Journal of Structural Engineering and Geotechnics,
2 (2), 11-17, Summer 2012

Seismic Performance of Torsionally Stiff and Flexible Single Story Buildings Designed Based on Iranian Seismic Code(Standard 2800)

S. A. Haj Seiyed Taghia[*a], A. S. Moghadam[b]

[a]*Assistant Professor, Qazvin Branch, Islamic Azad University, Qazvin, Iran*
[b]*Assistant Professor International Institute of Earthquake Engineering and Seismology (IIEES), Tehran, Iran*

Received 1September 2012, Accepted 28 October 2012

Abstract:

This paper examines differences in performances of a range of torsionally stiff and flexible single story buildings designed with the provisions of Iranian Standard 2800. Seismic nonlinear dynamic time history behavior of eight building models subjected to seven horizontal bi-directional design spectra compatible ground motions are investigated. These models cover a wide range of very torsionally stiff to very flexible buildings. Response parameters are element ductility demand and building story drift ratio. These criteria are appropriate indices for structural and nonstructural damages, respectively. This investigation shows that the linear static analysis of building code such as Iranian Standard 2800 is not generally adequate for structures with very low torsional stiffness.

Keywords: Torsion, Torsionally stiff, torsionally flexible, Standard 2800, Non-linear dynamic analysis, Story drift ratio, Performance level.

1. Introduction

The assessment of structural performance during past earthquakes have indicated that irregularity due to asymmetric mass, stiffness and strength distribution is one of the main reasons of structural vulnerability. Such plan-wise irregular structures are divided into two classes with low or high "torsional to translational modal frequencies ratio (Ω_R)" that are named torsionally flexible and stiff structures, respectively. The farthest and the nearest edge of building from the center of rigidity are named flexible and stiff edges, respectively. In torsionally flexible buildings (Ω_R<1), both edges generally experience more displacement compared to their symmetric building counterpart. In torsionally stiff structures ($\Omega_R \geq 1$), flexible and stiff edges, respectively experience more and less displacement compared to their symmetric building counterpart [1].

In this paper, an overview of torsional provisions and their frameworks in the 2005 NBCC [2] and torsional provisions in Standard 2800 (2005) [3] are presented first, then the capability of these torsional provisions in Standard 2800 (2005) to control the effect of torsion in torsionally flexible and stiff structures are evaluated. [3]

Corresponding Author Email:Ali.Taghia@qiau.ac.ir

2. Background

Humar et al. (2003) investigated torsional provisions in the 2005 editions of the NBCC using models similar to Fig. 1. In this model, the building floor is assumed to be infinitely rigid in its own plane. The entire mass of the structure is distributed at the floor level. The origin of the coordinate system is assumed to be at the mass center, denoted by *CM*. Lateral resisting elements are shown as hatched lines. The center of stiffness, which in the elastic range is also the center of resisting forces *CR*, is eccentric with respect to *CM* and lies at a distance from *CM*

The frameworks of the NBCC are based on equaling displacements in static analysis with dynamic analysis for flexible and stiff edges in order to derive formulas to be used in static analysis [1].

The 2005 NBCC restrict the use of the equivalent static load method of design to buildings that are relatively stiff in torsion. An alternative measure of torsional stiffness is being proposed. In the 2005 NBCC, a building with a rigid diaphragm will be considered torsionally sensitive if a ratio *B* exceeds 1.7.

Parameter *B* is determined by calculating the ratio B_x for each level x, and independently for each orthogonal direction, according to the following equation:

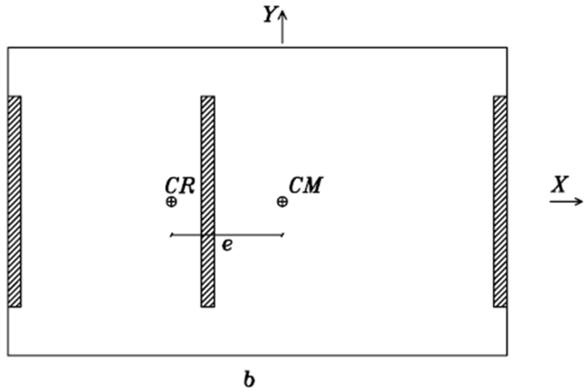


Fig. 1. General configuration of the models

$$B_x = \frac{\delta_{\max}}{\delta_{ave}} \quad (1)$$

Where δ_{max} is the maximum story displacement at the extreme points of the structure at level x in the direction of the earthquake induced by the equivalent static forces acting at a distance $\pm 0.1b$ from the centers of mass at each floor; δ_{ave} is the average of the displacements of the extreme points of the structure at level x produced by the above forces, and *b* is the dimension of floor x perpendicular to the direction of earthquakes. Ratio *B* is then taken as the maximum of all values of B_x in both orthogonal directions.

Determination of δ_{max} and δ_{ave} requires that a three-dimensional (3D) static analysis of the structure be carried out. The 2005 NBCC require that a dynamic analysis be carried out for determining the design forces whenever *B* exceeds 1.7.

For buildings where the equivalent static load method of design is permitted, the 2005 NBCC specify the following values for the design eccentricities:

$$e_{d1} = e + 0.1b \quad (2)$$
$$e_{d2} = e - 0.1b \quad (3)$$

Standard 2800 (2005) applies seismic forces in mass center for equivalent static analysis and changes mass center location equal to accidental eccentricity. This standard simulates the effect of uncertainty in mass, stiffness distribution and earthquake rotational component through the accidental torsion. This standard defines amplification factor (A_j) that is multiplied by the accidental torsion for torsionally flexible structures in order to decrease ductility demand in critical edges through increasing the strength and stiffness for lateral resisting elements.

In the following parts of this paper, building models that cover a large variation of frequency ratios are introduced first. Then these models subjected to seven recorded events with pairs of appropriate horizontal ground-motion time history components are analyzed. This research shows that Standard 2800 (2005) is not always satisfactory and the application of static analyses for torsionally flexible structures does not lead to structural safety.

3. Building models characteristics

Fig. 2 shows the architectural plan of models considered in this study to cover a wide range of torsional to lateral stiffness ratios. Dimensions of plan are 42m x10m and the story height is 3m. These models are able to represent stiff and flexible torsional behavior. The lateral stiffness of models is symmetric in the y direction. The structural system in two directions is concentric braced frame (*CBF*). Applying different configurations of braces in the x direction, eight models that cover the required range for frequency ratio are produced. Each model has brace in A, B and C axes in the y direction. In the first four models, stiffness distributions in the x direction are symmetric too. Fig. 3 shows the locations of braces, schematically.

Models are considered residential buildings with rigid floors located in high seismic zone. Loads on structures are assumed based on Iranian national building code (2004) [4]. Accidental torsion is 0.05*b*, where *b* is the dimension of the building model perpendicular to the direction of the earthquake. Using linear static analyses, the buildings are designed based on UBC97 design provisions. Locations of braces in the x direction are used to identify each models such as$O^NX^NO^N$, where letters O and X refer to frames without and with braces, respectively. Letter N shows the number of O or X

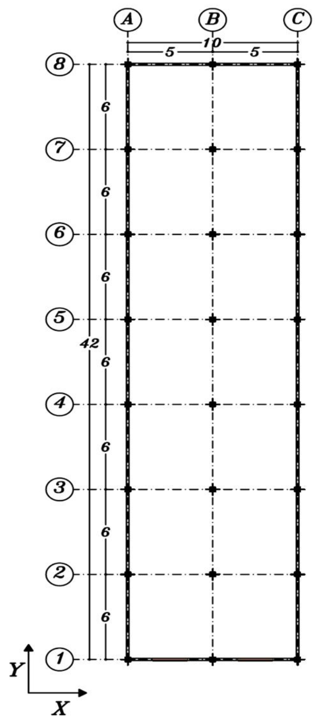


Fig. 2. The plan of the models.

frames. For all models, configurations of braces in the y direction are the same.

According to Standard 2800 (2005) torsional irregularity shall be considered to exist when the maximum story drift computed, including accidental torsion at one end of thestructure transverse to an axis, is more than 1.2 times the average of the story drifts of the two ends of the structure. If this rule is not satisfied, A_j factor should be calculated.

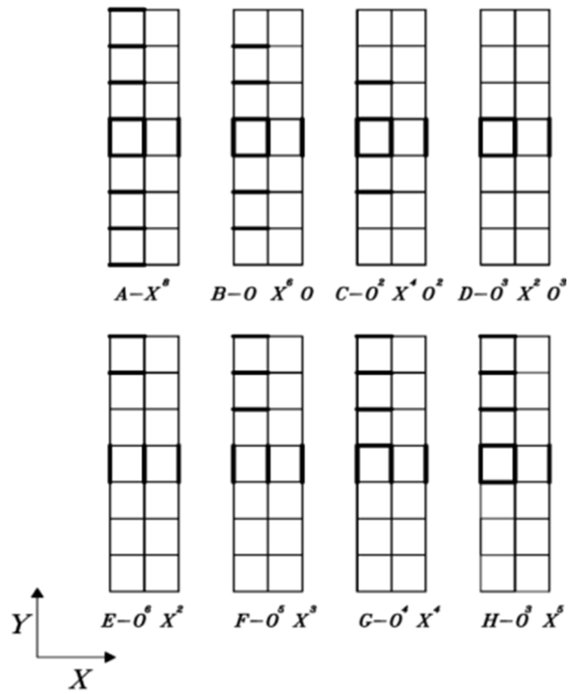


Fig. 3. Locations of braces in the eight selected models; models A to D with symmetric stiffness distribution in the x direction; models E to H with asymmetric stiffness distribution in the x direction; in all models the stiffness distribution in the y direction is symmetric

$$A_J = \left(\frac{\delta_{\max}}{1.2\delta_{ave}}\right)^2, \quad 1 \le A_J \le 3 \tag{4}$$

Where:

d_{avg} = the average of the displacements at the extreme points of the structure at Level J.

eight selected models. According to Table 1, models A and B are almost torsionally stiff while the others are torsionally flexible structures

According to the provisions of the 2005 NBCC, Table 1 shows that for torsionally stiff structures (models A and B) equivalent static analysis is allowed marginally (maximum of B is equal to1.77), but for torsionally flexible structures, equivalent static analysis is not allowed (maximum of B is equal to 3.74 in model D). d_{max}= the maximum displacement at Level J.The value of A_J need not exceed 3.0.

Table 1 shows fundamental period, rotational to translational frequencies ratio (Ω_R), parameter B, whether static analysis is allowed or not also parameter A_j for the

Finally, in according to Standard 2800 (2005) [3],Table 1 shows that for torsionally stiff structures (models A and B) amplification factor can be neglected, but for torsionally flexible structures, amplification factor is necessary (more than 1). Based on new amplified eccentricities, models are analyzed again. The designed models are used in nonlinear bi-directional dynamic time history analyses subjected to seven pairs of horizontal ground motion components.

Table 1. Fundamental period, rotational to translational frequencies ratio (Ω_R), parameter B, whether static analysis is allowed or not also parameter A_j for the eight selected models

Model	*T* for direction		Ω_{RX}	Ω_{RY}	Ω_R*	B	static analysis is allowed or not	A_j	
	X	Y						X direction	Y direction
$A-X^8$	0.13	0.20	1.12	1.69	1.41	1.45	Yes	1.05	0.70
$B-OX^6O$	0.15	0.20	0.86	1.12	0.99	1.77	No	1.35	0.71
$C-O^2X^4O^2$	0.18	0.20	0.63	0.67	0.65	2.41	No	2.09	0.75
$D-O^3X^2O^3$	0.23	0.20	0.49	0.41	0.45	3.74	No	3.37	0.84
$E-O^6X^2$	0.56	0.14	0.52	0.44	0.48	2.12	No	3.09	0.82
$F-O^5X^3$	0.53	0.18	0.57	0.54	0.56	2.29	No	3.53	0.77
$G-O^4X^4$	0.39	0.20	0.63	0.73	0.68	2.42	No	3.84	0.74
$H-O^3X^5$	0.13	0.20	0.75	0.91	0.83	2.37	No	3.50	0.72

Note: *Ω_R is average value for two directions

Table 2. The selected records and their characteristics

No.	Earthquake	Station	Date	Magnitude	Distance from source(km)	PGA (g)
1	Northridge	24087 Arleta Nordhoff Fire Sta	1994/01/17	6.7	9.2	0.344
2	Northridge	24400 LA – Obregon park	1994/01/17	6.7	37.9	0.355
3	Northridge	90014 Beverly Hills -12520 Mulhol	1994/01/17	6.7	20.8	0.314
4	Victoria Mexico	6604 Cerro Prieto	1980/6/9	-	-	0.304
5	N.Palm Springs	12149 Desert Hot Springs	1986/7/8	6	8	0.331
6	N.Palm Springs	5071 Morongo Valley	1986/7/8	6	10.1	0.395
7	Whittier Narrows	24400 LA – Obregon park	1987/10/4	5.3	-	0.374

4. Nonlinear dynamic time history analyses

For a better probabilistic evaluation, seven events recorded on soil profile type II based on Standard 2800 (2005) and high seismic zone are selected from Pacific Earthquake Engineering Research Center Database (PEER) [5]. Table 2 shows the selected records and their characteristics.

For each pair of horizontal ground motion components, the square root of the sum of the squares (*SRSS*) of the 5 percent-damped site-specific spectrum of the scaled horizontal components is constructed. The motions are scaled such that the average value of the *SRSS* spectra does not fall below 1.4 times the 5 percent-damped spectrum of the design-basis earthquake in the period range of $0.2T$ to $1.5T$ seconds. Two components of each pair of time histories shall be applied simultaneously to the models. Fig. 4 shows the *SRSS* spectra and its average. There are satisfactory compatibility of frequency content between the scaled average value of the *SRSS* spectra and 1.4 times the design spectra of Standard 2800 (2005) in the period range of interest (Fig. 5). SAP2000 (2008) is used for conducting nonlinear dynamic time history analyses [6].

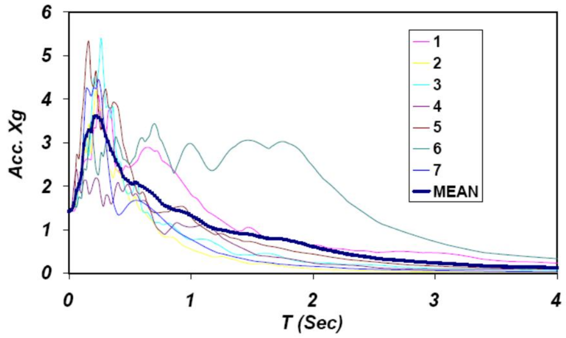


Fig. 4. The *SRSS* spectra and its average

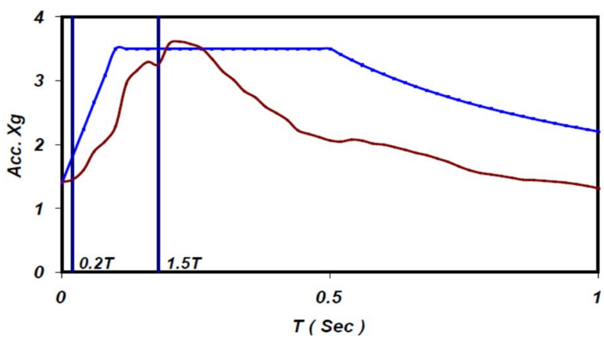


Fig.5. Comparison of the scaled average value of the *SRSS* spectra with 1.4 times the design spectra of Standard 2800 [3]

5.Dynamic nonlinear analyses assumptions

Some of the main modeling and analyses assumptions are:
1- The effect of accidental eccentricity is considered by change in mass distribution in such a way that the center of mass displaces by 0.05*b*, where *b* is the dimension of the building model in the direction of the structural eccentricity.
2- The walls are assumed isolated from the frames, thus infill effects have been neglected.
3- The proportional damping is assumed with damping ratio equal to 5 percent at two periods of 0.1 and 1 second.
4- Plastic hinge model considering axial load-moment interaction (*PMM*) is assigned to the middle and two edge columns and axial plastic hinge model (*P*) is assigned to braces based on FEMA356 [7]. Fig. 6 shows force-displacement relation for brace, schematically.
5- Analysis type is nonlinear direct integration time history.
6- The time step in analysis is considered as 0.0025 second, for convergence matters.
7- Geometric *P*-D nonlinear effect is considered.
8- Each analysis is performed initially for gravity load, then seismic analysis is started from the state at the end of previous analysis. The parameters of interest shall be calculated for each time history analysis. Because seven time-history analyses are performed, the average values of the response parameter of interest should be used.

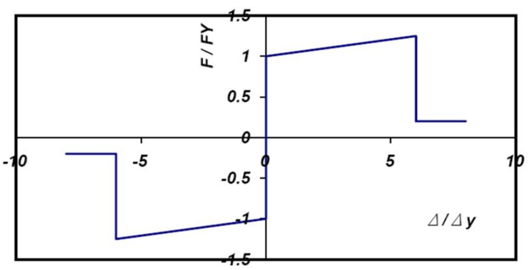


Fig. 6. Force-displacement relation for brace, schematically

Response parameters considered here are ductility demand in plastic hinge and story drift ratio. The story drift ratio is calculated in both flexible and stiff edges of each orthogonal direction.

6. Nonlinear analyses results

For torsionally flexible or stiff "symmetric" models (A to D) in the x direction, in accordance with Fig. 7,the provisions of Standard 2800 (2005) [3] have adequate efficiency in limiting story drift ratio for critical edges (to a value less than 0.68%).
For models E and F in the x direction, story drift ratios (2.95%, 2.58%) are greater than drift limit. These models (E and F) are representative of very low torsional stiffness structures (Ω_{RX} are equal to 0.52 and 0.57, respectively) and distribution of their resisting elements are asymmetric in plan. Therefore, the efficiency of the provisions of Standard 2800 (2005) for drift limit in such models are questionable. It is concluded that drift has been increased with decreasing parameter Ω_{RX}.
In considering drift, these provisions are only adequate for models G and H with Ω_{RX}>0.63 (maximum drift of 1.28% in model G).

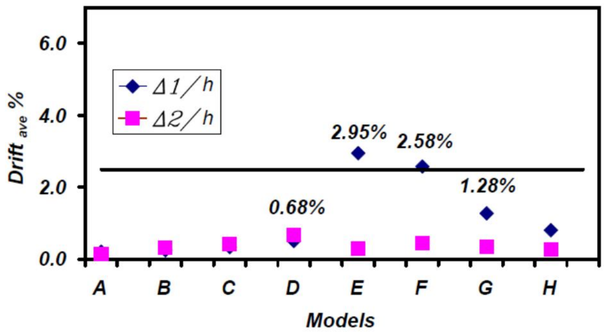


Fig. 7. Average maximum story drift ratios for the flexible and stiff edges in the x direction

Average maximum story drift ratios for the flexible and stiff edges in the y direction are shown in Fig. 8. All the structures are symmetric in the y direction. Conclusion is similar to those obtained from Fig. 7 for symmetric cases. Therefore, the provisions of Standard 2800 (2005) [3] have adequate efficiency in limiting drift for critical edges (maximum drift of 0.62% in model F).

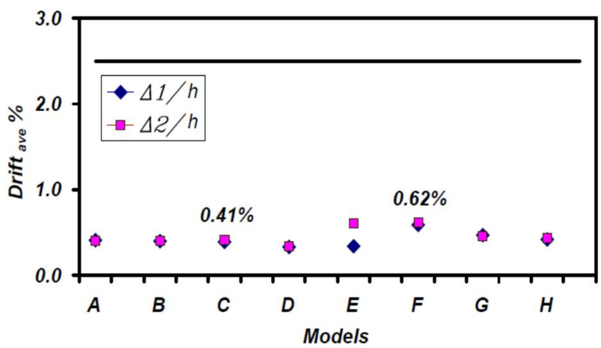


Fig. 8. Average maximum story drift ratios for the flexible and stiff edges in the y direction

Fig. 9 presents overall evaluation of the frameworks and code provisions in limiting drift of all edges at each direction. It is realized that structural behavior in the x direction is generally dominant with respect to overall structural behavior. Therefore, the conclusions in Fig. 7 are also valid here.
Fig. 10 shows the average maximum floor rotations for the eight models. In models with symmetric mass and stiffness distribution (A to D), floor rotation caused by accidental torsion is small (less than $0.51\text{x}10^{-3}$ radian), but for models with asymmetric stiffness distribution, floor rotation is larger and it is increasing, with decrease in

parameter Ω_R. Model E, with respect to brace configurations, has minimum frequency ratio and its floor rotation is maximum (2.30x10^{-3} radian).

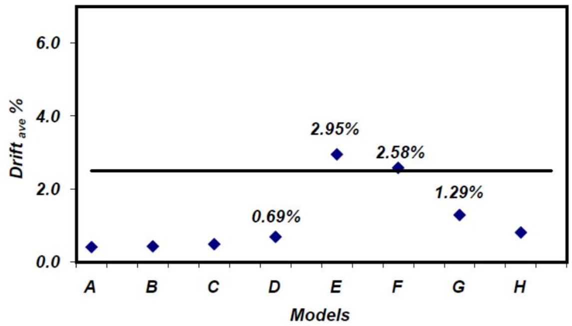


Fig. 9. Average maximum story drift ratios of all edges and each direction

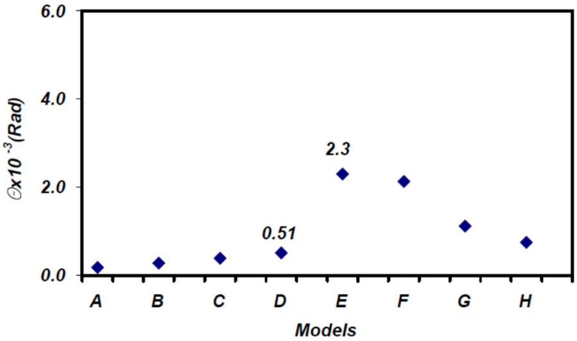


Fig. 10. Average maximum floor rotations for the eight selected models

Fig. 11 shows average maximum ductility demand (μ) in plastic hinges for the eight selected models. The ductility demand is defined here by subtracting elastic deformation from plastic deformation divided by yield deformation. Fig. 11 shows for symmetric models (A to D), ductility demand is less than 2, but for asymmetric models (E to H), ductility demand is less than 4. Standard 2800 (2005) considers life safety performance level (μ<6) for a residential building. It is concluded that ductility demand in the selected models, especially torsionally stiff models, is less than acceptable limit. Therefore, the provisions of Standard 2800 (2005) [3] have appropriate efficiency in limiting ductility demand.

7. Conclusions

It should be emphasized that the results of this study are valid only within the framework of presented modeling and analysis assumptions such as: soil profile type, earthquake and excitation angle, behavior of material, analysis type, and capability of software, sizes of steel shapes, geometry and the number of models.

The main results of this study are as follows:

1- For symmetric models, including both torsionally flexible and stiff structures, the provisions of Standard 2800 (2005) for static analysis have adequate efficiency to limit story drift ratio for critical edges. Also for asymmetric cases with low torsional stiffness those provisions are suitable. But for asymmetric cases with very low torsional stiffness, the efficiency of the provisions is questionable. It is concluded that drift has increased with decrease in parameter Ω_R.

2- For symmetric models, floor rotation caused by accidental torsion is small, but for asymmetric models, floor rotation is larger and it is increasing with decrease in parameter Ω_R.

3- It is concluded that ductility demand in the selected models, especially torsionally stiff models, and is less than its limitation. Therefore, the provisions of Standard 2800 (2005) have appropriate efficiency to limit ductility demand.

As a final conclusion, parameter A_j in the provisions of Standard 2800 (2005), generally increases the stiffness of structure and therefore, seismic forces due to the shape of design spectra in Standard 2800 (2005) for torsionally flexible structures. In this study the provisions of Standard 2800 (2005) were able to limit ductility demand, but they did not limit drift to the allowable level for very low torsional structure with Ω_R<0.68. Based on these observations four strategies can be considered in future studies:

1- Limit the use of linear static analysis for structures with at least a minimum frequency ratio (Ω_R).

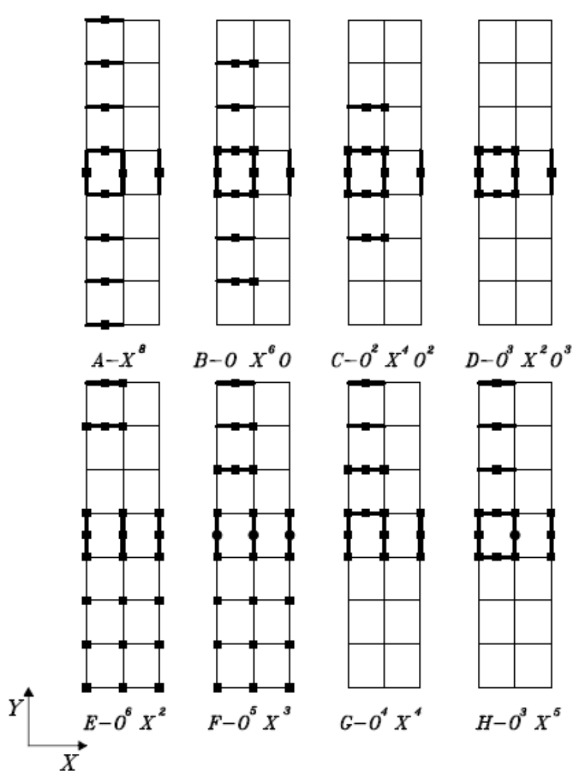
2- With appropriate design, limit frequency ratio (Ω_R) of the building to mentioned bounds.

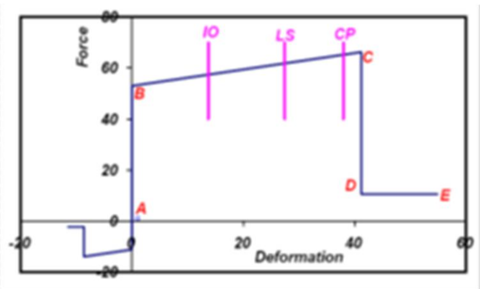
3- Proposes modified formulas for increasing the structural stiffness in order to reduce drift.

4- Reduces R factor (i.e. force modification factor related to ductility).

Acknowledgments

This study was conducted to evaluate efficiency the provisions of Standard 2800. The authors gratefully acknowledge the contribution of the members of the International Institute of Earthquake Engineering and Seismology, IIEES and Qazvin Islamic Azad University, QIAU, through critical comments and suggestions.





symbol	Ductility demand	Performance level
■	< 2	B
●	2-4	IO
×	4-6	LS
◆	> 6	CP,C,D,E

Fig. 11. Average maximum ductility demand for the eight selected models.

References

[1] Humar, J., Yavari, S., and Saatcioglu, M. 2003. Design for forces induced by seismic torsion. Can. J. Civ. Eng. 30

[2] National Building Code of Canada (NBCC) 2005. Associate Committee on the National Building Code. National Research Council of Canada, Quebec

[3] Standard No. 2800. 2005. Iranian Code of Practice for Seismic Resistant Design of Buildings, Third Revision, Building and Housing Research Center, Iran

[4] Standard No. 519, Part 6. 2004. Iranian National Building Code for Structural Loadings, Ministry of Housing and Urban Development, Tehran, Iran, Uniform Building Code (UBC). 1997.

[5] PEER Strong Motion Database, peer.berkeley.edu website

[6] SAP2000, Structural Analysis Program 2008. Computers and Structures Inc. Berkeley, California, U.S.A.

[7] FEMA 356. 2000. Prestandard and Commentary for the Seismic Rehabilitation of Buildings, Federal Emergency Management Agency