decades, the seismic resistance has gained a lot of interest on both research and design practice level. Design of earthquake resistant structures is based on two main requirements. First, the structure must remain serviceable during the frequent occurring load applications. This is usually accomplished by designing the structure so that it remains elastic and provides adequate stiffness to limit deflections. Second requirement is to prevent a disaster during a major earthquake. For such an extreme event, considerable inelastic deformation is usually allowed. One of the most important factors used to approximate seismic forces is the force reduction modification factor R. The force reduction factor R represents the ratio of the maximum lateral force of the linear elastic response of a structure V_e under specific ground motion, and the actual inelastic response of the structure V_w under the same ground motion. The ratio R, expressed by the equation:

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

force reduction factor is usually divided into three components which are over strength R_S, ductility R_μ, and redundancy R_R of the structure. The general formula proposed to define the reduction factor is given by:

$$R = R_\mu\, R_S\, R_R \tag{2}$$

As shown in Fig. 1, the three components R_μ, R_s and R_R represent the Energy absorption/dissipation capacity, expected over-strength, and degree of redundancy of the structure system, respectively.

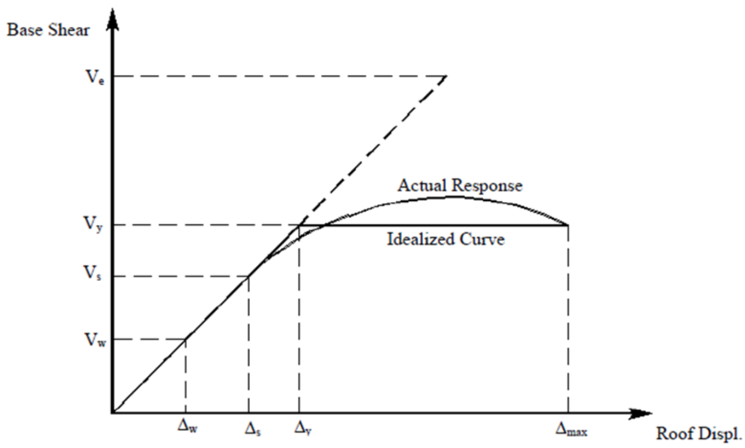

Fig. 1: Typical elastic and inelastic pushover response curve

Several researchers have studied the force reduction factor components considering different structural systems. Newmark and Hall (1982) [1] related the ductility reduction factor R_μ to the ductility of the system by considering single degree of freedom systems. Riddell and Vasquez (1988) [2] used a simple SDOF system with three types of material nonlinearity: elastoplastic, bilinear and stiffness-degrading to calculate elastic and inelastic response spectra. The behavior of long links in EBFs and its failure mode was studied by Engelhardt and Popov (1992) [3]. Miranda (1993) [4] studied a group of 124 ground motions recorded on a wide range of soil conditions to study the influence of local site conditions on ductility reduction factors. Borzi and Elnashai (2000) [5] investigated the relationship between ductility supply and demand. They used two reinforced concrete regular frames designed according to EC8 provisions to evaluate ductility supply by means of inelastic static pushover analysis. Maheri and Akbari (2003) [6] used inelastic pushover analyses for braced frames of different configurations to evaluate the seismic behavior factor considering x-bracing and knee bracing configurations. McDaniel et al. (2003) [7] performed a cyclic test of two full scale built-up shear links for the main tower of the new San Francisco-Oakland to evaluate the strength and deformation characteristics of the links. Sarraf and Bruneau (2004) [8] performed a cyclic test on two different configurations of eccentrically braced frames and vertical shear link. Moghaddam and Hajirasouliha (2006) [9] conducted nonlinear dynamic analysis on 5-, 10- and 15-storey braced frames subject to fifteen earthquake records to verify the reliability of pushover analysis. Berman and Bruneau (2008) [10] studied a self-stabilizing link for eccentrically braced frames utilizing a tubular cross section and its mode of failure. Hassan et al. (2009) [11] evaluated (R_μ) factor using two different methods; static pushover analysis applied with N2-Method and time history method using seven earthquake ground motion records. Mansour et al. (2011) [12] developed a method for designing EBFs with replaceable shear links. This method allows wider clearance for openings or circulation while still achieving a desirable link shear yielding inelastic response Mahmoudi and Zaree (2013) [13] evaluated the (R_μ) for 20 braced frames using N2-Method using the nonlinear static (pushover) analysis on models with different brace configurations. Yahmi and Branci (2017) [14] studied the effect of number of storeys and bays on the redundancy reduction factor (R_R). However, there are insufficient studies addressing the impact of structure characteristics on the behavior of different structural systems designed according to the Egyptian code. This study focuses on calculating the force reduction factor values for eccentric braced frames designed according to the Egyptian Code for different storey height structures.

STRUCTURAL MODELS

The aim of this study is to measure the effect of height, shear link length, number of bays and location of braced bay on force reduction factor of eccentric braced frame designed according to the Egyptian code of practice [15,16] requirements for different steel building configurations. The structures are modeled as two-dimensional (2D) frame elements using SAP2000 program (CSI, 2003) [17], considering eccentric-braced frames (EBFs) of K-bracing shape with different heights 3, 6, 9, 12, 15 and 18 storey steel frames, and variable shear link length (e) equal to 1, 1.5 and 2m. The models are divided to three bays with single bracing in the middle bay and four bays with double bracing in the edge bays, while the remaining bays include simple beam-to-column connection not resisting any lateral loads as showed in Fig. (2). The steel building is symmetric in plan with a typical bay width 6m for all frames, and the spacing between frames is 6m. The typical storey height is 3m and the column support at the base is hinged.

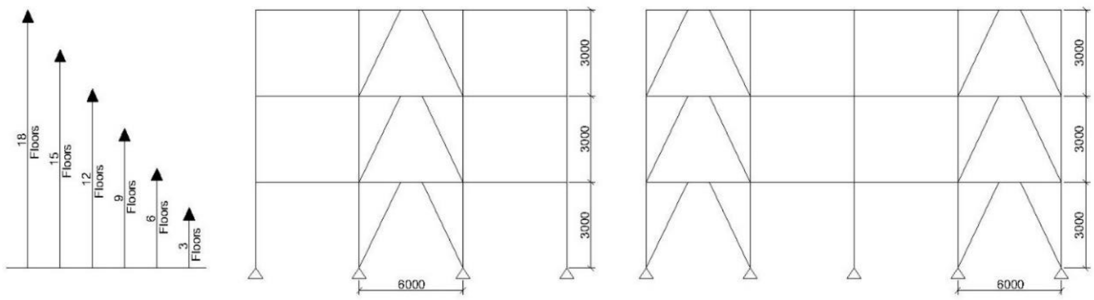

Fig. 2: layouts for structure models

All of the steel elements are made of steel having modulus of elasticity, yield strength and ultimate strength equal to 210000 MPa, 240 MPa and 360 MPa, respectively. Design loads are 5.3 kN/m^2 dead load, 5 kN/m2 live load and 0.7 kN/m^2 wind load. A force reduction factor of R = 4.5 was considered in frame design according to considered structural system. The earthquake loads are calculated according to the Egyptian code simplified modal response spectrum method, where the ultimate base shear force Vb is calculated based on the following formula:

$$V_b = S_d\,(T_1)\,\lambda\,W/g \qquad (3)$$

The applied ultimate base shear force (V_b) in Eq. (3) is then distributed along the height of the steel structures in the form of triangular static load concentrated at the location of floor level, the applied lateral load is given by:

$$F_i = \frac{m_i h_i}{\sum_{j=1}^{n} m_j h_j} V_b \qquad (4)$$

According to the Egyptian code of loads and forces, the mass source is applied using the combination of total dead load (DL) plus 25% of live load (LL).
The considered parameters in the current study are number of storeys, number of bays, shear link length and peak ground acceleration using static pushover and dynamic time history analysis.

The model name is chosen in the form of E number of bays - number of storeys – shear link length, for example, E3-6-1
Where:
E: represents Eccentric braced frames.
3: represents number of bays.
6: represents number of storeys.
1: represents shear link length.

Table 1: Ranges of studied parameters

Number of storeys	3 storeys	6 storeys	9 storeys	12 storeys	15 storeys	18 storeys
Number of bays	3 bays	4 bays	--	--	--	--
Shear link length	1 m	1.5 m	2 m	--	--	--
Peak ground acceleration	0.2g	0.3g	0.45g	--	--	--

Table 2 shows the main characteristics of links and the expected type of behavior.

Table 2: Shear link cross-section properties and check for the link type.

Section	**Aw (cm2)**	**Z (cm3)**	**Mp (kN.m)**	**Vp (kN)**	**1.6Mp/Vp (m)**	**2.6Mp/Vp (m)**	**e (m)**	**Link Type**
350x220/6x20	18.60	1455.82	349.39	267.8	2.09	3.39	1	Shear Link
							1.5	Shear Link
							2	Shear Link
450x250/6x20	24.60	2209.08	530.17	354.2	2.39	3.89	1	Shear Link
							1.5	Shear Link
							2	Shear Link
550x280/8x22	40.48	3438.00	825.12	582.9	2.26	3.68	1	Shear Link
							1.5	Shear Link
							2	Shear Link
650x300/10x22	60.60	5063.46	1215.23	872.6	2.23	3.62	1	Shear Link
							1.5	Shear Link
							2	Shear Link

STRUCTURAL ANALYSIS

Nonlinear static (pushover) and time history analyses are performed using SAP2000 software program to calculate the ductility reduction factor for structural models, under displacement control where the maximum permissible interstorey drift is 2% of the total height of the building. This limit is specified as per Mwafy and Elnashai 2002 [18], Massumi et al. 2004 [19], and close

to those adopted by seismic design codes Eurocode 8 (2004) and UBC 97 (1997) [20], which vary between 2 and 3%.
The non-linear behavior of the material is simulated through plastic hinges in frame elements. For the objective of this research, plastic hinges are assigned to the ends of frame elements according to FEMA-356 (2000) [21]:

- For all columns, the (P-M3) hinge is assigned to take into consideration interaction between axial forces and bending moments on the plastic hinge.
- For all shear links, the shear hinge (V2) is used.

Pushover is a simplified nonlinear procedure, in which the structure is subjected to a monotonically increasing lateral load of a predetermined pattern. As the load increases, critical zones deform beyond the yield limit and the structure degrades. At each load step, the relation between the base shear and lateral drift is acquired. This procedure continues until the structure collapses or reaches a predetermined lateral deflection. At the end, the relationship between base shear and lateral deflection, that is called capacity curve, is determined.
Nonlinear time history analysis is also performed using seven earthquake records as illustrated in Fig.3. Both material and geometric non-linearities are considered in the analysis. After applying the ground motions, the relation between base shear and top displacement is obtained for different peak ground acceleration (PGA) levels corresponding to different ground motion records. The area enclosed by the maximum excursion is then calculated. This area corresponds to the maximum energy dissipated by the structure.

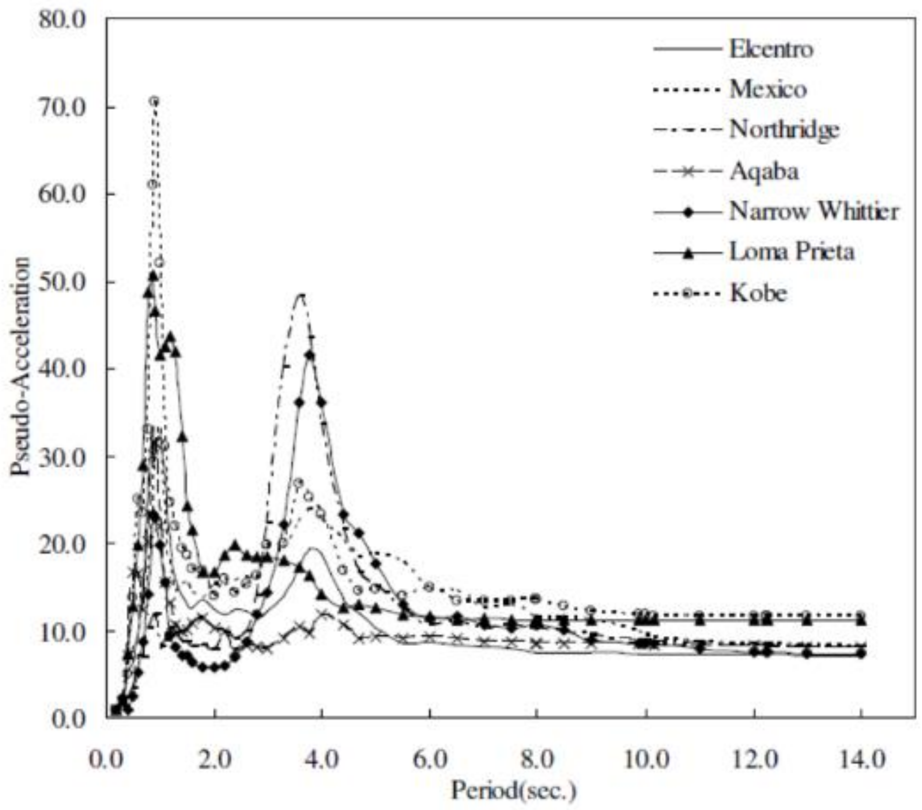

Fig. 3: Response spectra for the used seven earthquake records

RESULTS

The fundamental natural period of vibration is evaluated by the finite element model. Tables 3 and 4 show fundamental natural periods of vibration for the different models considering the first five modes. It is worth mentioning that the group (2) models are much stiffer than group (1) due to the fact that they include two bracing bays whereas group (1) include only one braced bay.

Table 3: Fundamental natural period of Group (1).

Model	Fundamental Natural Period (Sec.)				
	Mode (1)	Mode (2)	Mode (3)	Mode (4)	Mode (5)
E3-3-1	0.412	0.149	---	---	---
E3-3-1.5	0.473	0.170	---	---	---
E3-3-2	0.546	0.194	---	---	---
E3-6-1	0.807	0.286	0.159	---	---
E3-6-1.5	0.896	0.322	0.181	---	---
E3-6-2	1.000	0.365	0.206	---	---
E3-9-1	0.899	0.302	0.158	---	---
E3-9-1.5	1.012	0.341	0.176	---	---
E3-9-2	1.148	0.384	0.194	---	---
E3-12-1	1.251	0.407	0.214	0.139	---
E3-12-1.5	1.372	0.457	0.239	0.152	---
E3-12-2	1.523	0.513	0.264	0.164	---
E3-15-1	1.560	0.498	0.255	0.167	---
E3-15-1.5	1.679	0.554	0.285	0.185	---
E3-15-2	1.832	0.618	0.318	0.202	---
E3-18-1	1.983	0.603	0.304	0.200	0.171
E3-18-1.5	2.100	0.664	0.339	0.221	0.171
E3-18-2	2.254	0.737	0.379	0.243	0.171

Table 4: Fundamental natural period of Group (2).

Model	Fundamental Natural Period (Sec.)				
	Mode (1)	Mode (2)	Mode (3)	Mode (4)	Mode (5)
E4-3-1	0.340	0.127	---	---	---
E4-3-1.5	0.389	0.145	---	---	---
E4-3-2	0.448	0.166	---	---	---
E4-6-1	0.666	0.238	0.159	---	---
E4-6-1.5	0.737	0.268	0.181	---	---
E4-6-2	0.827	0.304	0.206	---	---
E4-9-1	0.672	0.220	0.158	---	---
E4-9-1.5	0.759	0.257	0.176	---	---
E4-9-2	0.865	0.297	0.194	---	---
E4-12-1	1.096	0.347	0.214	0.139	---
E4-12-1.5	1.182	0.389	0.239	0.152	---
E4-12-2	1.293	0.437	0.264	0.164	---
E4-15-1	1.411	0.427	0.255	0.167	---
E4-15-1.5	1.490	0.472	0.285	0.185	---
E4-15-2	1.595	0.525	0.318	0.202	---
E4-18-1	1.811	0.525	0.304	0.200	0.171
E4-18-1.5	1.889	0.574	0.339	0.221	0.171
E4-18-2	1.995	0.634	0.379	0.243	0.171

Fig.s 4 to 9 show the relationships between results of (R) and the height of the structure for different shear link length calculated for various (PGA) levels (0.20g, 0.30g and 0.45g). The results show that a general decrease in the values of the force reduction factor (R) occurs as the number of storeys increase. It is observed that the force reduction factor decreases as the length of the shear link (e) increase and this is due to the increase in the inelastic deformations of the structure.

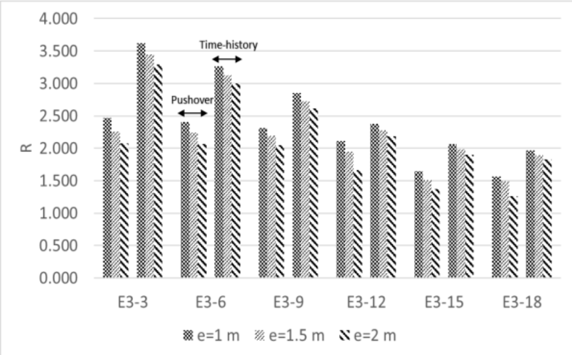

Fig. 4: Force Reduction Factor (R) for Different models in Group (1) (ag=0.2g).

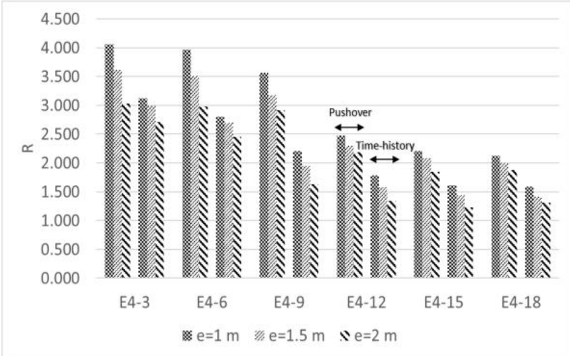

Fig. 5: Force Reduction Factor (R) for Different models in Group (2) (ag=0.2g).

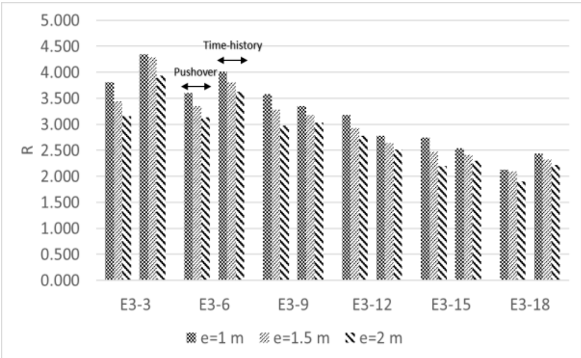

Fig. 6: Force Reduction Factor (R) for Different models in Group (1) (ag=0.3g).

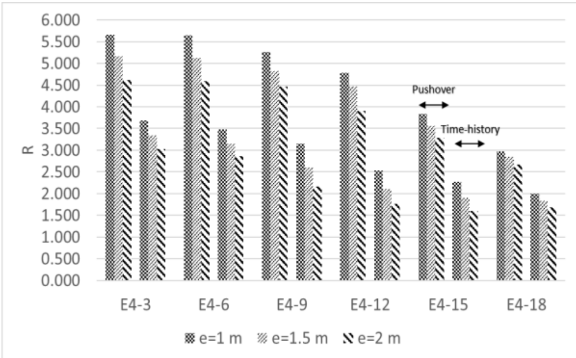

Fig. 7: Force Reduction Factor (R) for Different models in Group (2) (ag=0.3g).

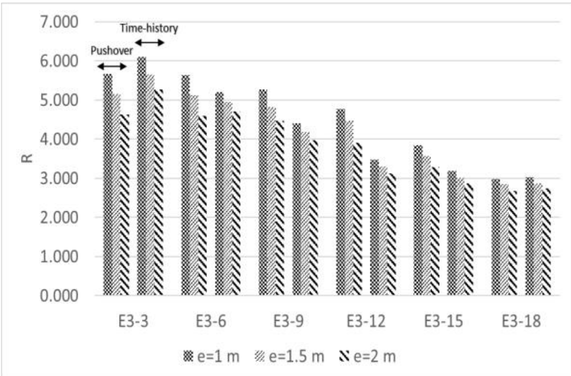

Fig. 8: Force Reduction Factor (R) for Different models in Group (1) (ag=0.45g).

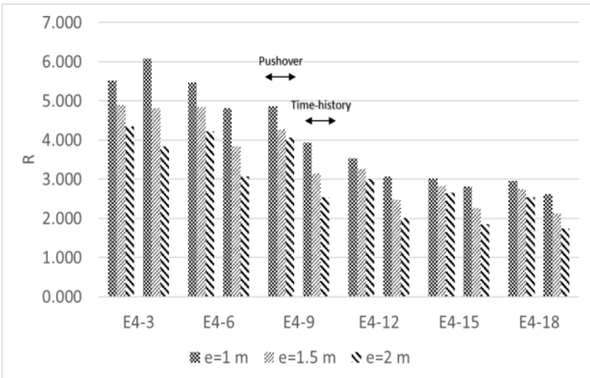

Fig. 9: Force Reduction Factor (R) for Different models in Group (2) (ag=0.45g).

Fig.s 10 to 15 show the relationships between results of (R) and the number of storeys of structures for deferent number of bays for various (PGA) levels (0.20g, 0.30g and 0.45g).

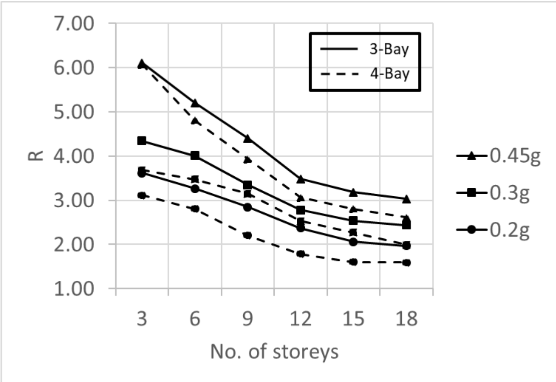

Fig. 10: Force Reduction Factor (R) Vs. Number of storeys for Time history analysis (e = 1m).

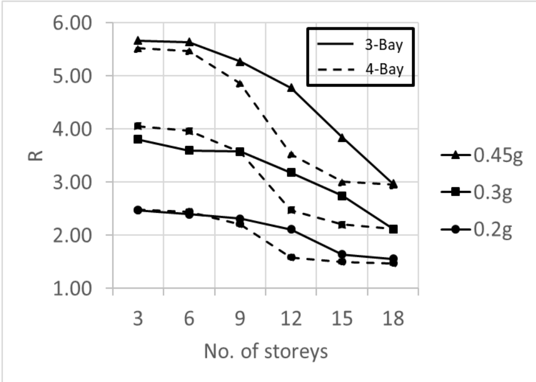

Fig. 11: Force Reduction Factor (R) Vs. Number of storeys for Pushover analysis (e = 1m).

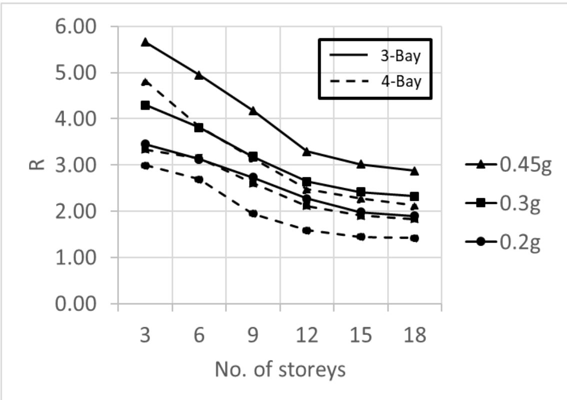

Fig. 12: Force Reduction Factor (R) Vs. Number of storeys for Time history analysis (e = 1.5m).

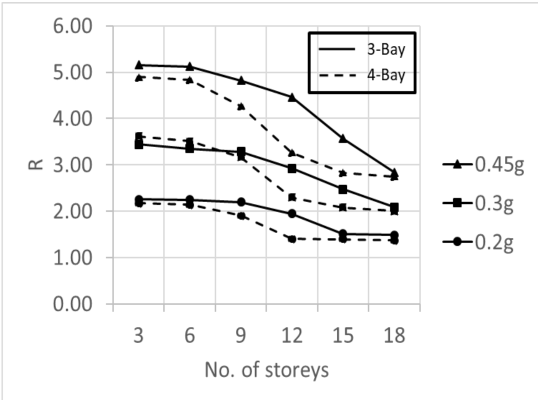

Fig. 13: Force Reduction Factor (R) Vs. Number of storeys for Pushover analysis (e = 1.5m).

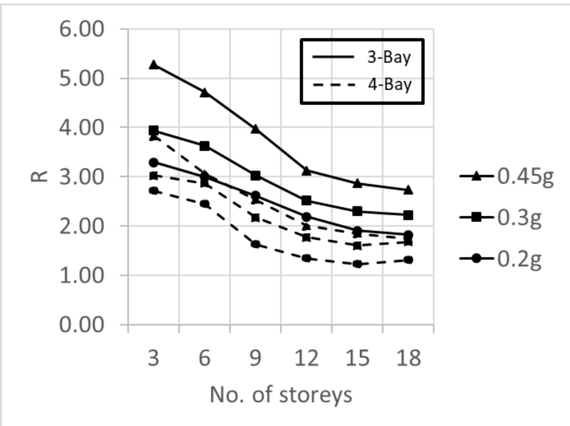

Fig. 14: Force Reduction Factor (R) Vs. Number of storeys for Time history analysis (e = 2m).

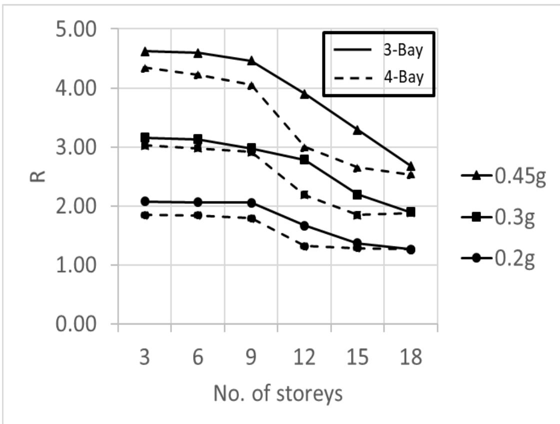

Fig. 15: Force Reduction Factor (R) Vs. Number of storeys for Pushover analysis (e = 2m).

It can be observed that the force reduction factor for the studied models ranges from 1.3 to 6.1. the lowest values of the reduction factor are exhibited by the highest frames and the highest force reduction factors are exhibited by the shortest frames. The reduction factor values increase as the level of peak ground acceleration increases.

Table 5 shows a Comparison between the force reduction factors proposed by ASCE7-10, EC8 and ECP201 for different structural systems. The results indicate that the force reduction factor (R) is inversely proportional to the height of the structures. Hence, using a single value for the reduction factor may result in uneconomic design in some cases.

Table 5: Comparison between R factors proposed by different codes

<table>
<tr><th rowspan="2">Structural system</th><th rowspan="2">ASCE7-10</th><th colspan="2">EC8</th><th rowspan="2">ECP201</th></tr>
<tr><th>DCM</th><th>DCH</th></tr>
<tr><td>Steel Special Moment Frames</td><td>8</td><td rowspan="2">4</td><td rowspan="2">6</td><td rowspan="2">5</td></tr>
<tr><td>Steel Ordinary Moment Frames</td><td>3.50</td></tr>
<tr><td>Steel Special Concentrically Braced Frames</td><td>6</td><td colspan="2" rowspan="2">4</td><td rowspan="2">4.5</td></tr>
<tr><td>Steel Ordinary Concentrically Braced Frames</td><td>3.25</td></tr>
<tr><td>Steel Eccentrically Braced Frames</td><td>8</td><td>4</td><td>6</td><td>From the Study 1.3~ 6.1</td></tr>
</table>

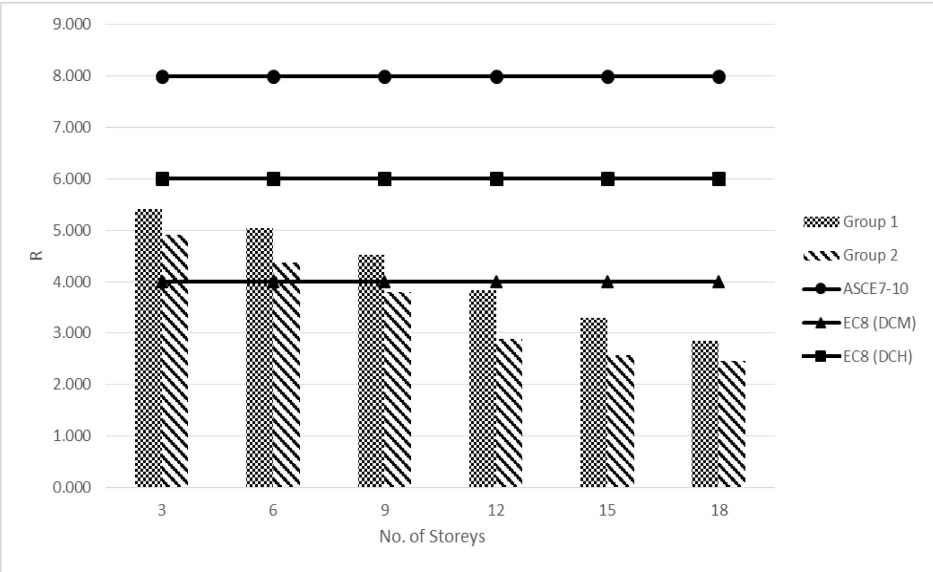

Fig. 16: Comparison between R factors for eccentric braced frames proposed by different codes

CONCLUSION

Force reduction factor (R) values range between 1.3 and 6.1 the studied models in the present work. Using a single value for R considering the type of the structural system may not be the best practice. It is recommended to provide a detailed explanation for the definition of eccentric braced frames for the application of R factor in the design process to ensure that the structure will provide the required ductility during earthquake events. Based on the results of work conducted in this paper and within the range of the investigated parameters, the following conclusions were drawn:

- For eccentric braced frames, the ductility reduction factor (R_μ) is inversely proportional to the number of storeys of the studied frames.
- The ductility reduction factor (Rμ) values increased when the number of bays decreased for the structures that have the same height.
- At the same Natural period, force reduction factor increases with the increase of the peak ground acceleration (PGA).
- The force reduction factor decreases as the length of the shear link (e) increase.
- Some earthquake records resulted in larger ductility reduction factors than other records at the same peak ground acceleration level. This is due to the fact that the response of buildings shows great variations from one earthquake to another as it depends on the frequency content and the duration of strong shaking of the used earthquake.
- The force reduction factor (R) are inversely proportional to the height of the structures, so using a single value for the reduction factor may result in uneconomic design in some cases.

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